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13 forecasting
15 socioeconomics
16 user needs
21 facilities design
51 transportation safety
53 vehicle characteristics
54 operations and traffic control
55 traffic flow, capacity, and measurements

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Transportation Research Record 1280

Contents

Foreword

This combined Transportation Research Record is the first to be published in subject areas related to operations and traffic control. The papers in this Record cover a broader scope than has been the case for previous Records. TRB trusts that publishing larger Records containing more papers related to more general themes will be of greater benefit to the reader than publishing greater numbers of Records covering limited subject areas.

The papers from the 1990 Annual Meeting of TRB contained in this Record are related by their pertinence to traffic operations, geometric design, and traffic management. Readers will find papers of interest in the areas of traffic mitigation ordinances; demand management techniques such as staggered work hours and ride-sharing; transportation system management techniques; HOV-lane operations, effectiveness, and enforcement. Readers who have an interest in geometric design or the effects of geometrics will find papers on evaluation of urban interchange design; passing on two-lane, two-way highways; intersection sight distance related to trucks; use of double-line pavement markings; estimating road design safety factors; ramp terminals sight distance issues; operating speeds on two-lane rural highways when dry and wet; safe operating speeds of trucks on ramps; and sight distances at vertical curves.

Status of Traffic Mitigation Ordinances

Marta J. Jewell, Raymond H. Ellis, and Richard L. Oram

Traffic mitigation ordinances have emerged as a compelling new strategy for reducing automobile congestion related to commuting. Development of the ordinance approach is rooted in a range of transportation policies and activities that have sought to achieve relief from congestion through programs and facilities intended to change the demand on the transportation system. Major activities and programs led to the traffic mitigation ordinance approach. The current status of the use of traffic mitigation ordinances to reduce traffic congestion is reported. Findings are based on a review of 24 traffic mitigation ordinances, which have been adopted or are in some stage of development in 20 jurisdictions throughout the United States. Although the state of California appears to lead the way in development and adoption of traffic mitigation ordinances, a review is also included of ordinances in Arizona, Maryland, New Jersey, Virginia, and Washington. Major components of traffic mitigation ordinances are identified and discussed and the jurisdictions' approaches for each component are compared. Traffic mitigation ordinances hold promise as a widely applicable tool for managing traffic congestion. Limited empirical evidence exists to date on the actual effectiveness of ordinances because of the limited time of their application. Some areas, such as Pleasanton, California, have demonstrated that the ordinance has been effective in maintaining reasonable traffic conditions in spite of increased development and employment. The ordinance concept appears appropriate for increased emphasis, promotion, and study.

Traffic mitigation ordinances, also referred to as transportation demand management (TDM) ordinances, have emerged as a compelling new strategy for reducing automobile congestion related to commuting. The approach is an outgrowth of a range of initiatives, pressures, and precedents affecting urban transportation over the past 15 years.

EMERGENCE OF TRAFFIC MITIGATION ORDINANCES

The concept of transportation system management (TSM) can be traced to a joint UMTA and FHWA policy promulgated in September 1975. This initiative changed the metropolitan transportation planning process by requiring development of a short-range, low-capital, management-oriented strategy as a companion to traditional long-range planning products. An annual TSM element was required in the metropolitan area's transportation improvement plan.

M. J. Jewell and R. H. Ellis, KPMG Peat Marwick, 8150 Leesburg Pike, Suite 800, Vienna, Va. 22182. R. L. Oram, R. L. Oram Associates, 160 West 87th Street, Suite 1–D, New York, N.Y. 10024.

UMTA and FHWA initiated substantial new demonstration and technical assistance programs. TSM-related demonstrations included measures to (a) improve the capacity of the existing transportation system through modifications, such as preferential freeway and arterial lanes for high-occupancy vehicles (HOVs) and traffic signaling improvements, and (b) change demand on the existing transportation system through measures, such as ridesharing programs, transportation brokerage, and other management strategies. Employers were found to significantly influence the success of alternative commute programs through measures such as providing free parking or comparable incentives for people who use transit or ridesharing, offering flextime, and appointing employee transportation coordinators.

Defined as a public sector planning requirement, TSM became a management approach for pursuing near-term action on persistent traffic congestion. It stressed coordination and interagency activities and, by emphasizing alternate commute and work hour strategies, involved the business community.

The role of the private sector in transportation began to expand as the Reagan administration's policies were implemented. Greater appreciation of transit's economic constraints supported both increased use of private operators and recognition that continued pursuit of the peak-hour commuter market was not a cost-effective strategy for transit. It became clear that reducing demand would be far more cost-effective than increasing supply.

Transportation brokerage, another federally created transportation concept, stimulated interest in TDM. Brokerage was hoped to generate a new model for a more market-based transit organization that stressed the idea of market niches and multiple services and tended to champion paratransit and ridesharing. Brokerages took advantage of interests and opportunities to reduce traffic by working with employers and developers.

As interest in ridesharing stabilized or diminished in the 1980s, many brokerages have shifted their attention to attempting more directly to influence traffic demand. Maturing of the brokerage concept and ridesharing profession has spawned the Association for Commuter Transportation (ACT), an active national organization working to build support for TDM strategies.

By the mid-1980s, the concept of the transportation management association (TMA) emerged as a new mechanism for increasing corporate involvement in urban transportation issues, specifically urban traffic congestion. TMAs pursue a cooperative, consensus-based strategy to gain a common view of the causes of traffic congestion and to arrive at joint solutions. Benefits of traffic congestion relief are generally shared among the entire community and only become appreciable when a critical mass of employers are involved.

TDM ordinances in some cases have emerged as a way to pursue the same ends as a TMA—widespread congestion relief—without dependence on leadership and with a regulatory structure that affirms the continuation of the process. Many ordinances can be traced to a task force or other TMA-type endeavor that generated common understanding of the problems and thereby successfully garnered business community support for the ordinance strategy.

Transportation strategies put in place during the 1984 Los Angeles Olympics had a notable effect on the emergence of the area-wide Regulation XV adopted by the South Coast Air Quality Management District (SCAQMD) and now being introduced in the four-county Los Angeles region. Although the city feared regional gridlock from the major increase in traffic that the Olympics would bring, the mitigation measures introduced cooperatively with the business community, with emphasis on short-term ridesharing and flextime, made the traffic conditions experienced better than normal. The need and means for sustaining these achievements in transportation efficiency received substantial attention by the city of Los Angeles and other parties.

TRAFFIC MITIGATION ORDINANCE APPROACH AND APPLICATION

Major components of traffic mitigation ordinances and issues in the development and application of ordinances as a means of reducing traffic congestion are discussed on the basis of a review of adopted or drafted ordinances in the following jurisdictions:

- Alexandria, Virginia, adopted May 1987;
- Bellevue, Washington, non-central business district (non-CBD) ordinance, adopted May 1987; Interim Traffic Ordinance (CBD), adopted September 1988;
- Berkeley, California, in draft;
- Concord, California, adopted October 1985, revised 1987;
- Contra Costa County, California, Pleasant Hill BART Station Area ordinance, adopted June 1986; county-wide ordinance, adopted October 1987;
- El Segundo, California, adopted November 1985;
- Golden Triangle area, Santa Clara County, California, model ordinance in draft;
- Maricopa County, Arizona, effective December 31, 1988;
- Montgomery County, Maryland, adopted November 1987;
- North Brunswick, New Jersey, adopted October 1987;
- Oxnard, California, in draft;
- Pasadena, California, adopted June 1986;
- Pima County, Arizona, adopted in five jurisdictions in April and May 1988;
- Placer County, California, adopted May 1982;
- Pleasanton, California, adopted October 1984;
- Sacramento, California, employer and developer ordinances in draft;
- San Buenaventura, California, adopted July 1988;
- San Rafael, California, adopted July 1983;
- Seattle, Washington, Major Institutions Ordinance adopted 1983. Land Use Code revised in 1985; and
- SCAQMD, Los Angeles, California, Regulation XV, adopted December 1987 and implemented July 1988.

Traffic Management Strategies

Historically, there have been three general strategies for dealing with traffic congestion problems:

- Transportation Facility and Development (TFD)—developing new highway systems, transit services, or equipment, such as freeway on and off ramps.
- Transportation System Management (TSM)—adjustments to the existing transportation system to improve its capacity and allow traffic to flow better, such as improved signalization, change in direction of traffic flow, and establishment of HOV lanes.
- Transportation Demand Management (TDM)--development of programs and construction of facilities to change demand on the system by changing user behavior. TDM programs include information and incentives to encourage employees to travel by means other than the single-occupant vehicle (SOV) during peak travel periods. TDM measures include flexible work hours, ridesharing, and preferential vanpool parking. TDM facilities include vanpool staging areas, transit shelters, and bicycle lockers.

No single approach can solve a jurisdiction's traffic congestion problems. Each strategy should be considered within a broader transportation and land use strategy including growth management policies and zoning to provide development patterns that will reduce overall automobile use.

TDM Approaches

A number of approaches are currently being used to reduce traffic congestion by changing user behavior, including

- Voluntary. Employers or developers start a TDM program voluntarily, frequently in the form of a TMA.
- Incentive. A local ordinance is adopted that offers benefits (such as reduced parking) to developers to encourage TDM program implementation.
- Voluntary-Mandatory. An ordinance is adopted initiating a voluntary TDM program that becomes mandatory if specified rates of progress in traffic reduction do not take place. An example is the model ordinance being developed by the Golden Triangle Task Force in Santa Clara County.
- Mandatory. A TDM program is required by local ordinance or administrative guidelines. The program may involve the following:
  —Developer conditions of specific demand management strategies required as conditions for approval of development permits (e.g., Bellevue and Contra Costa County), which may be recorded as conditions, covenants, and restrictions on use of the property and included in leases.
  —Employer requirements for employers that meet specified criteria for implementing TDM programs to achieve

desired levels of use of commute alternatives (e.g., Placer and Contra Costa counties) or to reduce vehicle trips by a certain percentage (e.g., city of Pleasanton).

This study focuses on local ordinances requiring TDM measures to reduce traffic congestion. However, many of the ordinances reviewed contain voluntary components for certain sizes of employers and development categories.

Goals

TDM ordinances typically set a goal or standard that employers or developers must achieve to mitigate traffic congestion and improve air quality.

Participation Rate

This goal is measured as the percentage of an employer's workforce expected to commute to and from work by non-SOV mode and may be expressed as a decrease in the percentage of SOV commute trips. Emphasis is on mode change rather than change in travel time or peak shift.

Achievement of participation rate goals can be easily calculated from employer surveys. Jurisdictions using this goal must compile data on preprogram non-SOV driving rate.

Jurisdictions with participation rate goals include Contra Costa County (maximum, 65 percent SOV), Pima County (25 percent non-SOV by the third year), Sacramento (35 percent non-SOV), San Buenaventura (55 percent non-SOV), Alexandria (30 percent non-SOV), and Bellevue (18 percent for specified land use districts outside the CBD). The model ordinance being developed by the Golden Triangle Task Force in Santa Clara County uses a participation rate goal of 24 percent non-SOV by 1992 and 35 percent non-SOV by 2000.

Vehicle Trip Reduction

Vehicle trip reduction is expressed as the percentage reduction in vehicle trips, generally as a result of decreasing SOV commute trips. Program results can be easily translated into effect on traffic conditions (i.e., percentage change in traffic volume) by using vehicle trip reduction figures.

A baseline must be established against which reduction in vehicle trips can be measured. The baseline can be (a) the number of vehicle trips that would occur if all commuters drove alone, or (b) the number of vehicle trips before the program was implemented. Most jurisdictions use the number of trips that would occur if all commuters drove alone as the baseline because this number is simpler to determine.

The wide range in vehicle trip reduction rates required in various ordinances can be attributed to how the baseline is computed. The goal for a city, such as Seattle (50 percent in the major institutions ordinance), in which the baseline is calculated as the number of trips that would occur if all commuters drove alone, will be higher than the goal established in a jurisdiction using the actual trip rate as the baseline, such as Maricopa County with a vehicle trip reduction goal of 5 percent in each of the first two program years. El Segundo also uses the trip reduction goal (20 percent) in its ordinance.

Peak-Hour Vehicle Trip Reduction

Jurisdictions may focus on a reduction in vehicle trips during specified peak hours, which can result from (a) increases in use of commute alternatives (ridesharing or transit), or (b) shifts to off-peak-hour travel (staggered work hours).

Ordinances adopted by the city of Pleasanton and Placer County require reductions in peak-period employee commute trips of 45 and 25 percent, respectively.

Level of Service (LOS)

Another strategy focuses on the desired traffic conditions on specified road facilities and may specify maintenance of existing LOS ratings or prevention of deterioration of traffic conditions.

Measuring LOS goal attainment requires a traffic monitoring program. However, the measured results may not accurately reflect the program's effects because the program could result in a large change in SOVs while traffic remains high because of pass-through and noncommuter traffic.

Bellevue's CBD interim traffic ordinance has two LOS goals: (a) to maintain p.m. peak-hour LOS D on any portion of the street system affected by proposed new development, and (b) to permit no further degradation of traffic conditions on portions of a street system affected by proposed development that is currently at LOS E or worse. Oxnard established a goal to maintain LOS C at city intersections. Berkeley's ordinance includes maintaining LOS D on downtown streets and achieving a participation rate of 40 percent non-SOV.

The city of San Rafael established maximum p.m. peak-period trip allowances for various land uses, such as 0.7 trips per small residential unit, 2.6 trips per 1,000 ft^2 of general office space, 1 trip per 1,000 ft^2 of industrial space, and 3.3 trips per 1,000 ft^2 of retail space.

Montgomery County uses participation rates of 25 percent transit for existing employers, 30 percent transit for new employers, and 5 percent walk and average automobile occupancy rates of 1.3 for all employers.

North Brunswick uses goals based on peak-period trips as a percentage of workforce (maximum 60 percent overall and maximum 40 percent within any 15-min interval of peak period).

Regulation XV, adopted by SCAQMD, established goals of 1.3, 1.5, or 1.75 average vehicle ridership (employees per vehicle trip), depending on area.

Ordinance goals are frequently staged over several years. This practice reflects an understanding that programs require start-up time and time to change employee commute habits. The ordinance adopted by the city of Pleasanton has an overall goal of 45 percent reduction in peak-period employee commute trips. Required progress toward the overall goal is staged over a 4-year period (15 percent in Year 1, 25 percent in Year 2, 35 percent in Year 3, and 45 percent by Year 4). The model ordinance being developed by the Golden Triangle Task Force has a short-term goal of 24 percent nonsolo driving by 1992 and a long-term goal of 35 percent by 2000. Ordinances in Pima and Contra Costa counties also include staged goals.

The ordinance adopted in 1986 by Contra Costa County for the Pleasant Hill Bay Area Rapid Transit (BART) station area established a primary goal of no more than 65 percent of all employees commuting in SOVs, with an alternative goal, for employers demonstrating that the primary goal is not feasible, of no more than 55 percent of all employees commuting during peak periods in SOVs. Montgomery County established a less stringent goal for existing employers (25 percent of employees commute non-SOV) than for new developments (30 percent commute non-SOV). The ordinance adopted in San Rafael in 1983 established peak-period trip allowances for various types of development.

Goals sometimes vary for geographic areas within a jurisdiction. For example, in Contra Costa County, higher goals are specified for the I-80, I-680, and State Route 24 corridors than for the rest of the county. Additionally, a separate ordinance was developed for the Pleasant Hill BART station area.

The ordinance adopted in Pasadena in 1986 does not specify measurable goals. General goals are to encourage use of alternate modes and work hours. However, this ordinance requires developers to take specific TDM measures rather than providing a menu of TDM options.

Scope

TDM ordinances may apply to employers (existing or new), developers and property owners, office or industrial complexes, retail developments, and residential developments.

An equity issue arises in determining the groups held responsible for reducing congestion through the traffic mitigation ordinance. Existing employers may argue that new developers and employers should be more responsible or solely responsible for mitigating traffic congestion because the new development causes the traffic conditions to move from acceptable to unacceptable. New developers or employers may argue that existing employers should be equally responsible because they contribute to the overall congestion problem.

The landmark Pleasanton area-wide employer TDM ordinance adopted in 1984 can be directly traced to municipal deliberations with the developers of a major new suburban business park. Faced with traffic mitigation approval conditions, the developers argued successfully that for the process to succeed, standards and requirements should be imposed on all employers, not just the developer or the employers residing in the new development.

The ordinances reviewed can be categorized in terms of scope as follows:

- Ordinances applicable to new and existing employers and new developments—Concord, Contra Costa County, El Segundo, Golden Triangle area, Montgomery County, North Brunswick, Oxnard, and Placer County;
- Ordinances applicable only to employers (new and existing)—Maricopa County, Pima County, Pleasanton, Sacramento (draft employer ordinance), and SCAQMD; and
- Ordinances applicable only to new developments and substantial expansions of existing structures—Alexandria, Bellevue, Berkeley, Contra Costa County (Pleasant Hill BART station area), Pasadena, Sacramento (draft developer ordinance), San Buenaventura, and San Rafael.

Seattle's developer ordinance applies to new and existing developments.

Most jurisdictions exclude residential developments because generally it is easier to initiate transit and ride share incentive programs at the destination rather than origin of commuter trips. Exceptions are as follows:

- Bellevue's non-CBD ordinance includes requirements for new development of residential or multiple-family dwellings with at least 16 units,
- Contra Costa County's county-wide ordinance includes requirements for residential projects with at least 13 dwelling units,
- North Brunswick's ordinance includes requirements for new residential developments with at least 20 units,
- Concord's ordinance includes requirements for new residential developments with at least 100 units, and
- Alexandria's ordinance includes requirements for new residential developments with at least 250 units.

In general, retail developments are also excluded from traffic mitigation ordinances. San Rafael's and Alexandria's ordinances are exceptions.

Decisions regarding which groups will be subject to ordinance requirements are determined, in part, by the objectives of the ordinance. If the objective is to reduce the traffic impacts of new development, only new developers and new employers may be affected. If the objective is to maintain existing traffic conditions, additional traffic from the new development will have to be offset by a reduction of vehicle trips from existing development. Similarly, if the objective is to improve traffic conditions, the ordinance must apply both to new and to existing employers.

Ordinance requirements typically vary by the size of the employer or developer with thresholds based on gross square feet or number of employees. Small employers, above the minimum threshold, may be subject only to informational requirements, such as providing information regarding the number and commute habits of their employees and providing their employees with information on alternative commute modes and alternative work hour programs. Larger developers and employers may be required to develop and implement programs with specific TDM measures or to select TDM measures from a menu of options.

Some ordinances phase employers or developers into the program over time. The requirements of the county-wide ordinance adopted in Contra Costa County were applied to new employers and project sponsors as of the effective date of the ordinance (November 27, 1987). Existing employers and project sponsors were not subject to the ordinance requirements until 1 year later.

SCAQMD's Regulation XV provides for phasing in of requirements—July 1, 1988, for employers with at least 500 employees; January 1, 1989, for employers with 200 to 499 employees; and January 1, 1990, for employers with 100 to 199 employees.

Maricopa County will phase employers into ordinance requirements on the basis of the number of employees between December 31, 1988, and December 31, 1989.

Geographic Coverage

Traffic mitigation ordinances can apply throughout a jurisdiction or to selected areas, depending on the ordinance's goals. Perceived equity is a potential advantage of applying the ordinance requirements jurisdiction-wide. The objective of Montgomery County's ordinance is to permit greater development in the Silver Spring CBD and therefore it only applies to that portion of Montgomery County. Other ordinances, such as the one adopted city-wide in Alexandria, are intended to reduce traffic in the entire city.

Jurisdiction-wide application may have greater impact on commuters' travel between areas than would application to selected areas. Applying ordinance requirements to critical growth and traffic congestion areas only may allow stricter TDM. To obtain political support for programs that focus on highly visible traffic congestion problems may also be easier.

The majority of ordinances reviewed have been adopted jurisdiction-wide. Jurisdictions with ordinances covering selected areas only include

- El Segundo—applicable to the city's commercial and manufacturing zones,
- Montgomery County—applicable to Silver Spring CBD,
- Placer County—applicable to the unincorporated portion of South Placer implementation area,
- San Buenaventura—applicable to Arundel office, commercial and retail areas, and
- San Rafael—applicable to Northgate activity center overlay district.

Bellevue adopted two ordinances, one for the CBD and one for specified land use districts outside the CBD. Contra Costa County adopted an ordinance applicable to the redevelopment area covered by the Pleasant Hill BART station area specific plan and another ordinance applicable to the rest of the county.

Ordinance Requirements

Ordinances typically contain four types of requirements:

- Data collection, survey, and report requirements;
- Information dissemination;
- Designation of transportation coordinator; and
- Development of traffic mitigation program.

Many ordinances establish requirement thresholds. Requirements are generally more stringent for larger employers and developers. Small employers may be subject only to data collection, survey, and information dissemination requirements.

In Pleasanton, all employers must annually submit survey information to the city to establish commute pattern data and to provide carpool and vanpool matching information. Employers with at least 10 employees are also required to develop and implement an employee information program. Employers of at least 50 employees must appoint a workplace coordinator and develop and implement a TDM program to achieve target reductions in peak-period traffic.

Data Collection, Survey, and Report Requirements

Most ordinances require employers to annually collect and submit information to the jurisdiction regarding employee commute characteristics, including the numbers of employees beginning and ending work during designated peak periods, employees commuting by various means, and employees participating in alternative work hour programs. Pleasanton and North Brunswick established minimum response rates for the employee surveys.

Some jurisdictions develop and distribute standardized survey and report forms to employers to increase participation and aid in survey tabulation and analysis. Ideally, employers conduct an initial survey before implementing the program to establish a baseline for measuring progress in achievement of ordinance objectives. The annual survey is then used to assess progress.

Information Dissemination

Most ordinances also require employers to provide information on alternative commute mode options, alternative work hour programs, and travel reduction measures to employees. Typically, employers are required to provide written information on an annual basis to all existing employees and to all new employees on the date of hire. Some ordinances require employers or property owners to display alternate commute mode information in common areas such as the lobby or cafeteria. Information (brochures) is typically provided by the jurisdiction, local rideshare matching agency, or local transit agency.

Seattle's ordinance requires property owners to construct permanent commuter information centers and to conduct semiannual promotions of the TDM program (2-hr to full-day commuter fairs, depending on the size of the development). City and METRO staff assist in conducting these promotions.

Designation of Transportation Coordinator

Many ordinances require large employers and developers to designate a transportation coordinator to take responsibility for implementing, monitoring, and reporting on the progress of the travel reduction program. The transportation coordinator may also represent the employer or complex on a transportation management task force. Property owners of large complexes may be required to appoint a complex coordinator who will be responsible for this function for all small employers within the complex.

Development of Traffic Mitigation Program

Requirements to develop, submit, and implement a TDM program designed to achieve the ordinance objectives are typically applicable only for large employers or developers. Employers are generally allowed to select a specified number of activities from a menu of options, including

- Instituting flextime or compressed work weeks,
- Establishing shuttle services,
- Developing ridesharing programs,
- Subsidizing transit,
- Subsidizing ridesharing,
- Providing preferential parking for rideshare vehicles,

- Providing loading and unloading areas for rideshare and transit vehicles,
- Providing amenities for commuters walking or bicycling to work,
- Permitting employees to work at home or to telecommute.

Menu options for an ordinance establishing developer conditions of approval may be more capital-related (i.e., involving construction of shelters, loading and unloading areas for car and vanpools, bicycle racks, showers, and lockers), whereas the menu options for employer ordinances tend to be more program-related (i.e., involving alternate hours of work, rideshare matching, and subsidies).

Some ordinances also provide options for financing transportation service improvements and operations to meet the ordinance requirements. The draft Sacramento developer ordinance includes the following options for developments within 1,320 ft of an existing or designated bus route or light rail station: (a) agreement to pay all or part of the cost of land, construction, and maintenance of transit center or station; and (b) agreement to pay a one-time transit operating subsidy to the Sacramento Rapid Transit District.

Bellevue dictates specific TDM measures for specific developments, such as (a) preferential parking during peak periods for registered car and vanpools; (b) financial incentives for employees commuting by car, vanpool, and transit; and (c) a taxi-script system of low-cost rides home for employees who miss their bus, car, or vanpool because of employer requirements or emergencies.

Pasadena's ordinance mandates specific TDM measures for new developments, including preferential carpool parking (10 percent), commuter matching services, bicycle parking, and car and vanpool loading areas.

The menu approach is more defensible politically than is the requirement of any one specific action. An individual action may not be appropriate to all employers, for reasons such as business type, size, location, or corporate culture and may only be associated with a nominal reduction in trips. The menu approach is more compelling, as few can object to the principle that employers should promote traffic reduction or deny that at least some actions on a comprehensive list are appropriate. Most ordinances require affected employers to submit an annual summary report describing the transportation management measures implemented and the program's progress.

Parking Reduction Options

Some ordinances reduce parking requirements for developments that achieve a specified level of trip reduction through implementation of TDM strategies. This option is seen as an incentive or reward for compliance. Sacramento and Pasadena ordinances include a parking reduction option for developers.

Program Management

Three groups are generally involved in implementation and management of TDM ordinances:

- Public-private task force,
- Jurisdiction, and
- Employer or developer.

Public-Private Task Force

Many jurisdictions have established a public-private task force to provide policy guidance and to assist jurisdiction staff in managing the program. Typically, task forces are composed of local jurisdiction management and representatives from large employers or developers, local transit authorities, and regional agencies and associations. Task force responsibilities may include one or more of the following:

- Serving as advisory body to local jurisdiction staff,
- Establishing guidelines for program implementation,
- Reviewing employer or developer TDM programs,
- Mandating revisions to TDM programs when results are not being achieved,
- Monitoring program performance and recommending changes,
- Serving as a hearing board for appeals.

In Pima County, a regional task force has been formed, consisting of one representative of each participating jurisdiction; 10 members elected by major employers; two business park, office building, or shopping center owners; and two public interest group representatives. A technical advisory committee consisting of staff from the participating jurisdictions will support the regional task force in survey design, data collection, and analysis.

Jurisdiction Management

Local jurisdictions typically hire a program manager to implement and oversee the traffic mitigation program. The manager may be supported by staff depending on responsibilities, scope of the ordinance, and financial resources of the jurisdiction. Many jurisdictions, in fact, have one-person operations (e.g., Contra Costa County, Oxnard, El Segundo, and Berkeley). San Buenaventura and Alexandria have only one part-time position responsible for the TDM program. SCAQMD, which will be responsible for annually reviewing 8,000 TDM plans when the program is fully implemented in January 1990, has a staff of approximately 15.

Ordinances to be adopted in the Golden Triangle area will be managed by a central implementation agency. However, participating cities may elect to provide employee outreach services.

The regional program in Pima County is being implemented by the Pima Association of Governments with a staff of five. As noted previously, a regional task force has been formed to oversee implementation of the ordinances and a technical advisory committee consisting of technical staff from each of the participating jurisdictions will support the regional task force.

Responsibilities of the local jurisdiction management staff may include

- Developing employee outreach programs;

- Providing technical assistance to employers and developers;
- Training employee coordinators;
- Producing marketing materials;
- Developing guidelines, procedures, and forms for submittal of annual surveys and TDM reports;
- Monitoring and reporting on program performance;
- Reviewing and approving TDM programs;
- Reporting to an advisory committee, task force, city counsel, or county board of supervisors;
- Monitoring compliance and initiating or recommending enforcement action; and
- Recommending changes to ordinance provisions.

Effective implementation of an ordinance requires that the jurisdiction provide services to assist employers in complying. Pleasanton has a full-time TSM coordinator who provides support and assistance to employers and tracks compliance. This person serves as staff to the employer task force that has a de facto management role for the ordinance. Pleasanton also provides data processing services for the employee surveys that each firm must submit. Providing these and other services is vital to achieving compliance with a new ordinance. North Brunswick's experience supports this view. The town had no plans or resources to support implementation of the ordinance when it was adopted, but quickly realized this need.

Dedication of a staff person to this function may be significantly beyond the financial and administrative abilities of small jurisdictions. Moreover, the technical staff skills required are not likely to be available to many small communities. This limitation suggests that a multijurisdictional approach, at least for support services, will be needed for small communities.

Employers/Developers

Most ordinances require employers and developers of a certain size to designate a program coordinator for implementing the ordinance requirements, with the following responsibilities:

- Disseminating information to employees on commuting alternatives and alternate work hour programs,
- Coordinating data collection activities for annual surveys,
- Developing and submitting TDM programs,
- Implementing approved TDM programs,
- Serving as liaison to city staff, and
- Participating in public-private task forces.

In order to meet requirements for industrial or office complexes, a complex coordinator may be designated with the responsibility for these activities for employers within the complex.

Ordinances adopted in Montgomery County and El Segundo do not require appointment of a transportation coordinator; however, most large employers have appointed a transportation coordinator to implement the TDM programs.

The Contra Costa Center Association, a nonprofit organization composed of individual developers within the Pleasant Hill BART station area, was formed in 1985 to implement and manage shared commitments, which include the TDM program. All but one property owner has voluntarily joined this association.

Other Management Groups

Maricopa County will contract with the Regional Public Transportation Authority (RPTA) for advertising, public information work, and training and technical assistance to employers. Before the state legislation mandating a TDM program, the RPTA sponsored several voluntary TDM programs.

Funding

Most ordinances are supported by the jurisdictions' general funds. Fees and grants are also used to support traffic mitigation ordinances.

SCAQMD established plan submission and revision fees designed to cover program costs ($125 for initial plan and $50 for annual update). Sacramento's draft employer ordinance provides for fees to be assessed for the issuance and renewal of transportation management certificates. These fees will be used to defray the costs of administration, monitoring, and enforcement. The city's draft developer ordinance establishes a filing fee for the transportation management permit required of all new developments. The El Segundo ordinance provides for the establishment of filing fees by council resolution.

Maricopa County received a grant from the Air Quality Fund of Arizona's Department of Environmental Quality that will support the county-wide TDM program from October 1988 through June 1990. Beyond 1990, the probable funding source will be user fees, which are planned for 1990.

The TDM program in Pima County was initially totally locally funded, but the county anticipates receiving state funding in the future because the 1988 Air Quality bill passed, mandating TDM programs in counties of a certain size.

TDM programs in Seattle are funded through the city's general fund and FHWA's Federal Aid to Urban Systems.

Several jurisdictions also rely on parking fees to finance their TDM programs. Parking fees from the county lots in Silver Spring are used in part to fund the Montgomery County TDM program. Concord's TDM program is funded from a fund consisting of interest accrued on in-lieu parking funds and the net income derived from city-operated parking facilities and parking meters.

Lack of secure funding has been noted as a problem in TDM ordinance implementation. Many small jurisdictions do not have sufficient financial resources to hire adequate TDM staff and provide services, such as technical assistance and training, which are instrumental in TDM program success.

Enforcement

Ordinance compliance is generally determined by meeting program requirements rather than achieving specific goals. Jurisdictions typically identify as ordinance violations failure to conduct the survey; to provide ridesharing and transit information to employees; or to develop, submit, and implement an approved travel reduction plan. Violations are subject to increasing fines for each day of violation. SCAQMD and Contra Costa County ordinances include a fine and jail term for violation of their requirements.

Failure to achieve a specified goal is generally not considered a violation provided the employer or developer has made a good faith effort. Businesses or developers that fail to achieve a specified goal may be required to amend their plans and implement additional measures. Pleasanton may require employers who have failed to achieve the targeted reduction in vehicle trips to submit a revised program or the TSM task force may require the employer to implement specific measures. Failure to revise the plan or implement additional measures would be a violation subject to a civil penalty of $250 per day.

San Buenaventura's ordinance requires property owners failing to achieve the goal to provide stronger alternative mode incentives by levying in-lieu fees if the target participation rate is not met after a 6-month grace period. Bellevue's CBD ordinance requires financial contributions from property owners who fail to achieve the standard. In Concord, if a project sponsor fails to implement the TDM plan, the city may assume responsibility for implementing the plan directly with the costs borne by the sponsor.

Enforcement of ordinances placing conditions on developers typically involves denial of the building or occupancy permit for developers who fail to develop or implement an appropriate travel reduction plan in accordance with ordinance requirements. In Pasadena, the city zoning administrator can revoke the use permit for noncompliance with ordinance requirements. Similar provisions are contained in the ordinances adopted by the cities of Alexandria and Seattle.

Bellevue's ordinance includes a requirement for property owners to annually provide an assurance bond as a guarantee that required financial incentives will be provided. Forfeiture of the bond would occur for noncompliance.

Although most ordinances include specification of what constitutes a violation and the jurisdictions' recourse, to date there have been no reported cases of fines actually being levied for failure to comply with an ordinance.

Results

Most traffic mitigation ordinances are still in the development, adoption, or early implementation stages and are too new to draw conclusions regarding effectiveness.

Pleasanton, which adopted its landmark employer-based ordinance in 1984, has collected 4 years of performance data. The city experienced an overall reduction in peak-hour vehicle travel of 43 percent in 1988 (the target for employers in the 4th year is 45 percent). This figure may be low because it assumes that 20 percent of employees who did not respond to the survey drove alone. In 1988, 75 percent of the large employers reached their target. Of the 17 (25 percent) who did not reach their goal, 7 improved their performance over the prior year. Most of Pleasanton's trip reduction appears to result from changes in the timing of the trip rather than increases in nonsolo driving. Pleasanton has experienced an additional 10 percent shift to off-peak-hour commuting since the program's inception.

The San Rafael ordinance, adopted in 1983, requires developers, as a condition of permit approval, to maintain peak-period trip allowances. To date, 11 developments have been conditioned on trip allowances. All have met their goals except two that the city explains are unique land uses.

A key factor affecting the success of an ordinance is the area's stage of development. Newly developing areas may experience limited success in achieving participation goals. Lack of services and amenities has been noted as a problem in the Pleasant Hill BART station area, which is currently at approximately 10 percent of anticipated build-out. The ordinance is likely to be more successful when development is denser, providing more opportunity for ridesharing, and when sufficient services and amenities are provided.

Although it is too soon to know whether ordinances will be effective in achieving specific goals related to decreased traffic congestion, it is clear that the use of traffic mitigation ordinances is increasing. The public and private sectors are becoming more aware of the traffic problem and the positive effects that ridesharing, transit, alternative commute modes (other than SOV), and flexible working hours can have on congestion. Many developers, as a result of ordinances, are building infrastructure to accommodate SOV alternatives. This action is important for the long-term success of TDM programs. Many employers are educating their employees about potential solutions to the traffic congestion problem and are making alternative commute modes more readily available, more amenable, and less expensive. Some jurisdictions have noted that developers, recognizing the benefits of traffic reduction programs, are incorporating TDM measures in their marketing efforts to attract tenants.

Ordinance Development

Many ordinances have been developed through a joint effort of the jurisdiction and business community, typically represented by a task force. Other jurisdictions have involved developers and employers in the process through informal discussions. This public-private approach has been noted as a major contributor to the successful passage of several ordinances.

Obtaining support from developers is frequently easier than from employers. Jurisdictions have leverage over developers because many have the authority to establish conditions of development even without adoption of an ordinance. However, employers may not immediately understand how they will benefit from an ordinance or why they should support one. Jurisdictions considering TDM ordinances may need to spend considerable time educating employers to gain their support.

Development of the Pleasanton ordinance is noteworthy. A citizens' general plan review committee noted in 1984 that the county's transportation engineers assumed significant use of commute alternatives and flexible work hours in their studies. The committee reviewed the concept and recommended that a trip reduction ordinance be developed. City staff and employer and developer representatives subsequently developed a draft ordinance. From the beginning, developers supported the ordinance; however, employers were slow to accept and support the concept. A number of meetings were held to explain the ordinance's requirements and city staff talked to many employers individually. In response to employer concerns, the city agreed to hire a full-time program coordinator to assist employers in complying. The city agreed to assign enforcement responsibility to a TSM task force composed, in part, of representatives of the business community. After 6 months of cooperative effort, the ordinance was adopted on October 2, 1984, with no opposition.

SCAQMD passed Regulation XV on December 11, 1987, following a cooperative 7-month effort between SCAQMD and a 12-member trip reduction advisory committee composed of SCAQMD board members, Los Angeles Chamber of Commerce, Automobile Club of Southern California, Los Angeles Central City Association, University of California at Los Angeles' urban planning department, Atlantic Richfield Co., Disneyland, and the Irvine Co.

Similarly, the North Brunswick ordinance, adopted October 5, 1987, is the result of a 7-month study conducted by a task force comprising representatives of the local and county government, employers, and developers.

Regional Approach

The ordinance concept may be most viable as a regional strategy because it reduces fear of shifting the employment and tax base from one municipality to another. Several jurisdictions and government associations have taken the regional approach to traffic mitigation. Pima County and four cities within the county entered into an interjurisdictional agreement on April 18, 1988, to adopt consistent TDM ordinances. Each jurisdiction subsequently adopted an ordinance (with comparable provisions). The effort was spearheaded by the Pima Association of Governments, which continues to manage the travel reduction program in conjunction with a regional task force.

SCAQMD, a quasi-governmental agency with authority over four counties, adopted Regulation XV, which affects the entire region, in mid-1988. In the Seattle area, METRO and Puget Sound Council of Governments developed a model TDM ordinance in 1986 and are advocating that all jurisdictions in King County adopt similar ordinances to achieve regional consistency. To date, only Bellevue and Seattle have adopted TDM ordinances. Bellevue modeled its ordinance after the one developed by METRO, whereas Seattle took a different approach. The city of Kent drafted an ordinance, patterned after Bellevue's non-CBD ordinance. Other cities within King County are applying TDM conditions to developments through State Environmental Policy Act (SEPA) authority.

The Golden Triangle Task Force, a regional transportation planning effort, is drafting a model ordinance for use by the cities of Milpitas, Mountain View, Palo Alto, San Jose, and Sunnyvale, and the county of Santa Clara.

State Role

In the state of Washington, SEPA authorizes local governments to require development applicants to implement measures to mitigate the development's adverse environmental impacts. Developer conditions must be related directly to transportation goals documented in comprehensive plans or other previously adopted policies. The SEPA process allows for case-by-case negotiation with developers in all local jurisdictions. SEPA authorizes, but does not dictate, specific policies for placing conditions on developers.

Seattle uses SEPA authority to augment its land use code for mitigation of traffic impacts for downtown development and to condition developments on a case-by-case basis outside of downtown. Some jurisdictions, such as the city of Redmond, have adopted administrative procedures to formalize the policies and procedures for placing conditions on developers authorized by SEPA. Administrative guidelines provide staff with policy backing and help ensure consistency in case-by-case negotiations with developers. Local administrative guidelines are relatively easy to implement because they are developed within a department and do not require council approval.

In 1987, the Arizona state legislature passed Air Quality Bill 2206 that mandated travel reduction ordinances for counties of a certain size, with the state providing funding for travel reduction programs. Maricopa County is currently developing its program, which will become effective December 31, 1988. Maricopa County is not adopting an ordinance per se, but is operating from the state statute. Jurisdictions within Pima County adopted travel reduction ordinances prior to passage of the Air Quality Bill.

TRAFFIC MITIGATION ORDINANCES: DIRECTIONS AND PROSPECTS

Traffic mitigation ordinances provide substantial promise as a widely applicable tool for managing congestion. Limited empirical evidence exists on the effectiveness of ordinances because of the limited time of their application.

The ordinance concept clearly can apply to cities facing new traffic congestion problems, but its application to older cities with long-standing problems is less clear. In Los Angeles, where traffic congestion has long been severe, the ordinance quickly expanded from a small initiative by the mayor to a massive area-wide program involving all major employers in four counties. Los Angeles has a relatively low transit modal split and its problems are somewhat unique. Flexibility of the ordinance concept increases its applicability to a wide range of development environments.

State Role

The role of state governments in supporting the ordinance strategy is emerging. Sponsoring TMAs is a supportive action that can lead to ordinances. For example, the Connecticut Department of Transportation is developing a program to provide support services to municipalities desiring to develop a TMA. This program will prevent localities from duplicating their efforts.

States can develop model ordinances so that each jurisdiction does not have to start from scratch. This ordinance would also increase consistency between local jurisdictions. In some cases, enabling legislation may be necessary.

States can also provide other support services for jurisdictions or affected employers by sponsoring conferences, workshops, and training courses, and providing centralized data processing services. States can enact tax credit legislation that rewards employers for expenditures made supporting traffic reduction. Politically, tax credits can be vital in offsetting opposition to the ordinance strategy by business community members who may not accept the private sector's role in traffic reduction efforts. California, Connecticut, Massachusetts, and New York have initiated employer tax credit efforts.

Supply Side and Financing

The TDM ordinance concept focuses primarily on the demand side of the urban transportation problem. Most actions prompted by ordinances are designed to influence demand for existing services, such as subsidizing transit and providing preferential parking for car and van pools. However, some actions enhance the supply of services, such as the provision of employer-supported shuttles.

Although some shifting of demand to existing services can be achieved, peak-hour transit services may already be at capacity, or acceptable services may be unavailable. Enhancing the demand for transit and ridesharing, the primary effect of ordinances (other than shifting demand to less congested times), is not adequate to address these problems. Improved services will also be required.

Fee-in-lieu-of ordinances can provide financial resources to meet service expansion needs. A new development ordinance being developed in Stamford, Connecticut, allows less-than-code stipulated amounts of parking if compensating payments are made to the city, which have been used to support new shuttle bus service. A proposed employer ordinance could allow firms to opt out of meeting the required traffic reduction level by choosing to pay an annual fee-in-lieu-of for each peak-hour trip by which the standard is exceeded.

Linkage to Other Planning and Land Use Issues

Clear but unexploited linkages exist between the ordinance concept and other transportation planning and land use issues. For example, it makes little sense to adopt a traffic reduction ordinance if the development code still promotes or requires provision of excessive amounts of parking spaces. Floor area ratios and site design requirements should be considered before trip reduction measures. Development requirements for provision of on-site services, such as employee cafeterias, could also be adopted. TDM ordinances are not a panacea. Their institution should stimulate consideration of other available, supportive actions to reduce traffic generation even before it materializes.

Innovative Program Development

Simpler programs for employer use in reducing traffic are needed for successful implementation. For example, SCAQMD's Regulation XV may actually place a significant burden on the Southern California Rapid Transit District for provision of bus passes for employer-discounted sales. Substantial administrative expenses for the employer will be incurred for selling monthly bus passes to employees, collecting the nondiscounted share, and interfacing with transit operators. This burden increases dramatically when multiple private bus operators are involved rather than a single public agency.

The recent success in New York with the TransitChek multioperator transit voucher, which doesn't change monthly and is simply given rather than sold to employees, would thus be well applied in Los Angeles or other communities with employer ordinances.

Provision of transit information services needs to be streamlined before widespread employer support can honestly be expected or required.

RECOMMENDATIONS

Given the demonstrated acceptability and apparent effectiveness of the ordinances, the strategy appears appropriate for increased emphasis and promotion. This action is already happening, as evidenced by new state and regional level activities in California, Arizona, Connecticut, and elsewhere. In addition, UMTA's recent suburban mobility seminars have also spread the word about traffic mitigation ordinances.

An ongoing cataloging of developments in the field should be maintained. Evaluating the effectiveness of the ordinances that now exist is also necessary. Pleasanton is the only ordinance on which much can be concluded at this point, from an actual impact point-of-view. A thorough report on the Pleasanton experience could be beneficial to other areas considering the traffic mitigation ordinance strategy.

It may be possible to evaluate other ordinances from a process perspective. The likely importance of SCAQMD's Regulation XV suggests that substantial efforts be commissioned quickly to document the process through which it emerged and to ensure that data are available to assist the formal tracking of its impacts.

Networking among the localities interested in pursuing TSM ordinance ideas should also be beneficial. This approach could help localities take advantage of past experience to anticipate requirements and avoid pitfalls, and would help maximize the number of successful programs implemented.

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Guidelines for Developing Local Demand Management or Trip Reduction Policies

T. H. HIGGINS

Local governments are increasingly turning to demand management or trip reduction strategies, policies, and programs to combat traffic congestion. Using various policy instruments, localities are encouraging employers and developers to implement transportation systems management (TSM) and parking management strategies. These strategies include encouraging use of transit, carpooling, bicycling, walking, and flextime. However, still other measures include reduced employer subsidies for employee parking, tightening of maximum parking requirements of zoning codes, reduced parking requirements in return for implementation of TSM strategies, and other measures to reduce solo driving. Recommendations are made to local government decision makers and planners on the adoption and implementation of demand management or trip reduction policy instruments, including: (a) recommendations about when demand management and trip reduction strategies and policies are appropriate to consider; (b) considerations in selecting trip reduction or demand management policy instruments, including ordinances, developer agreements, special permits, and parking code requirements; (c) suggestions on the design of particular policies, including application of requirements, specificity of requirements, uniformity and stringency of goals, and use of exemptions; and (d) guidance on program monitoring, enforcement, management, costs, and timelines for program development.

Demand management or trip reduction strategies are playing increasing roles in the attack on traffic congestion. The strategies generally fall into two important categories, transportation system management (TSM) and parking management (PM).

Generally, demand management approaches aim to reduce peak period automobile trips by encouraging the use of high-occupancy modes. TSM strategies include preferential parking for carpoolers; promotions for transit, carpooling, bicycling, walking, and flextime; designation of transportation coordinators at employment sites; and shuttle service to and from park-and-ride lots.

PM actions include raising existing rates or imposing new surcharges or differential rates at public facilities, imposing parking taxes at commercial facilities, reducing employer subsidies for employee parking, revising the supply of long-term parking through new maximum requirements in zoning codes, allowing reduced supplies of parking in return for in-lieu fees or implementation of TSM strategies, revising fines and enforcement, and other measures aimed at the provision and management of parking spaces for purposes of reducing solo driving.

Both the public and private sector play roles in the implementation of TSM and parking management strategies. Localities set regulations requiring private developers and employers to carry out strategies, and meet trip reduction objectives. Often, requirements also provide for an annual employee survey or other forms of monitoring. Sometimes, transportation management associations (TMAs) play a role in implementing the programs.

Numerous localities have fashioned and adopted policy instruments to encourage implementation of TSM and PM, including

- Ordinances,
- Developer agreements,
- Special permits, and
- Parking code provisions.

WHEN TO CONSIDER TSM AND PM POLICIES

Every community plagued with traffic congestion should not necessarily try to develop and adopt TSM and PM policies. Some important considerations include the degree of through traffic, size and nature of employers, management capability and program resources, importance of parking pricing, and the role of exogenous variables.

Degree of Through Traffic

If a large proportion of congested traffic is bound for developments and employers outside the locality, TSM and PM policies may have limited effect. TSM and PM policies aim at reducing automobile use to and from developments and employment sites within a community. Thus, if much local traffic is not generated by these sites, local ordinances, developer agreements, and other policies may not help. However, if several localities join together and form common policies to attack both local and regional traffic, then through traffic may be influenced by these policies. Localities should be aware that few local governments have successfully joined with other localities to adopt uniform local ordinances, joint-power arrangements, or regional programs. Several are trying, but

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the process of coming to agreement on common requirements as well as funding and program operations is long and laborious. For example, localities within the counties of Marin and San Mateo in California have debated for months the possible adoption of coordinated ordinances, without success. Maricopa and Pima counties in Arizona have adopted areawide ordinances applicable to employers across several cities, but only after the passage of special state legislation and much pressure from the U.S. Environmental Protection Agency to adopt trip reduction measures or face delays in federal highway funding.

Size and Nature of Employers

All else being equal, results of TSM and PM programs at multiemployer centers tend to be less successful than at single employers. One review of programs at multiemployer sites found the maximum drop in solo driving to be only 3 or 4 percent (*1*). Sites included in the study were El Segundo, California; Greenway Plaza in Houston, Texas; and Tysons Corner, Virginia. Another recent review of suburban TSM and PM programs suggests little success in ridematching at the Denver Technical Center in Colorado because of the preponderance of small firms in the center (*2*). The size and type of employer may also be important because TSM and PM programs tend to be more successful at larger companies with lesser proportions of professional staffs, though the evidence is not clear cut. Some studies suggest TSM and PM success stories tend to be with large employers and large pools of clerical and data processing personnel, as opposed to small employers with professional workers. Yet other literature contradicts these findings. For example, among nine leading companies in the Santa Clara County Manufacturing Group (SCCMG) in California, the proportion of employees in alternative modes averages only 21.5 percent with employment under 5,000 persons. Only four firms have sustained rates of 25 percent or higher and they tend to be larger firms (*3*). Nationally, the picture is similar with TSM and PM programs at larger companies showing the greatest success. For example, one survey shows alternative mode shares between 30 and 40 percent for companies with over 1,000 employees, but with companies under 1,000 the share is generally around 20 percent. Nevertheless, there are exceptions, such as Cenex in St. Paul, Minnesota, with only 730 employees and 47 percent in alternative modes and Minnesota Mutual Life Insurance also in St. Paul with just 1,000 employees and 39 percent using alternative modes (*4*). Furthermore, early studies of company vanpool programs found "no relationship between company size and . . . (success of) . . . ridesharing programs" (*5*). The overall lesson is that localities with a preponderance of small companies or largely professional workers should adopt TSM and PM policies with caution because the policies probably will not be as effective as those in communities with larger employers and more clerical or data processing workers.

Management Capability, Vigilance, and Program Resources

Localities considering adoption of TSM and PM policies must be prepared to support policy implementation. In the long run, management and resources may be more important than the type or stringency of the policy instruments adopted. One recent study of 40 suburban TSM and PM programs concludes: "More important than the policy instrument or its terms and provisions may be the resources devoted to the programs, vigilance of monitoring and general level of visibility and commitment to the TSM and PM effort" (*6*). Another review of TSM and PM programs in the San Francisco bay area supported by ordinances concludes, "The effectiveness of . . . programs hinges on the management commitment that is made at start-up, and its (sic) sustainability depends on the durability of that commitment" (*7*).

Importance of Parking Pricing

Limited or expensive parking combined with strong rideshare and transit incentives can reduce solo driving considerably. In Bellevue, Washington, a suburb of Seattle, Pacific Northwest Bell (PNB) has reduced solo driving to only 19 percent of the work force through a combination of scarce, expensive parking ($3.00 per day at the time of the study), reduced parking rates for carpoolers, and intensive ridesharing assistance (*8*). Likewise, Commuter Computer outside the Los Angeles, California, central business district (CBD) decreased the share of solo driving from 42 to 8 percent by eliminating free parking (*9*). Parking pricing also is creating effective demand management programs at several other employers including Twentieth Century Corporation in Los Angeles; the Nuclear Regulatory Commission in Maryland; and Bellevue City Hall in Bellevue according to a recent national survey (*6*).

Role of Exogenous Variables

Exogenous variables are important to program success. These variables include proximity of companies to transit service and preferential treatments for ridesharing and transit on streets and highways near employment sites. Parking availability and price surrounding the site also are important. For example, in Walnut Creek, California, one study shows the proportion of transit users varies in relation to proximity to transit, with twice as many Bay Area Rapid Transit (BART) users at offices close to the rail station compared to more distant offices (*10*). Preferential treatments also help. High-occupancy-vehicle (HOV) bypasses to ramp metering on Los Angeles (including some areas outside the CBD) freeways boosted weekly ramp usage by carpools from 125 to over 275. Transit use in the Minneapolis, Minnesota, I-35W corridor increased 6 percent after meter bypass systems were introduced (*11*). Finally, as the example of the PNB building in Bellevue, Washington, demonstrates, the supply and regulation of parking around work sites also are important. Limited parking and high prices are encouraging considerable ride-sharing at PNB, but some PNB employees are spilling over into uncontrolled parking on minor arterials near the building. Bellevue is expanding on-street controls in areas of major developments to guard against just such spillover (*8*).

Summary of Limits of TSM and PM Program Variables

Overall, the prospects of TSM and PM programs and the rationale for supporting policies depend on several variables.

In the best case, these variables align to favor reductions in solo driving and increases in ridesharing and transit use. In this case, localities should consider application of TSM and PM policy instruments. In the worst case, just the opposite pertains. The best and worst cases for TSM and PM programs are as follows:

Variable	*Best Case*	*Worst Case*
Traffic	Large proportion generated within or bound for the locality	Large proportion of through traffic
Employers	Large companies, numerous clerical or data processing staff	Small companies, high proportion of professional staff
Program	Strong management, high visibility, good commitment of resources, strong monitoring and vigilance	Little or diffuse management, low visibility, few resources, lack of monitoring and vigilance
Parking	Tight supply, moderate-to-high prices, low level of parking cost subsidy, little on- or off-street parking nearby	Ample supply, low or no prices, parking subsidies from employer, available nearby parking
Exogenous	Nearby transit service, HOV facilities, metered bypasses, little on- or off-street parking nearby	Little nearby transit service, HOV or bypass facilities, available on- or off-street parking

POLICY INSTRUMENT EXAMPLES

TSM Ordinances with Broad Applicability

Many localities have developed ordinances requiring employers and developers to implement TSM and PM programs. In many cases, such ordinances apply to new and existing employment centers and in a few cases include residential development. Some jurisdictions also are attempting to form coordinated programs across jurisdictions, including the following examples:

- Concord, California, requires TSM and PM programs of all new and existing nonresidential development within the city, provided development generates at least 100 peak-hour employee trips. Residential complexes with over 100 dwelling units also are covered.
- In San Mateo County, California, five cities are collaborating through a joint powers agreement to develop and adopt uniform ordinances and an intercity transportation management authority. The draft ordinance would require employers to implement TSM and PM programs resulting in 25 percent of employees using alternatives to solo driving.
- Pleasanton, California, applies its ordinance to the entire city and gears it to employers of 10 or more employees with escalating requirements for larger employers. Multitenant buildings and business complexes are specifically included.

Developer Agreements

Some communities use instruments appropriate to requiring TSM and PM programs as a condition of development. Developer agreements backed by covenants written into property deeds bind owners and successors in interest.

- In the case of Montgomery County, Maryland, the sample development agreement (Costain Agreement) is for 10 years, and on expiration the TSM and PM program is to be incorporated into a county ridesharing program. Materials, software, and supplies all transfer to the locality.
- In the case of Bellevue, Washington, the Bellevue Place agreement requires a broad set of TSM and PM actions, including limits on the parking supply, automatic vehicle counters for traffic monitoring and reporting, target maximum p.m. peak-hour vehicle trips, required membership in the local TMA, set-aside carpool spaces, required parking charges for employees, increasing levels of required actions depending on project performance, and an assurance bond guaranteeing the program terms are in force beginning with occupancy and continuing until no longer required by the city.

Special Permits

Various public entities require special use permits for projects, including binding commitments from project sponsors for TSM and PM actions, and other actions aimed at mitigating traffic and air quality problems:

- The Minnesota Pollution Control Agency requires an indirect source permit for parking facilities; retail, commercial and industrial facilities; office buildings; large housing developments; airports; racetracks; and other developments. TSM and PM requirements have included transportation coordinators, transit promotions, carpool incentives, and other actions. Some of the projects regulated are within "fast developing suburbs" (*12*).
- Alexandria, Virginia, requires a special use permit for new developments over a certain size including a transportation management plan for ridesharing, transit incentives, bicycle measures, flextime aimed at up to 30 percent use of alternatives to solo driving, or certain percent reductions in peak-hour travel by solo drivers.

Parking Code Requirements

Some localities have implemented parking code requirements aimed at encouraging TSM and PM. One approach is to establish a maximum rather than minimum parking requirement for certain developments. Another approach is to offer relaxations in minimum parking requirements in return for TSM and PM actions. Under relaxations, localities appear to reduce requirements by no more than 20 or 30 percent. Some require land set asides to be converted to parking if supply doesn't meet demand.

- Bellevue, Washington, sets both a maximum and a minimum parking space requirement for office use in the downtown area. Specific requirements are negotiated by site and set in developer agreements. The Bellevue Place agreement provides one specific example. An early precedent agreement for ENI Co. also limits parking supply, and requires priced parking.

- Fairfax County, Virginia, allows reduced parking in proximity to a mass transit station on the basis of projected reductions of automobile trips caused by proximity to transit.
- Sacramento County, California, allows reductions for TSM and PM measures, with showers and bike lockers rendering a 2 percent reduction, and one space reduction for every marked carpool space (Ordinance 83–59).
- Montgomery County, Maryland, requires land set aside be sufficient to provide "parking spaces equal in number to the reduction granted" (Ordinance 10–32).
- Palo Alto, California, has a similar contingency provision.

POLICY INSTRUMENT DESIGN ISSUES

Applicability of the Policy

A key issue is defining applicability. To what entities will the policy apply? Will all new and existing developments be included? What areas will be included, what uses, what size thresholds? For developer agreements, policies apply to new and usually large office projects. Parking codes usually apply to core areas. Applicability requires considerable attention in the design of ordinances. Several ordinances reviewed apply to employers, and scale requirements by size.

- Pleasanton, California, stages requirements on employers by size as well as whether or not they are located in complexes. Employee requires careful definition, as well as what constitutes a complex. The city's intention is to include complexes or employment centers with several small employers, as opposed to isolated small employers.
- Pleasant Hill, Contra Costa County, and Concord, California, include residential uses in their ordinances, in contrast to many other localities excluding these uses.

Specificity of Requirements

How much should the locality specify in the way of strategies and programs, and how much should be left to the regulated entity to plan and carry out? Localities must decide how certain they are specific TSM and PM strategies will work in the developments and areas subject to regulation. Are designated carpools worth requiring in a particular developer agreement or area-wide ordinance? What programmatic requirements should be set, such as designated coordinators or resources devoted to the program? Experience to date suggests the most common requirements in policy instruments is for distribution of information on car and vanpooling, transit, bicycling, and other alternatives to solo driving. Designation of an on-site coordinator responsible for carrying out the program is another commonly prescribed strategy. A few localities do require more aggressive strategies, including priced parking, designated carpool stalls, rideshare matching services, sale of discount transit passes, even implementation of shuttles.

In Bellevue, Washington, requirements in some developer agreements specify the number of car and vanpool spaces, membership in a local transportation association, on-site transportation coordinator, as well as added actions (sale of transit passes and discount parking for carpools) if certain mode share or traffic level targets are not achieved.

Fewer and more flexible requirements generally are specified in ordinances. For example, Contra Costa County, California, allows owners and employers to choose any combination of strategies and they are free to design their own information program. However, the ordinance does require an annual employee survey, designated coordinator, reference to program requirements in lease agreements, and specific annual report to the county.

Recognizing the importance of charge parking to the outcomes of TSM and PM programs, some localities impose requirements for pay parking through developer agreements. Developers will be concerned with the marketability of projects where rates are imposed versus others where they are not. Nevertheless, Fairfax County, Virginia, has required applicants to institute a parking policy with incentives for ridesharing. In the agreement with Bellevue Place, Bellevue, Washington, specified that parking charges be no less than certain transit fares in the area. Bellevue also required fee parking in its agreement with ENI Co.

Types, Uniformity, and Stringency of Goals

Localities must decide what goals, if any, to set in their requirements. Localities can select from goals in terms of mode share or occupancy (e.g., percent of employees traveling alone or by alternative transportation); traffic performance (vehicle trips at certain times and places, levels of service at intersections); proportion of commuting in peak periods; or combinations of these and other approaches. Goals must be set that are reasonable to attain given experience with TSM and PM. Goals also might vary by areas or proximity to transit. Perhaps more important, localities must decide whether the goals are good faith targets that employers and developers are expected to try to meet or are binding performance standards that, if not achieved, trigger certain consequences. Before opting for performance standards, localities must consider the possibility that an employer may make every effort to implement the TSM and PM program but still not achieve the standard. In some cases, the standard may be unreasonable, or gasoline prices may fall, or the economy may boom, or imported car prices may fall. These and other variables outside the TSM and PM program may encourage automobile use. Generally, it seems localities apply the most stringent goals to development agreements and the less stringent goals to broad-area ordinances. Examples of goals and stringency follow.

- Pleasanton, California, defines the goal in its ordinance as a 45 percent reduction of vehicle trips during a 1-hr peak period compared to the case where all employees commute by single-occupancy vehicle. If the goals (staged over time) are not met, the city may then require the employer to carry out a specific program.
- Contra Costa County, California, uses a binding primary and secondary goal. The primary goal is no more than 65 to 75 percent of employees commuting in single-occupant vehicles, depending on the area. But if the project sponsors can demonstrate the goal is unfeasible, the secondary goal applies, which is 55 to 65 percent solo drivers in the a.m. and p.m. peaks. If the goal is not reached, the county is entitled to mandate implementation of a revised program.

• Larkspur, California, has set a demanding goal in its Ordinance 737. Approved projects receiving a circulation permit—with or without TSM and PM actions—must not increase average daily traffic on any roadway segment or intersection of the city's principal circulation system by more than 1 percent or more than 100 vehicles, whichever is less.

• Walnut Creek, California, varies its goals not only by uses (retail, nonretail) but by area, with sites closest to a BART rapid rail station slated for the highest goal, i.e., "no more than 60 percent of all employee commute trips in single-occupant vehicles." Elsewhere the goal varies up to no more than 75 percent who drive alone.

• In Montgomery County, Maryland, the Costain agreement's goal is a reduction of 180 vehicle trips during the peak period, in the peak direction. If the goal is not reached, the county can draw on a letter of credit posted by the project sponsor, or transfer the program to the county ridesharing agency.

Nature and Timing of Plan Requirements

Often TSM and PM requirements specify development of a plan that spells out what TSM and PM strategies the developer and employer will carry out and how. The plan may have a one-time requirement, often before development of certain projects, or it may have a continuous (usually annual) requirement for reporting on the TSM and PM program and making modifications. The advantage of plan requirements is that they allow employers and developers to develop and propose strategies and programs tailored to particular sites, employee populations, and parking or traffic conditions. Of course, plans require time and expertise to review and negotiate. Small localities may not have the resources or experience to conduct reviews. In addition, the questions of which applicants should face the requirements and what plan contents will be required need to be answered. Another issue is how the first plan can be prepared for a proposed development without knowing exactly the tenant mix until occupancy begins. For example,

• Sacramento County, California, requires applicants of major developments to prepare a trip reduction plan on rezones, use permits, special permits, development agreements, or variances. The ordinance also specifies the contents of the plan (Ordinance 83–59, Section 330–147).

• Contra Costa County, California, requires a conceptual plan at the time of application and a final plan recorded as a covenant on the project in all cases in which reductions in parking requirements are allowed for the promise of TSM and PM actions.

• Concord, California, requires a final plan after occupancy to ensure the plan reflects actual employees and tenants locating in the building. A preliminary plan is submitted at the time of application. The contents of the plan are spelled out in the ordinance.

• South Coast Air Quality Management District (SCAQMD) in Los Angeles, California, requires a plan to achieve certain average vehicle ridership targets and also requires annual updates to verify TSM and PM strategies in place and to propose changes in strategies.

When and How to Enforce

All recently developed TSM and PM policy instruments contain provisions for monitoring and enforcement. Most commonly, localities require reporting from developers and employers and reserve the right to impose fines or other sanctions for failure to carry out such required actions as submittal of annual reports, implementation of the TSM and PM program, or designation of a transportation coordinator. Toward the end of ensuring against lagging programs, some localities require performance contracts and bonds. A disadvantage of this approach is that it binds only signatories. Purchasers of the property are not contractually bound. However, covenants running with the land may accompany performance contracts, thereby ensuring enforcement against new title holders. Few jurisdictions impose fines or noncompliance sanctions on ineffective programs, provided all required strategies and program operations are carried out. Nevertheless, some localities reserve the right to take some action in the case of ineffective programs. Actions include the locality assuming program operations or specifying how the program should operate or delaying further stages of building development until a program is effective. Examples include

• Bellevue, Washington, and Montgomery County, Maryland, sometimes use a performance bond in support of enforcement. Montgomery County requires posting of initial and subsequent replacement letters of credit. The county may draw on the letter if the developer is not operating the program or achieving goals.

• In Pleasanton, California, annual reports from employers are required. Failure to reach goals triggers a task force review, which can impose additional strategies. Failure to implement the program can result in a fee of $250 per day until compliance is complete.

• In Concord, California, the city again requires annual reports on program actions and proportions of employees using transit, carpools, and solo driving. The city reserves the right to require a traffic impact report and added strategies or capital improvements to roads and signals in cases in which the goals are not met.

• Fairfax County, Virginia, in its applicant agreement reserves the right not to issue building permits for development over a certain square footage if total peak-hour trips exceed specific levels. The county provides for appeals to the board of supervisors, independent traffic engineering analysis, and arbitration on the question of the traffic generation and impacts of the subject property (unspecified agreement, May 20, 1982).

Types of Exceptions, if Any

Localities must consider if and how to exempt employers or developers from requirements. Exemptions can make allowances for unusual situations and cases. For example, an ordinance may go into effect in an area where employers already operate effective TSM and PM programs and are subject to agreements or ordinances. Here, exemptions may be warranted. Exemptions also help make a policy acceptable where otherwise it would not be. On the other hand, exemptions may invite abuse or create continuous demands for more exemptions. Localities also must craft exemption language to include only the desired cases, but exclude others.

- Contra Costa County, California, exempts employers from TSM and PM requirements, provided the employer already meets the ordinance objectives in terms of the proportion of employees commuting alone and by alternative means of transportation (Ordinance 87–95).
- SCAQMD exempts employers already subject to local ordinances, provided the local ordinances are at least as stringent and effective as the district's Regulation XV.
- Maricopa County, Arizona, exempts employers opening for business, relocating, or otherwise adding employees, but employers do become subject to the ordinance within 60 days before the annual due date of the employee survey and plan. The county also exempts from ordinance requirements employers who can demonstrate effective programs already in place at least for 12 months before the date when the employer is subject to the ordinance.

Types and Purposes of Fees and Financing

Localities sometimes build into their policy provisions for fee collection in support of administering the policies or in support of TSM and PM program operations. Localities must decide if and how to set fees or financing provisions in policy instruments. Many localities have not built fees or financing mechanisms into policy instruments. Although not including finance and fee issues in policy instruments may ease passage or negotiation of the instrument, there remains the question of how plan review, monitoring, and implementation in which fees are not specified will be supported. Generally, it appears localities are more likely to impose fees in developer agreements and special permits than in broad-coverage ordinances, probably because it is politically more palatable to do so. For example,

- In Bellevue, Washington, the developer agreement for Bellevue Place specifies dues on the basis of employee vehicle trips generated by the project. Revenues go toward supporting the local TMA, a public-private organization responsible for many mitigation efforts downtown.
- In Montgomery County, Maryland, fees are specified in support of the county ridesharing agency, Share-A-Ride. The basis of fees is per $100 of real property value (Bill 19 84). Additionally, the county reserves the right to draw on a letter of credit posted as security in developer agreements and to use proceeds to support the county's rideshare program (Share-A-Ride). For Silver Spring, Maryland, the county may transfer revenue from parking fees in order to support the TSM and PM program (Bill 24–87).

IMPLEMENTATION EXPERIENCE

TSM and PM policies do not operate in a vacuum. Implementation of these policies brings management and organizational implications. National experience suggests important issues and lessons for jurisdictions.

Management and Organization

In the management and organization of TSM and PM programs, locality staffs, building managers and employers, and possibly a local committee are involved.

In most localities, planning departments are responsible for reviewing and approving any TSM and PM plans and parking relaxations. In many jurisdictions, a transportation coordinator designated within the planning department reviews plans submitted with applications, as well as required annual plans and employee surveys. In addition, the coordinator would

- Collect and analyze the annual employee surveys;
- Prepare the annual report to city or county council;
- Develop the central transit pass sales outlet;
- Organize promotional events across developments;
- Prepare, collect, and develop promotional materials;
- Develop and carry out promotional seminars and meetings;
- Conduct overall monitoring;
- Lobby for transit, bicycle, or other applicable services;
- Contract and direct TSM and PM consultant services; and
- Conduct training of on-site employer coordinators.

In many localities, the coordinator acts as the staff to a special committee responsible for overall review of TSM and PM programs and policies and reporting to decision makers. For example, the roles of the Pleasanton, California, Task Force are delineated in the ordinance as establishing program and plan guidelines, monitoring, deciding if mandatory provisions are necessary, and hearing disputes and appeals. A TSM or PM committee would

- Adopt TSM and PM policy and intent statement;
- Review the annual plan, suggesting directions and policies;
- Represent developers and employers before locality or transit agencies;
- Evaluate proposals for new TSM and parking strategies;
- Help suggest and design all promotional materials;
- Facilitate monitoring of program effectiveness;
- Assist in special events and company seminars;
- Review literature and visit model programs;
- Act as an information exchange on all strategies;
- Help provide access to employers for survey and promotions;
- Consider supportive tenant lease language;
- Review and respond to transit service proposals; and
- Arrange space for seminars, promotions, and training sessions.

City councils or county supervisors, in most communities, function as the point of last appeal on issues of noncompliance or nonperformance.

Developers and employers are responsible for setting up programs at the site. Often, ordinances or developer agreements specify that an on-site coordinator will be designated to carry out the program. Developer responsibilities typically include

- Attending committee meetings and supporting committee decisions;

- Installing bicycle lockers, if warranted;
- Implementing carpool stalls and easy exits, if warranted;
- Authorizing and helping to set up lobby displays;
- Informing tenant companies of program;
- Adding supportive lease terms; and
- Setting up transit and van pool stops.

For employers, the coordinator would

- Urge management support for employee participation,
- Distribute and collect employee and manager surveys,
- Post and update bulletin boards,
- Insert company newsletter articles,
- Distribute transit passes and carpool matching information, and
- Ensure new employee orientation.

Another important and emerging organizational entity in TSM and PM policies is the TMA. It is a private, nonprofit corporation composed of developers, employers, and representatives of public jurisdictions working to alleviate transportation problems. In some localities, the TMA has responsibilities in the management of TSM and PM programs. For example, in Bellevue, Washington, the city has required a developer to support the local TMA through dues on the basis of vehicle trips generated by the Bellevue Place project.

Monitoring

TSM and PM policy instruments often specify surveys, regular reports, and sometimes a form of traffic monitoring. A common requirement is some form of annual report from employers subject to requirements. Usually, the report is based on employee surveys. Surveys are aimed at determining the proportion of employees solo driving, using transit, bicycling, walking, and ridesharing. The Pleasanton, California, city council receives an annual report and employee survey results. Fairfax County, Virginia, requires a traffic analysis at different phases of the subject development. In case of dispute over results of the traffic analysis, the county provides an arbitration board to resolve disputes. Bellevue, Washington, requires traffic counters embedded in exits of the project and specifies the exact month and weekdays of counts. At the same time, the project occupancy is assessed to determine compliance with required limits on outbound employee vehicle trips in the p.m. peak.

Program Costs

Costs of TSM and PM programs vary widely by the nature and size of the program. For employer-based programs, costs are borne primarily by developers and companies responsible for implementation. Of course, localities also bear costs, especially if they designate their own coordinators to participate in and enforce programs. Some examples from employer-based programs in the San Francisco, California, area demonstrate cost ranges. At the high end of the cost range, a few programs provide examples.

- At Varian in Palo Alto, with about 5,000 employees, the program costs $72,000 per year, or $14.40 per employee (R. Loomis, unpublished).
- At Lockheed in Sunnyvale, about 25 percent of the 25,000 employees use alternative modes. Their program costs $25 per employee per year.
- Probably one of the most extensive programs is the Bishop Ranch office complex in San Ramon serving 4,000 employees. This program includes a full-time coordinator, transportation store, computer matching, and two luxury coach shuttles for an annual cost of about $200,000 or about $50 per employee.
- Chevron in San Ramon serves 2,000 employees and spends $110,000 on a full-time coordinator, BART shuttle, flextime, demonstration vanpools, and marketing materials. The annual cost of the program per employee is $55.

Other programs serving fewer employees, or not so comprehensive in scope, cost less and include the following:

- AT&T in Pleasanton serves 2,000 employees and spends $27,000 with a nearly full-time coordinator, monthly cash awards, carpool meetings, flextime promotion, transportation hotline, and information center. Unit cost is $13.50 per employee.
- Rolm Corporation in Santa Clara serves 4,000 employees and expends $40,000 for a cost of $10 per employee. The program entails a full-time coordinator, transit pass sales, bicycle lockers, semiannual drawings and transportation fair, and in-house matching.
- A 1985 study of employer programs in Santa Clara County reveals an average annual budget per employee of $6.15 (*13*).

Overall, it appears basic costs of moderate-sized TSM and PM programs range from $30,000 to $50,000 per year, excluding such costs as office space, computers and software, furniture, training, insurance, and survey analysis. At larger employment centers with as many as 5,000 employees, costs may reach $100,000 to $150,000. A shuttle operation might bring costs closer to $225,000 or even more. For small employers (e.g., less than 500 employees), costs for a modest program might range from $10,000 up to $20,000 and for extensive programs, between $30,000 and $60,000. For large employers (e.g., greater than 1,000 employees) a modest program could cost between $30,000 and $60,000, whereas for an extensive program the costs range from $100,000 to $250,000.

Program Financing

Both public and private financing arrangements are used to support employer-based programs. In some cases, programs are supported by private financing without enforceable commitments. These voluntary private commitments might include in-kind contributions of personnel, office space, computer facilities, and the like. Or, some employer dues and fees might be contributed, again without a legally binding commitment. In other cases, programs are financed by legally binding public mechanisms put in place by local government. These mechanisms include impact fees, business license taxes, benefit assessment districts, and others. Examples of public mechanisms in some jurisdictions include the following:

- In the Los Angeles area, the Coastal Corridor and Westwood ordinances require trip fees. The fee per p.m. peak-hour trip in the Coastal Corridor is $2,010, whereas in Westwood it is $5,600 per trip.
- Concord, California, has established a fund consisting of interest on the in-lieu parking fund, net income derived from any city-operated parking facilities, and other dedicated sources. The fund supports the activities of the city coordinator.
- Berkeley, California, imposes a one-time fee of $2.00 per ft^2 or an annual fee of $.20 per ft^2 for 30 yr. Fees that enter the transportation services fund are used to support ridesharing, transit, and bicycling.

Where TMAs are formed, they might use private commitments to support the program. For example,

- The TMA in El Segundo, California, levies an assessment of $1.25 per employee.
- The North Bay TMA in Marin and Sonoma counties in California charges an annual fee of $25 per employee up to a maximum of $250 per employer.

Enforcement and Legality

Thus far, enforcement and legality have not been large issues in the implementation of TSM and PM programs. Many localities check compliance with mitigation regulations by requiring annual reports from employers on employee travel modes and program activities. Others require traffic reports. Few TSM and PM programs have operated long enough to provide examples of localities invoking sanctions for noncompliance. However, localities and employers have negotiated issues of compliance without resort to sanctions or court tests. For example,

- In 1986 and 1987, the coordinator in Pleasanton, California, found it necessary to pressure several employers to submit annual reports and surveys. Finally, the reports and surveys came in without resort to notification from the city attorney or the need for other procedures (G. Gilpin, unpublished).
- Likewise, Montgomery County, Maryland, has never called in letters of credit in cases in which employers were not achieving mode-share or trip-reduction standards. The county has reviewed such cases carefully and is satisfied best and good faith efforts are occurring (J. Clark, unpublished).
- Novato, California, in an agreement with Fireman's Fund Insurance Co. required the implementation of a flextime program to ease traffic burdens on nearby streets. After a few years of successful operation, the company abandoned the policy, causing traffic to worsen in the area. The city pressured the company to again restart the program. The company complied without the city invoking sanctions (J. Bourgart, unpublished).

In sum, whether and exactly how localities will invoke sanctions specified in various policy instruments remain to be seen. The main lesson at this point is that various sanctions are specified in ordinances and agreements allowing for enforcement to proceed.

Concerning legality, courts have not yet tested the legality or reasonableness of ordinances, developer agreements, or other instruments. Still, there is little question localities may impose reasonable traffic mitigation requirements through agreements and ordinances. Generally, courts have ruled that reasonable traffic mitigation requirements and regulations are a proper exercise of police power. State constitutions expressly confer on cities the power to make and enforce within their limits all local, police, sanitary, and other ordinances and regulations not in conflict with general law. Most judicial authorities also appear to conclude that developing property is a privilege and that the dedication of land or payment of fees is voluntary in nature and developers must meet any reasonable condition imposed by local jurisdictions before issuance of building permits. Consequently, even strict traffic performance standards specified in developer agreements may be found reasonable and binding should they be challenged and tested. However, the same provisions imposed on existing employers and developers after the fact of development may not be so interpreted.

Parking Management Implementation

Parking management strategy implementation presents several issues. How can parking policies support program efforts? What is feasible and unfeasible to do?

Supportive Public Parking Rates and Policies

Some localities attend to pricing policies in publicly owned and operated facilities as a way to buttress programs and requirements. Important considerations include ensuring prices for long-term parking are not under market rates, or far below transit fares; providing location or price preference to rideshare patrons; and avoiding employee parking subsidies wherever possible. Montgomery County, Maryland, maintains market rates for long-term parking and offers discounts to carpoolers in facilities under its control. The county also recently halted block sales of parking permits to employers to discourage employer subsidies of employee parking.

Developer Agreements

As previously discussed, some localities use developer agreements to encourage pay parking for tenants and employees as in the agreement in Bellevue, Washington. However, a policy of pay parking will not necessarily lead to employees paying for parking. In buildings with multiple tenants, an owner may agree to institute pay parking at the garage or surface lot. Employees may pay the charge, but be reimbursed for all or a portion of charges by employers. Employer-subsidized parking is not uncommon in cities with pay parking. Also, such an approach will quickly generate spillover parking onto streets, commercial facilities, retail parking areas, vacant properties, and other areas not priced or regulated. The TMA in Bellevue, Washington, guards against such a possibility by contracting with employers to monitor and enforce short-term parking regulations in retail lots.

Enforcement

Enforcement is the key implementation issue with preferential parking for rideshare patrons. Many local ordinances, permit requirements, and developer agreements encourage preferential parking for car and vanpoolers. The key implementation issue is how to enforce use. One approach appropriate to garages with attendants is simply not to allow any vehicle to park in designated stalls without two or three persons in the vehicle at the time of parking. In short, no drop-offs are allowed. Alta Bates Hospital in Berkeley, California, uses this approach.

Flexible Parking Requirements

Where localities are using flexible parking requirements in codes to encourage developer-sponsored TSM and PM programs, experience suggests flexible requirements may not attract developers or lenders. It seems localities have a difficult time setting parking requirements in support of policy objectives. Several urban localities have provided for optional relaxations in parking requirements for various purposes (support of peripheral parking, ridesharing and other transit encouragements, and in-lieu funds) only to find developers not taking advantage of relaxations. Los Angeles, Hartford (Connecticut), and Seattle all provide examples (*14*). Difficulties in setting maximums, minimums, or relaxations to serve public purposes are understandable, whether in urban or suburban areas, because knowing what developers and lenders prefer to provide in the way of parking supply and setting requirement policy is not a simple task. Even if planners are able to determine the market demand and supply levels at any one time and place, the demand-supply equation is constantly varying because of everything from the state of the economy to the price of gasoline to the level of transit service. Thus, flexible parking requirements must be approached with caution.

CONCLUSION

Policy Instruments

Policy instruments are increasingly important in initiating TSM and PM programs. These instruments set the stage for monitoring and enforcement and, if necessary, for program modifications. Consequently, the design of policy instruments is important and experience suggests some lessons.

- For broad applicability of TSM and PM requirements across new and existing employers, TSM ordinances or special permits are preferred instruments. For focused requirements on new developments, developer agreement requirements are appropriate to consider. To date, there is little experience with cooperative or joint-power ordinances regulating more than one jurisdiction.
- Localities have had a difficult time establishing parking requirements and relaxations to attract developers and lenders. Apparently, it is difficult to anticipate what developers and lenders prefer in terms of parking supply and their interest in reduced supplies in return for TSM and PM.
- Parking price strategies can be encouraged by ensuring any publicly controlled parking is not priced under market rates and through developer agreements specifying pricing strategies. A danger in fashioning such policies is the possibility of encouraging spillover parking in uncontrolled areas.
- Given the wide variation in TSM and PM program results and the difficulty of knowing which strategies are most effective, localities must be cautious in establishing uniform or stringent goals in policy instruments, or prescribing implementation of specific strategies.
- Requiring program plans from developers and employers requires locality staff time and resources, which may prove to be a burden for small localities. However, requiring and negotiating plans has the advantage of tailoring TSM and PM programs to each site, a strong plus given the many program and site variables influencing program outcomes.
- Though courts have yet to test TSM ordinances and regulations, state law generally should enable localities to set TSM and PM requirements and enforcement provisions. Fines and civil penalties for failure to act in accordance with requirements also are possible under ordinances, provided usual appeal procedures are added. Performance contracts, bonds, and letters of credit are possible assurance mechanisms in developer agreements, though these must be added to covenants running with the land to provide maximum assurance. One area of caution is in stringent and binding traffic performance standards or goals. Although these may be upheld in developer agreements, presuming acceptable contractual practices were followed, ambitious and binding goals in ordinances applying to existing employers may be successfully challenged on the grounds of reasonableness.
- Exemptions to policy requirements are not very common in policy instruments, but are useful in cases with preexisting TSM and PM regulations or in cases where annual plan and survey deadlines may unreasonably burden new, expanding, or relocating employers.
- Fees and financing mechanisms in support of TSM and PM programs are not built into many local policy instruments. This practice may speed passage of policy instruments, but may hinder later monitoring, plan review, and enforcement.

Implementation

Comprehensive TSM and PM programs in localities require participation by numerous parties (public and private) and monitoring and financing mechanisms. In particular,

- Localities with comprehensive programs involve planning departments, task forces, or review committees with monitoring responsibilities and possibly private TMA organizations. Local decision makers also serve as points of appeal in the enforcement of policy instruments.
- Monitoring of mode shares, traffic levels, and parking volumes are important for determining program effectiveness.

In light of the many variables affecting travel behavior to and from employment centers, comparisons of program results with control sites without TSM and PM programs would be useful.

- Annual program costs at employment sites range from a few thousand dollars at small employers with modest programs to $250,000 at large employers with extensive programs. Both voluntary and legally binding mechanisms are in place, as well as TMA fee structures in support of private financing.

RECOMMENDATIONS

Policies

Localities do not need to institute stringent policies to ensure program success. More important than the exact policy terms and provisions is how implementation proceeds. Nevertheless, policy instruments are important for initiating TSM and PM efforts, setting commitments and resources, and establishing the evaluation framework.

Before considering local TSM and PM policies, localities should check with county, regional, and state air quality or other agencies with responsibility for transportation control or traffic mitigation. Increasingly, these agencies are developing their own trip reduction regulations, which may supersede local regulations. Los Angeles Regulation XV provides an example. Where such regulations are not developing, localities may wish to cooperate with one another to institute consistent instruments across jurisdictions. However, localities should proceed with caution because aside from Maricopa County, Arizona, there are no region-wide policy instruments serving as models.

Before selecting the type of policy instruments to develop, localities must consider their traffic problem and objectives (reduced solo driving, shift in peak travel, focus on internal versus through traffic); the source of the problem (all employers or just new employers); the best types of TSM and PM strategies to encourage; and the difficulty of getting approval for proposed instruments and implementing them. Generally, larger communities with area-wide traffic problems caused by new and existing employment should consider ordinances applicable to all medium-to-large employers. Of course, new ordinances will require public hearings, legal council review, and passage through decision-making bodies. Smaller communities with spot congestion problems attributable to new development should consider special permits and developer agreements secured by covenants. These instruments may involve less time-consuming review and consensus building with decision makers to gain passage. In addition, these instruments may require only staff review and negotiations to carry out. Developer agreements also are more appropriate for securing specific physical facilities such as bicycle racks, transit turn outs, or parking areas devoted to carpoolers.

Generally, localities should not require implementation of many specific strategies in policy instruments. Instruments may require a designated coordinator, regular reporting, annual survey, and distribution of basic rideshare and transit information. However, instruments should avoid requiring specific proportions of parking devoted to carpool stalls or the provision of discounted transit passes or imposition of specific parking prices. The preferred approach is to require and negotiate plans spelling out strategies and then to evaluate and approve these on the basis of their suitability to the site and employee population. This approach is especially recommended for special permits and ordinances applying to large areas. Localities should develop plan requirement guidelines to ease compliance and speed review. Concord, California, provides one model for such guidelines. Developer agreements for particular sites may require some specific strategies in which there is little doubt about effectiveness. For example, bicycle lockers or transit pass promotions may be required as complements to other locality programs such as bicycle paths or transit centers near the subject development. But as a general rule, localities must be cautious about specifying TSM and PM strategies because it is difficult estimating their probable effectiveness.

Localities must monitor and enforce policy instruments, but must be careful not to develop or try to enforce binding traffic or mode share standards that are too stringent, especially in area-wide ordinances and permits. Ambitious goals may invite successful legal challenge because attainment of such goals may not be possible. Localities must appreciate that some of the variables influencing traffic volumes and commuting patterns to and from employment sites are not within the control of employers or developers. Localities probably can be more secure in applying stringent and binding performance requirements to developer agreements. Experience suggests such provisions may be enforced without legal challenge. Novato, California, provides one example in the case of Fireman's Fund. Exemptions should be developed in policy instruments only to allow for cases of duplicating regulations or unusual hardship in complying with survey and reporting deadlines. Policy instruments should include provisions for financing monitoring, plan review, and enforcement. Too often, instruments ignore the need for fees and financing.

Implementation

Localities must provide local resources in support of TSM and PM programs; monitoring both of regulated and of unregulated sites as well as spillover parking should accompany PM strategies. In addition, the private sector needs to be involved in program development and appraised of the costs involved in implementing the programs.

Consideration should be given by localities for establishing a transportation coordinator position in support of TSM and PM programs, especially programs required by ordinances or permits over broad areas. The coordinator should serve to explain requirements, review plans, and survey results, provide technical assistance, and possibly centralized rideshare matching services if not available through other agencies. A coordinator may not be required where only a few developer agreements are in place or planned, though staff still needs to be designated for monitoring and review.

Localities should organize a review and support task force to help monitor the program, recommend enforcement and policy changes, and assist with special events. The private sector should participate in the task force or committee, whether through representation from the local TMA or from local employers. Private employers should be appraised of policy instrument provisions and provided information on typical TSM and PM programs, levels of effectiveness, and costs.

Monitoring of mode shares should not only occur at employers subject to TSM and PM requirements, but also at sites not subject to requirements. Additionally, localities should pay special attention to monitoring of PM strategies such as pricing or restricted supplies negotiated through developer agreements or required by parking codes. These strategies may be accompanied by spillover into residential or retail areas. If so, localities should be prepared to enforce against spillover parking. The enforcement procedures of the TMA in Bellevue, Washington, provide one model.

All program participants should be prepared to develop, monitor, and modify the local program and policy instruments over a period of several years because programs typically take considerable time to evolve and can experience declining effectiveness over the long haul.

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Improving the Effectiveness of a Transportation Demand Management Program Through Evaluation: A Case Study

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Evaluations are an important source of information for improving the effectiveness of transportation demand management (TDM) programs. Evaluations of the TDM program at Bishop Ranch, California, provide insight on how to improve services by strengthening the link between the commuter and the operation of a TDM program. Thorough evaluation requires both essential core data (commute distance, home location, commute mode, and arrival time) and supplemental information about topics such as the flexibility of work hours, analysis of subgroups, and attitudes toward various incentives and disincentives. Key issues include problems with the use of the current mode as a measure of effectiveness, lack of knowledge about the effect of flexible work hours on mode choice, and the importance of the initial design for time-series comparisons.

Continued expansion of transportation facilities is no longer economically or environmentally feasible. As a result, transportation demand management (TDM) programs have emerged as a potentially important component of the solution to urban traffic congestion (*1*). TDM is a relatively new and untested approach; little is known about its long-term effectiveness (*2*). A significant amount of research will probably go into the evaluation of TDM programs in the near future.

This research can serve two related but distinct purposes. The more traditional purpose is that of monitoring the effectiveness of the program (e.g., the number of vehicle-trips reduced or shifted to off-peak hours). A second purpose is that of providing the TDM program staff with information needed to improve services and develop strategies for marketing those services to commuters.

In simplified terms, TDM is changing commuters' behavior to make better use of existing transportation facilities. Although the commuter is the ultimate consumer of services, a direct link does not exist between the commuter's satisfaction and the operation of the TDM program. The commuter's purchase of services does not fund the program's operation. Consequently, the relationship between the effectiveness of a TDM program and a thorough understanding of the commuters it is designed to serve is often overlooked.

RIDES for Bay Area Commuters, Inc., 60 Spear Street, Suite 650, San Francisco, Calif. 94105-1512.

The collected data and the analysis process will be analyzed from the perspective of providing the TDM staff with useful information about commuter behavior. This approach to selecting and presenting information on the commuter can be incorporated directly into the operation of a commute management program. The analysis uses 3 years of evaluations at the Bishop Ranch business park in San Ramon, California, as an example. A self-administered questionnaire has been distributed annually to all employees. This 585-acre business park employs about 14,000 people in a low-density suburban setting. The TDM program is operated through a transportation management association (TMA) and staffed by two full-time employees.

The Bishop Ranch evaluations are not displayed here as examples to be emulated, but as starting points from which to discuss the potential of the evaluation process to support the work of a TDM program. The shortcomings, as well as the strong points, make the evaluations valuable examples. Because the merits of the TDM program at Bishop Ranch and the characteristics of that site are not discussed, the analysis can be applied to a broad range of settings.

A discussion and example format will be followed. The merits of collecting specific data are discussed and, when applicable, examples from the Bishop Ranch evaluation are used to further illustrate the discussion. Questionnaire and sampling design, although critical components of the evaluation process, are omitted in order to focus on the analysis process.

Several pieces of information on commuter behavior are identified as the core of the evaluation: commute distance, home location clustering, commute mode, and arrival and departure times. Supplementing these core data with information on the flexibility of work hours, analysis of subgroups, and attitudinal questions provides a more complete evaluation.

DISTANCE

For discrete data on distances traveled to work to be useful to the TDM program staff, the data must be aggregated into

TABLE 1 ONE-WAY COMMUTING DISTANCES

Year	0 to 5 mi	6 to 10 mi	11 to 20 mi	>20 mi
	Percent			
1986	18.1	8.5	40.7	32.7
1987	20.9	9.3	40.4	29.4
1988	23.1	7.9	41.5	27.6

ranges. From the perspective of a statistician, ranges containing roughly an equal proportion of respondents might be created. However, from the TDM program's perspective, ranges that represent unique commute situations are most useful. The Bishop Ranch evaluation uses four ranges (Table 1).

With the exception of the >21-mi category, these ranges were chosen somewhat arbitrarily. The point at which vanpooling becomes economically feasible is 21 mi. The lower end of the 0- to 5-mi group might be considered a good target group for the nonmotorized commute options, such as walking and bicycling. The 6- to 10-mi group might be looked at as having a high potential for transit use; because of short driving times, it might also be seen as the most difficult group to deter from driving alone. The 11- to 20-mi group is probably where carpooling starts to become attractive; the inconvenience of meeting a carpool becomes offset by savings in cost and the fatigue of driving every day.

The points at which commute distances define commute options may not always be clear; appropriate ranges may also differ from one geographic location to another. Defining these ranges on the basis of the merits of potential mode choice will provide the most meaningful data for the TDM project staff. Distance alone is obviously insufficient for geographic analysis; home location needs to be added to the equation.

HOME LOCATION CLUSTERING

Home location clusters identify concentrations of commuters in specific geographic areas. In the case of Bishop Ranch, approximately 14,000 home locations are reduced to 14 clusters (Figure 1). Each cluster is identified as an aggregation of zip codes. Zip code data collected on the questionnaire are recoded through a customized subroutine to the appropriate location. Although elaborate for a single evaluation, writing the subroutine is well worth the effort if the project will be evaluated periodically; comparing changes in home location patterns provides the TDM staff with a preview of where to concentrate future efforts. The following excerpt from the Bishop Ranch evaluation helps demonstrate the utility of cluster analysis.

> In order to examine in greater detail employee home locations and temporal changes, zip codes were aggregated into groupings along major commute corridors (Figure 1). Areas H and J, which are directly north and south of Bishop Ranch along the 680 corridor, house almost 50 percent of the Ranch's employees. Area H is remarkably stable when compared with 1987 densities (up only 0.2 percent); area J accounts for the majority of the increase in short-distance commute (up 1.2 percent). Area M, south of Interstate 580, appears to be where most of the decrease in medium-distance (6- to 10-mi) commutes occurred.

Cluster analysis is highly dependent on the geography of the region. Areas should generally be oriented along commuter corridors, considering major access routes and transit service. Because the concentration of employees will be greater closer to the work source, the areas will tend to increase in size further out. Keeping the number of areas manageable without combining areas with unique characteristics is the most challenging part of defining areas.

COMMUTE MODE

Current commute mode is probably the most basic piece of information collected for the evaluation. Unfortunately, accurate information is sometimes difficult to gather because commuters use different modes on different days or multiple modes on the same day. Asking for the normal commute mode often elicits a multiple response, which generally must be eliminated from the data file. Figure 2 shows one approach that seems to elicit an accurate response in a suburban setting. Multiple answers with an explanation are precoded for data entry. Multiple answers without an explanation must be eliminated; however, these cases have been relatively rare at Bishop Ranch.

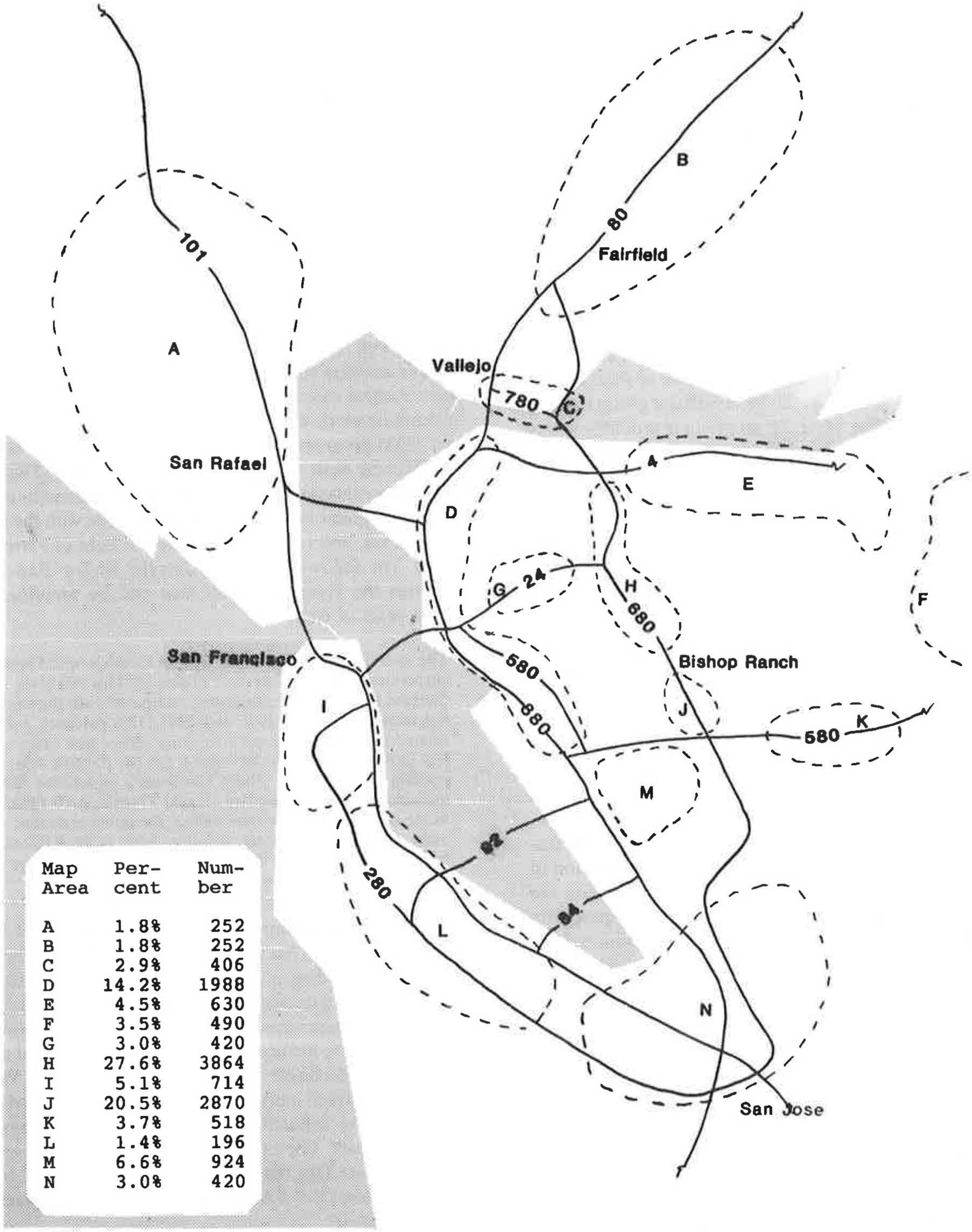
Commute mode data are not particularly useful for improving a TDM program's effectiveness when viewed in isolation. They become most meaningful when (a) compared with numbers from previous years, (b) compared with the results achieved by other programs in a similar environment or with the region's normal level, and (c) when examined in light of various subgroups. The following excerpt from the Bishop Ranch study illustrates the types of trends that can be identified from several years of data.

> The overall drive-alone rate at Bishop Ranch is up 2.5 percent; carpooling is down 2.2 percent (Table 2). This relatively small increase in solo driver commutes, compared with the substantial increase between 1986 and 1987 (12.6 percent), may be related to the population's stabilizing (fewer new employees, less movement of home locations). On the positive side, vanpooling is up slightly. There has been a significant shift of individuals from the Bay Area Rapid Transit (BART)/Shuttle mode since 1986. More than half of the individuals that were using that mode in 1986 are no longer using it. With the exception of club bus riders, this is a much greater percentage shift than found in any of the other modes.

One of the shortcomings of the data collected at Bishop Ranch is that it does not identify intermodal shifts. For example, in the preceding paragraph a significant shift from the BART/Shuttle mode was pointed out. What is not known is to which mode these former BART/Shuttle users have switched. A more complete picture, including both former and current modes, would facilitate a more complete analysis. Without knowledge of which individuals changed modes and what factors might have influenced that change, many unanswered questions remain. For example, did a large percentage move their residence? Did they all switch to vanpooling? Is there something about the BART/Shuttle option with which they were dissatisfied?

A second comparison that adds perspective is modal use at comparable sites. Without this comparison to provide some sort of benchmark the progress of a TMA is difficult to put into perspective. An excerpt from the Bishop Ranch evaluation follows.

> In order to place the mode split characteristics of Bishop Ranch in some perspective, Table 3 provides information on other



Map Area	Per-cent	Num-ber
A	1.8%	252
B	1.8%	252
C	2.9%	406
D	14.2%	1988
E	4.5%	630
F	3.5%	490
G	3.0%	420
H	27.6%	3864
I	5.1%	714
J	20.5%	2870
K	3.7%	518
L	1.4%	196
M	6.6%	924
N	3.0%	420

FIGURE 1 Employee home locations.

suburban locations around the Bay Area. With the exception of the San Francisco Airport, the drive-alone rate at Bishop Ranch is lower than at the other projects. All of the sites listed here are involved in some type of traffic mitigation efforts.

Although these sites might be ranked in order of drive-alone rate, with the lowest being most effective, the "why" portion of the equation remains unanswered. An in-depth understanding of the setting and the organizational structure is needed to understand why one may be more effective than another. For this reason, a comparison with the ambient level (which the Bishop Ranch evaluation does not include) may be more useful. In many regions, however, a current ambient level may be difficult to find.

Even the comparisons of modal change from year to year within the project site have some inherent error. Along with changes influenced by the efforts of the TDM program, other variables, such as gas prices, congestion, home relocations, and employee turnover, exert considerable influence on the choice of commute mode. Because so many factors affect mode choices, the success of a TDM program should not be evaluated solely on the basis of observed changes in mode split.

Thus, a significant gap in the evaluation remains. What should be used as a basic measurement of a program's success?

TABLE 2 NORMAL COMMUTE MODE

	1986	1987	1988
Drive Alone	55.1%	67.7%	70.2%
Carpool	26.6%	18.5%	16.3%
Vanpool	7.7%	8.3%	8.7%
Club Bus	2.8%	0.8%	0.7%
BART/Shuttle	6.2%	3.3%	2.5%
Other	1.3%	1.4%	1.7%

One approach might be to measure the level of awareness of the services or incentives offered through the TDM program. Another, which might be complicated in terms of questionnaire design, would be to identify individuals that made mode changes and determine whether the program's services influenced those changes.

MODE AND DISTANCE COMBINED

Strong relationships have become apparent from examining the combination of mode and distance characteristics. Along with highlighting some of the obvious characteristics (e.g., the drive-alone rate decreases as mileage goes up), the excerpt following includes some of the most valuable insights provided by the evaluation.

As noted in the earlier surveys, the drive-alone rate decreases as commute distance increases. Both carpooling and vanpooling rates are at their highest for the >21-mi category. Table 4 presents mode and distance changes between 1986 and 1988 (1987 was omitted to keep the table "reader friendly"). The increase in number of individuals driving alone is fairly even across the mileage ranges (i.e., it is not simply carpoolers moving closer and deciding to drive alone that has pushed up the drive-alone rate). The 11- to 20-mi group shows the sharpest increase in driving alone, and its share of the Bishop Ranch population has remained relatively constant. The sharp decrease in BART/Shuttle use noted earlier is most evident among the medium- and long-distance commuters.

Although large amounts of data are commonly digested and discussed at length without any practical recommendations resulting, the data from the mode-by-distance analysis can be directly linked to recommendations at Bishop Ranch. The increase in drive-alone rate across all mileage categories, as well as the sharp increase in the 11- to 20-mi category, both led to recommendations to the TDM program staff.

ARRIVAL AND DEPARTURE TIMES

For some TMAs, spreading the peak may be as important as increasing vehicle occupancy. Data collection and analysis for

How do you <u>normally</u> commute to work?

a. ___ Drive alone	f. ___ Bus
b. ___ Carpool	g. ___ BART
c. ___ Vanpool	h. ___ Bicycle
d. ___ Dropped Off	i. ___ Shuttle
e. ___ Walk (3 blocks +)	j. ___ Other

If you checked more than one method of commuting, please explain: ______________________________

____

FIGURE 2 Sample format for eliciting commute mode.

this purpose are relatively straightforward. Information on normal arrival and departure times are adequate for estimating the spread of the peak.

A more difficult issue relating to work hours is that of flextime and its effect on vehicle occupancy. Studies at Bishop Ranch and nearby Pleasanton (*3*) have provided some interesting insights into this issue. An initial examination of the data indicates that flexibility has a negative influence on the propensity to share rides. As flexibility increases, so does the drive-alone rate (Table 5).

Unfortunately, a literal interpretation of these data may not be accurate. A subsequent test of the question used in the survey indicated that the wording may have caused some confusion. The question was whether respondents' daily work hours were fixed, flexible up to 30 min, or flexible by more than 30 min. Some respondents may have indicated that their hours were fixed, not because of company policy, but because their carpool or vanpool required them to have a fixed schedule. Consequently, a higher percentage of those who were ride-sharing appeared to have inflexible hours. Because of this ambiguity, what might have proved to be one of the evaluation's clearest recommendations may be premature. The question needs to be reworded to ask specifically about company policy and flexibility of work hours in relation to commuting (Figure 3).

An alternative approach to considering the merits of flextime is to assume that those employees arriving outside the peak hour (i.e., 7:00 to 8:00 a.m.) are exercising a flextime

TABLE 3 DRIVE-ALONE RATES AT OTHER SUBURBAN LOCATIONS

	1986	1987
Santa Clara County Civic Center	81%	77%
Contra Costa Center (Pleasant Hill BART)	78%	81%
Concord (Downtown Area)	80%	80%
City of Pleasanton	84%	86%
Hacienda Business Park (1988)	--	75%
P.I.B.C. (South San Francisco)	--	88%
San Francisco Airport	--	66%
Bishop Ranch	**68%**	**70%**

-- = data not available

TABLE 4 MODE AND DISTANCE

	0 to 5 mi		6 to 10 mi		11 to 20 mi		>20 mi	
	Percent							
Mode	1986	1988	1986	1988	1986	1988	1986	1988
Drive alone	77	86	70	81	57	74	36	48
Carpool	17	9	25	16	28	16	30	23
Vanpool	0	0	1	1	4	5	19	23
Club bus	0	0	0	0	0	0	9	2
BART/Shuttle	0	0	0	2	9	4	6	2

TABLE 5 FLEXIBILITY BY MODE

	Fixed	Flex To 30 Min	Flex More 30 Min	Other
Drive Alone	65%	69%	73%	83%
Carpool	19%	18%	14%	11%
Vanpool	12%	7%	8%	6%
Club Bus	1%	1%	0%	0%
BART/Shuttle	3%	3%	2%	0%

option. Then, mode can be compared with arrival time, as in the following table.

Arrival Time, a.m.	*Percent Driving Alone*
Before 6:30	89.3
6:30 to 7:00	69.3
7:00 to 7:30	57.4
7:30 to 8:00	65.6
8:00 to 8:30	77.8
8:30 to 9:00	82.9
After 9:00	87.0

The data clearly show an increase in the drive-alone rate outside the peak hour. There are two potential interpretations. One is that the higher drive-alone rate outside the peak period demonstrates that employees are taking advantage of the flextime privilege to drive alone at a more convenient hour. The other interpretation is that it is more difficult to make ridesharing arrangements outside the peak period, because fewer individuals commute at those times. Because the Bishop Ranch TDM program's goals are currently for a peak-period reduction, moving trips out of the peak is valuable. If the emphasis should change to a more narrow vehicle occupancy perspective, this analysis would provide useful input into the development of an appropriate flextime policy.

SUPPLEMENTARY DATA

The information discussed to this point represents the core of the evaluation. The possibilities for supplementing these core data are infinite. No attempt is made to create a comprehensive list of supplementary data. However, if the Bishop Ranch TDM program is representative of other programs, the supplementary data identified will provide strong support for the evaluation of other TDM programs. Two potentially important topics that the Bishop Ranch survey does not address are parking and the need for vehicles for noncommute purposes (e.g., midday errands or dropping off and picking up children).

SUBGROUPS

As is the case with distance categories, subclassifications of commuters are of little value unless there is a practical way for the TDM program to target each group, such as a direct line of communication or a distinct service area. The two subgroups highlighted are based on job classification and employer.

Among the most common but least useful subgroupings is that of job classification. A person's perception of his or her own job classification often differs from another's perception, which results in inaccurate data. But more important, the TDM program staff has limited ability to affect the commute decisions of individuals from different job classifications. In a primarily white-collar setting such as Bishop Ranch, the differences in commute habits noted between people in different job classifications have been minimal.

> Between the three major categories of employees (executive or manager, clerical or administrative, and professional or technical), the propensity to drive alone is not different, although clerical or administrative employees are somewhat more likely to ride-share. The executive or manager group appears less likely to vanpool, but more likely to carpool.

In a multitenant setting such as Bishop Ranch, examining the data by subgroups of employers is useful. Apart from providing input to the TDM program, employer data are useful for checking the validity of the sample. Because the actual number of people employed by each employer is generally known or can be estimated by the TDM staff, a comparison of the survey response with the population is a good check of the sample's representativeness. Grouping people by their employer is done on the assumption that there is a difference in commute habits of employees at different employers. At Bishop Ranch, the evaluations have indicated notable differences.

Combining employer information with the core data enables the TDM staff to have an individual profile of each major employer. The example presented in Table 6 compares commute habits by employer and mode. The data are difficult to interpret if too many individual employers are identified. Because Bishop Ranch houses two large employers and numerous small employers, all responses were grouped into three categories. Part of the analysis from the Bishop Ranch evaluation follows.

> Table 6 presents a comparison of the mode for respondents from three major groups at Bishop Ranch. Company B is holding steady in the drive-alone category and actually increasing in the carpool and vanpool categories. Company A shows the largest increase in driving alone and decrease in carpooling, but their drive-alone rate is still well below that of the other two groups. The lower drive-alone rate at Company A is caused by a much higher carpool rate than at Company B; their vanpool rates are nearly equal.
>
> The All Others group appears to be a potential target for vanpooling. Their distance characteristics are similar to those of the other two groups, but their vanpool use is well below that of Companies A and B. However, they tend to start work somewhat later (only about half are at work before 8:00 a.m., compared to over 60 percent of Company A and B employees), and vanpools tend to arrive quite early.

Another useful comparison at Bishop Ranch is location within the park. Similar to the preceding employer analysis, the ultimate usefulness of this analysis is based on the ability to treat each location as a separate market. Commuters from individual home locations are another subgroup that the TDM staff may find useful.

ATTITUDE VERSUS FACTUAL QUESTIONS

When providing information to a TDM program on how to better design and market services, there will always be a need

```
Would your employer allow you to adjust your
work hours for commuting purposes?

___ no, my hours are fixed

___ yes, but by no more than 30 minutes

___ yes, pretty much as needed

___ not sure
```

FIGURE 3 Proposed format for flextime researcn.

TABLE 6 SELECTED EMPLOYERS BY MODE

	1986	1987	1988
	Drive Alone		
Company A	48%	59%	63%
Company B	**63%**	**73%**	**73%**
All Others	71%	78%	80%
	Carpool		
Company A	32%	26%	22%
Company B	**19%**	**11%**	**13%**
All Others	19%	13%	12%
	Vanpool		
Company A	9%	11%	11%
Company B	**8%**	**9%**	**10%**
All Others	4%	4%	3%

to go beyond factual commute patterns and explore personal opinions, attitudes, and preferences. Responses to these questions have a potentially larger margin of error because of the dynamic nature of commute behavior and the subjective nature of the questions. Small differences mean little in response to attitudinal questions.

Two distinct types of attitudinal questions have been used in the Bishop Ranch evaluations. One investigates the reason behind current behavior, e.g., What is the main reason you commute the way you do? Question design is an important consideration with attitudinal questions because of the legitimacy of multiple answers and the legitimacy of different answers under slightly different conditions. Most commuters do not choose a commute mode for only one reason and can not quickly rank their reasons in terms of importance. The second type of attitudinal question asks about potential future behavior, e.g., During the upcoming highway reconstruction project would you consider the following alternatives? This type of question can place options on a relative scale and help the TDM staff decide where to focus their effort.

COMPARISONS OVER TIME

Time-series comparisons add a new dimension to an evaluation. A table with the commute distances of 1 year provides a good reference point, but a table with 3 or 5 years of information identifies trends and leads to insights that would not be obvious from a single year's data. In addition, TDM is a relatively new, evolving field, and commute behavior is constantly changing. Its dynamic nature and the value of time-series comparisons underscore the need for careful initial design. The time to expend extra effort, get a second opinion, and think problems through thoroughly is at the questionnaire and sampling design stage.

RECOMMENDATIONS TO THE TMA

After developing pages of detailed analysis of commuter behavior, pages of insightful recommendations might be expected. Although conclusions have been reached about the effectiveness of the program at Bishop Ranch (e.g., vehicles removed from the peak period), not until the 3rd year of data had been analyzed were any substantial recommendations for program direction offered. The following two recommendations are the first to directly provide input on improving the effectiveness of the TDM program's work.

1. The low vanpool participation rate of the All Other companies (i.e., not Company A or Company B) indicates that they are a high-potential group at which to direct vanpool formation efforts. Two characteristics of the All Other group are important to remember. They tend to start work a little later, making vanpool formation more difficult, and they have a higher BART/Shuttle rate than the other groups. Encouraging vanpooling with this group may move trips from the BART/Shuttle mode.
2. The 11- to 20-mi commuters may be the key group to work with in the near future. They are the largest group—their size has actually increased over the past 2 years. They have shown the largest increase in driving alone of all the mileage ranges. The long-distance group has high motivation to rideshare, and there are few tangible incentives to offer the short-distance commuters (plus, it can be argued that they are already part of the solution). This reasoning leads to the 11- to 20-mi group as an excellent audience for targeted marketing and potential new services.

CONCLUSIONS

Two objectives inspired this look at the data collection and analysis process. First, TDM is a young and largely unproven field; good data collection and analysis are critical to its future. Second, it is too easy for the process to become mechanical. The same data can be collected and observations and even recommendations can be made without enlisting the creative thought process. Some of the questions raised here may strengthen the link between the service provided by the TDM program and the needs of the commuter. Commute distance, home location, commute mode, and arrival and departure times are identified as the core elements of a commute program evaluation. These data alone, however, would make for an unimaginative analysis. Supplementing this information with variations, such as flexibility of hours, analysis of subgroups, and attitudinal questions, can provide the insights needed to suggest ways to improve a TDM program's effectiveness.

As several years of data on a TDM project are accumulated, the temporal comparisons become much more valuable than the individual data sets. In order to ensure that subsequent data sets are comparable, it is important to start with a good design.

The geographic variables—commute distance and home location—are a key to orienting the evaluation. Both commute distance and home location clustering are indicators of the potential of various modes. The process of defining distance ranges and cluster components brings the evaluation to a more practical level. Defining distance ranges and home location areas that can be targeted as separate markets is the key to making these data useful.

Current commute mode is the basic element in a TDM program evaluation. Often the evaluation of a program's success is tied to this measurement. Unfortunately, the dynamics of observed change are not well understood for three reasons. First, it is sometimes difficult to interpret the response of a commuter who uses multiple modes or different modes on different days. Second, it is difficult to find control projects or an ambient modal split level with which to compare measurements at the project site. Finally, a great number of variables beyond the control of the TDM program exert influence on commute behavior, making even periodic comparisons of the same program inaccurate. More detail is needed on the motivation behind individual changes in commute behavior and their relationship to services offered by the TDM program to accurately assess their effect.

Another illustration of the immaturity of TDM programs is the lack of knowledge about the effect of flexible work hours on mode choice. For some time, it was assumed that fixed work hours were a deterrent to ridesharing, making it difficult to coordinate ridesharing hours. However, some recent evidence has suggested that too much flexibility may actually discourage ridesharing. Further study is needed, and with the appropriate questionnaire design, the TDM program evaluation is a good vehicle with which to determine an appropriate flextime policy.

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Development of a Short-Range Travel Demand Management Program: The I-35W Experience

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Travel demand management (TDM) is a technique being used in many metropolitan areas to address growing traffic congestion problems. TDM strategies provide for better management of the transportation system, with emphasis on maximizing the number of people carried, rather than vehicular volumes. TDM covers a variety of actions that better manage the demand on transportation facilities by acting to shift more commuters into transit and multioccupant vehicles and into less congested travel times. One approach to the development of a TDM program was used in the I-35W corridor in Minneapolis, Minnesota. The process provided a vigorous examination of the effectiveness of existing TDM measures, travel markets, the evaluation of additional TDM strategies, and the development of a short-term TDM program. The basis for the examination of the effectiveness of potential TDM strategies was the development and application of a microcomputer spreadsheet model. The process, which was conducted in a relatively short time period with a modest budget, may prove beneficial to other areas facing the same types of problems.

Traffic congestion in growing metropolitan areas is a problem receiving increasing attention. Concerns about urban mobility rate high in surveys around the country and have been the focus of numerous recent news articles and reports. Complicating the situation, most metropolitan areas are facing congestion issues in a time of limited resources, both in terms of funding for highway expansion and of land for new construction.

One approach being taken to address these issues is better management of the transportation system, with emphasis on maximizing the number of people carried, rather than vehicular volume. Travel demand management (TDM) is a technique being actively pursued in many parts of the country. TDM covers a variety of actions that better manage the demand on transportation facilities by acting to shift more commuters into transit and multioccupant vehicles and into less congested periods. TDM actions focus on inducements to ridesharing, transit use, and peak-period spreading, combined with deterrents to single-occupant automobile use.

TDM plans often need to be developed under relatively short time frames and with limited resources. Problems endemic to this approach include an unclear definition of the problem, addressing potential solutions in a hit-or-miss fashion, and an overly ambitious program that tries to address all possible approaches. These problems can result in a plan that is not focused and spreads resources too thin by trying to do too much; such a plan may build unrealistic expectations and ultimately lead to the failure of the program.

One logical approach to the development of a TDM program was used in the I-35W corridor in Minneapolis, Minnesota. The process was conducted in a relatively short time period, with a modest budget. However, the process provided for a rigorous examination of the effectiveness of existing TDM measures, travel markets, the potential effect of additional TDM strategies, and the development of short- and long-term TDM programs for the major travel markets in the corridor. The basis for the examination of the effectiveness of potential TDM strategies was the development and application of a microcomputer spreadsheet model. This model provided a low-cost tool, easily understood and used, for examining the effect of alternative TDM scenarios.

The approach used in the development of the I-35W TDM program, especially the microcomputer spreadsheet model, may prove beneficial to other cities facing the same types of problems. The relative ease of application and the more focused approach this process provides, while being relatively quick and inexpensive, should recommend the use of the process in other situations.

I-35W CORRIDOR

I-35W is an important element of the Twin Cities metropolitan freeway system. The 11-mi segment leading southward from downtown Minneapolis through the cities of Richfield, Bloomington, and Burnsville carries approximately 170,000 vehicles per day. This segment, which is shown in Figure 1, has been identified for improvement by the Minnesota Department of Transportation (MN/DOT) and the Metropolitan Council because of high congestion and accident levels. The initial scoping decision-making process was complete in 1988, and work on the environmental impact statement (EIS) is underway.

One of the issues that emerged during the scoping process was the need to more closely examine the use of TDM activities in the corridor. A variety of transportation system management (TSM) elements, including strategies classified as TDM actions, has been in use in the corridor since the early 1970s. I-35W was one of the first highway corridors in the

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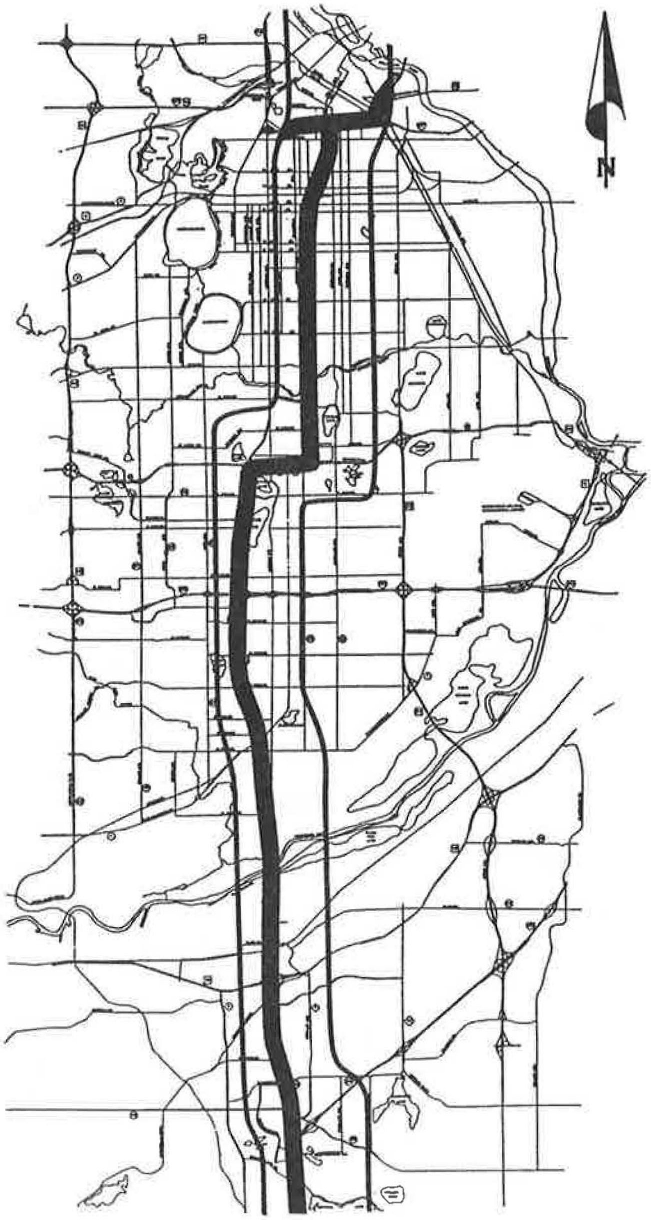

FIGURE 1 I-35W corridor study area.

country to successfully implement many of these actions, which included ramp metering, high-occupancy-vehicle (HOV) bypass lanes at ramps, an extensive express bus network, rideshare promotions, and an overall traffic surveillance and monitoring program.

Most of these activities were originally implemented in the early and mid-1970s. Thus, it was time to reexamine existing elements, current travel behavior, travel markets, and potential new strategies in the corridor. New TDM strategies, including the different institutional arrangements such as transportation management organizations (TMOs), have emerged over the last few years.

PROCESS

The Regional Transit Board (RTB), which is responsible for transit planning, policy making, and administration in the seven-county Twin Cities metropolitan area, took the lead role in the development of the I-35W TDM program. The effort was coordinated with the I-35W EIS process. A project management team (PMT), consisting of representatives from involved agencies and communities, actively participated in the development of the program. The RTB used a three-man consulting team to assist with the evaluation activities and development of the I-35W TDM program.

The process used to develop the I-35W TDM program is shown in Figure 2. The first step was to examine the effectiveness of existing TDM and TSM actions in place or used in the corridor. This examination evaluated the effect of existing measures and identified areas for improvement or expansion. The second step was to identify major travel markets being served by I-35W, to ensure that TDM strategies focus on the prominent travel markets, instead of wasting resources on markets with little impact on the facility. The third step was to identify additional TDM strategies or the refinement of existing activities for further examination. These strategies were then evaluated for the major markets through the use of a microcomputer spreadsheet model.

The results of this process were evaluated by the PMT on the basis of the following measures:

- Existing performance of current activities,
- Potential for added impact,
- Affordability,
- Acceptability, and
- Implementability.

WORK TASKS
- Examination of the effectiveness of existing actions
- Identification of major travel markets
- Identification of potential TDM strategies

↓

MARKET ANALYSIS
Evaluation of the effectiveness of different strategies by the major travel markets

↓

EVALUATION MEASURES
- Performance
- Potential impact
- Affordability
- Acceptability
- Implementability

↓

I-35W TDM PROGRAM

FIGURE 2 I-35W TDM program development process.

The advantages and disadvantages of each strategy were identified and discussed by the PMT, as were potential barriers and problems. The result of this effort was the development of a short-term action plan for the I-35W corridor and the identification of longer-term strategies. The short-term program focused on overall strategies applicable on a corridor-wide basis and those specifically tailored to the three major markets.

INVENTORY OF EXISTING TDM ACTIVITIES

The first step in the process was an inventory and review of the TDM and traffic management elements that had been in use in the corridor since the early 1970s. Most of these activities were implemented as part of the I-35W Urban Corridor Demonstration Project and the Bus-on-Metered Freeway System (*1*). The project, which was funded primarily through a federal demonstration program, included the implementation of a traffic management system and express bus network, along with supporting components.

Three different categories of existing TDM and traffic management activities were reviewed. These categories—I-35W traffic management, transit, and ridesharing—included the following activities:

1. I-35W traffic management
   - Traffic management center,
   - Ramp metering,
   - HOV bypass ramps,
   - Highway Helper program,
   - Voluntary truck restrictions, and
   - Other activities.
2. Transit
   - Express service on I-35W,
   - Local service in the corridor,
   - Park-and-ride lots,
   - Supporting downtown elements (contraflow bus lanes, Nicollet Mall, and downtown dime zone), and
   - Employer pass subsidies.
3. Ridesharing
   - Areawide marketing,
   - Corridor-specific matching and outreach programs,
   - Corridor vanpool programs,
   - Downtown vanpool staging areas, and
   - Downtown parking management strategies (preferential parking and free parking for carpoolers and vanpoolers).

The information on each of these activities was examined. Sources of information included transit ridership and bus mileage levels, park-and-ride lot use, traffic volumes and cordon counts, ramp volumes, safety and accident levels, and other data. As is often the case in a review over an 18-year period, some data were either not kept or were not available. The best available information was used for each type of activity.

In general, the existing TDM and traffic management elements had been relatively successful at maintaining the efficiency of I-35W during a time of increasing travel demand. However, most of the improvements and their resulting effects were accomplished during the 1970s. Since the early 1980s, few additional improvements have been made. In some instances, the level of activity has declined. The analysis of one element, the express bus system, provides an example of the type of analysis conducted in this step.

Between 1971 and 1974, the Metropolitan Transit Commission (MTC) implemented 12 I-35W express routes as part of the Bus-on-Metered Freeway demonstration. Three additional I-35W flyer routes, as the express service is called, were added in the late 1970s. The I-35W flyer routes provide peak-period express service from suburban communities to downtown Minneapolis. The service is oriented toward park-and-ride lots, with some neighborhood stops.

Historical mileage, ridership, and level of service information from the MTC was examined for these routes. The mileage and ridership information is shown in Figures 3 and 4. In 1989, approximately 7,335 mi of service per day were provided by these express routes. This number represents a decline in service from a high of approximately 7,900 mi in 1980. Daily ridership in 1989 was approximately 9,500 passengers. This number represents a decline in ridership from a high of 11,700 passengers in 1980.

This analysis indicates the significant impact that the express transit service has on the I-35W corridor. The service, which represents the best express route network provided in any corridor in the Twin Cities, keeps a significant number of automobiles off I-35W. Without the transit service, the additional automobiles on the system would further congest the facility, creating the need for additional capacity.

However, the analysis also indicates that the express bus service, as reflected both by passenger volumes and by miles of service, has declined. Service miles and passenger levels both peaked in 1980 and have declined during most of the 1980s, until a recent leveling off and slight increase in 1988. The potential has existed for increasing both service and ridership levels on the I-35W express bus service. Service improvements have been identified as a potential strategy to be considered and evaluated with the microcomputer spreadsheet model.

MARKET ANALYSIS

A market analysis identified the origins and destinations of travelers using portions or all of I-35W. This analysis indicated the location and general size of the different markets, so that specific strategies could be better tailored to each. This step was important, because each market had different characteristics and thus needed different strategies and implementation approaches.

Estimated daily home-based work (HBW) trips for the years 1980 and 2010 were examined in this step, using regional forecasts obtained from the Metropolitan Council of the Twin Cities. The select link assignment technique was used by the Metropolitan Council to identify home origins and workplace destinations for I-35W commuter traffic at five critical locations. Inbound and outbound traffic were examined separately. The resulting data were mapped and analyzed. Estimated mode splits for the markets identified were also obtained from the regional forecast, providing base-case shares of transit use, group-ride automobile use, and single-occupant automobile use.

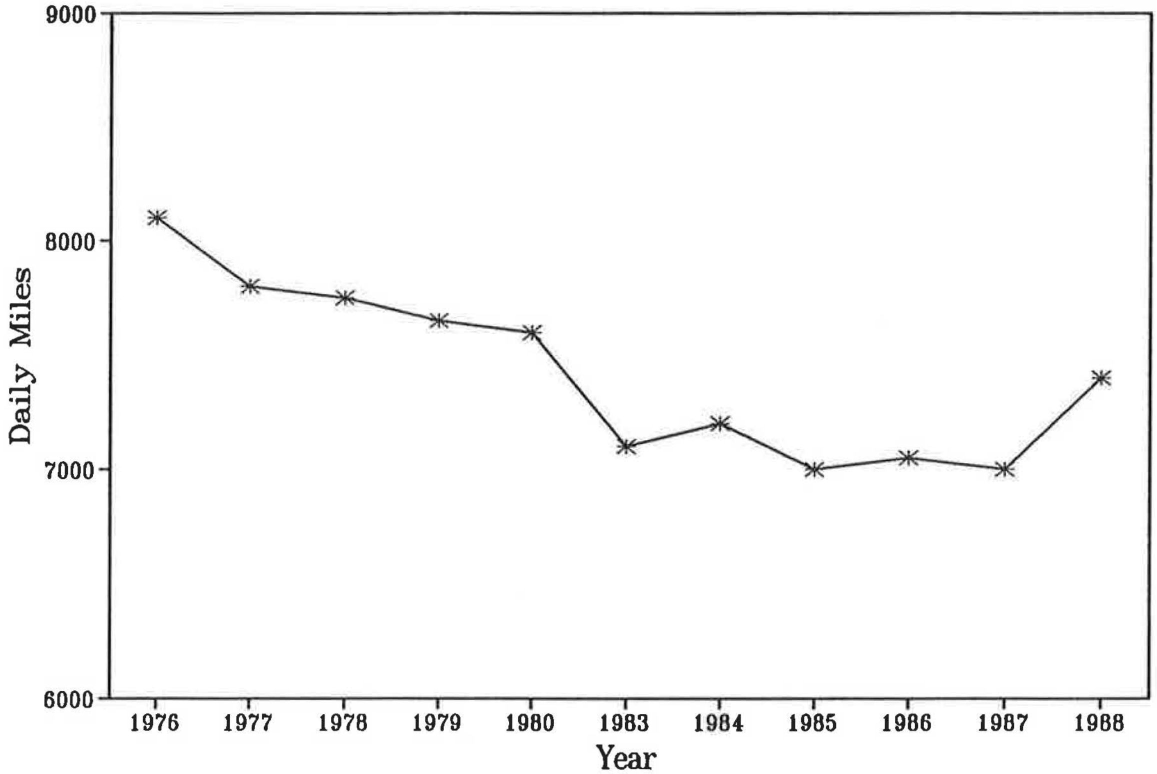

FIGURE 3 Daily express miles on I-35W.

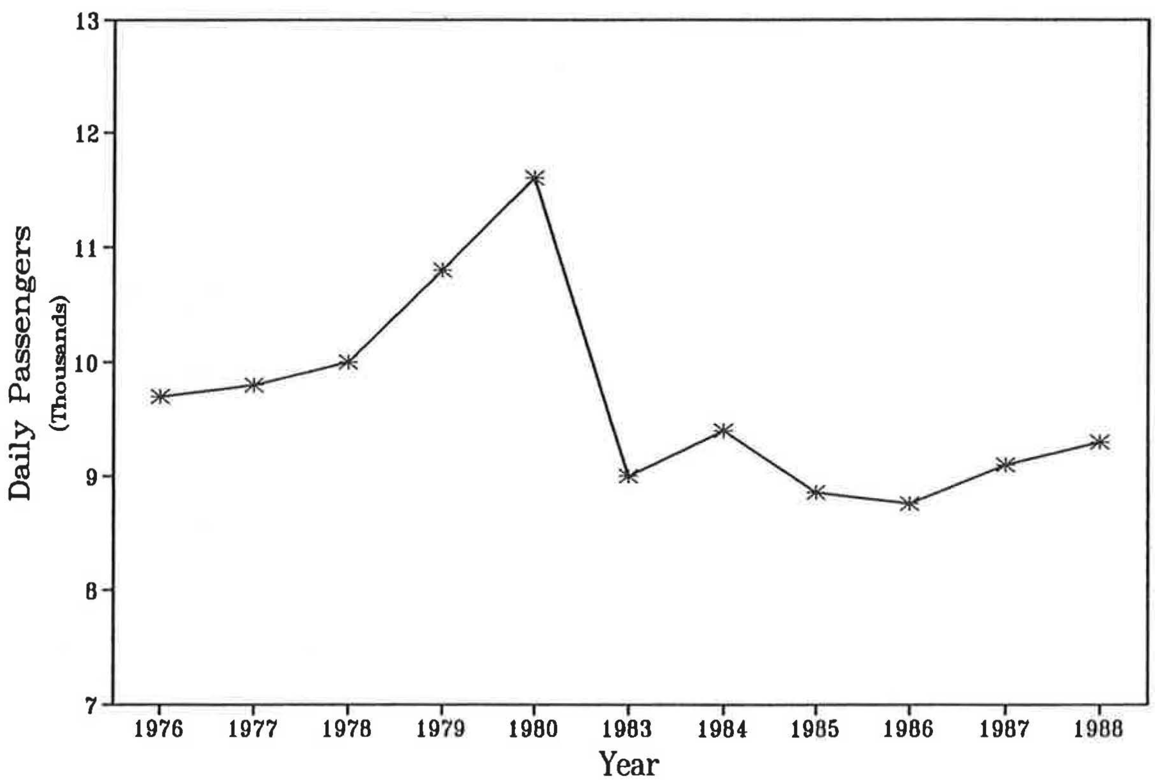

FIGURE 4 Daily express passengers on I-35W.

TABLE 1 RELATIVE CONTRIBUTION OF DESTINATION MARKETS TO DAILY HOME-BASED WORK PERSON-TRIPS ON I-35W

Destination	1980 Percent	2010 Percent
Minneapolis CBD	21.0	20.6
CBD Fringe-South	7.0	5.1
University of Minnesota	3.5	2.8
Minneapolis South	10.1	6.2
1-494 West	12.3	14.1
I-494 East	9.5	10.2
Bloomington	5.4	7.6
Burnsville	1.6	3.0
Eden Prairie	1.3	3.6
CBD Fringe-Northwest	2.9	2.6
CBD Fringe-Northeast	10.1	8.5
All Others	15.4	15.8
Total	100.0	100.0

The results, as presented in Table 1, identified the central business district (CBD) of Minneapolis as the destination for the largest number of trips on I-35W. When the University of Minnesota and the CBD fringe are added, this general area becomes even more significant. Areas along the I-494 circumferential freeway and southern Minneapolis also claimed large percentages of trips. The suburban communities of Bloomington, Burnsville, and Eden Prairie attracted a smaller percentage, but their numbers are forecast to almost double by the year 2010.

On the basis of this information, three general markets were identified for further examination of specific TDM strategies. These markets, shown in Figure 5, are the Minneapolis CBD, the CBD fringe, and the I-494 corridor. The CBD fringe area includes the University of Minnesota, the employment area northeast of the university, and the hospitals and Honeywell south of the CBD. The I-494 market was divided into two markets: northbound trips on I-35W from the southern suburbs to destinations along I-494, and southbound trips on I-35W from the north to destinations along I-494.

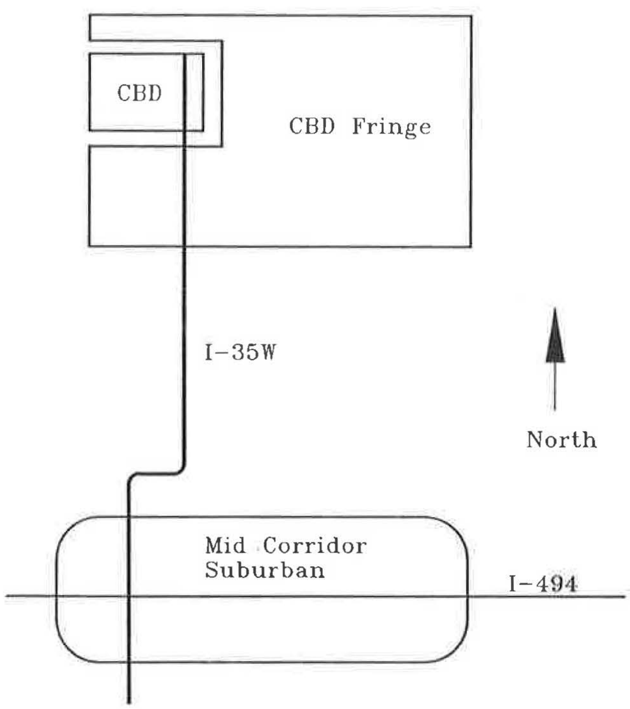

FIGURE 5 TDM general markets in the I-35W corridor.

TDM MICROCOMPUTER SPREADSHEET EVALUATION MODEL

A microcomputer spreadsheet model was developed by Richard H. Pratt, Consultant, Inc., and COMSIS to evaluate the effectiveness of the TDM strategies being proposed. The spreadsheet model is an analytical tool that combines both pivot-point mode choice modeling and experience-based calculations of the shifts in mode share and traffic peaking that result from different TDM strategies. The effect of each TDM strategy, in terms of the potential reduction in the number of vehicles, is calculated.

Included in the microcomputer spreadsheet model is the capability to apply the estimated mode share and peaking shifts only to that portion of HBW travel associated with employers estimated to be participating in the employer-dependent TDM strategies in question. In this manner, the dissipation of TDM trip reduction when moving from the level of participating employers to the level of all area employers is addressed (*2*). Dissipation related to intermixing with other unaffected traffic is addressed by the overall analytical approach of estimating vehicular reduction. This reduction is estimated only on the basis of those HBW trips for the market identified in the I-35W select link analysis.

Figure 6 shows a diagram of the analytical process encompassed by the TDM spreadsheet evaluation model. The starting point is the product of the previously described market analysis. The market analysis provides the number of HBW trips to the different workplace markets at specific locations along I-35W. At each location, one market is analyzed at a time.

The origins, destinations, and travel modes of trips on each section of I-35W were derived from Metropolitan Council commitment by employers, developers, and others in the private sector.

The passive approach to ridesharing had a low impact. The more active approach had a higher impact, but not as high as some other strategies. Experience indicates that ridesharing strategies have more impact when the private sector has more active involvement and a stronger commitment. This involvement can occur through the actions of individual employers, but the strategy has been more effective when employers and developers act together through TMOs or some other organization.

Parking Management

Two approaches were examined in the modeling process: preferential parking and an additional parking fee. In the first approach, preferential parking would be provided for carpools and vanpools. Approaches to preferential treatment might include providing spaces close to the front door or inside, heated parking for rideshare vehicles. Two different parking fees, a $1.50 and a $3.50 charge, were examined in the modeling process for single-occupant automobiles. Actual implementation of such an approach could take many forms, including charging for parking now being provided free by employers, reducing rates for rideshare vehicles, or adding a surcharge to single-occupant vehicles.

Overall, the impact of preferential parking is low, whereas the use of an additional parking fee has a medium-to-high impact. Experience with the free downtown carpool parking program in Minneapolis indicates that people view reduced or free parking for carpoolers as a significant benefit.

Variable Work Hours

Increasing the use of variable work hours was examined. A 4 to 37 percent participation rate for office employers was modeled. This range, determined on the basis of previous national studies, added approximately 3 percent to the overall effectiveness of the different strategies. The traffic volume information examined indicated that although the hours on either side of the peak period are close to capacity, a limited capacity for additional vehicles is available.

Traffic Management

In addition to these strategies, the expansion and enhancement of traffic management activities were examined. Many activities focus on safety and incident management and could not appropriately be included in an effectiveness evaluation. However, because they can have an important effect on the capacity of the facility, they were addressed in the program. A public information program on the use of ramp metering and proper merging, additional enforcement of HOV bypass ramps, and additional Highway Helper vehicles and expansion of their hours of coverage were among the strategies examined.

Evaluation

The potential TDM strategies were evaluated for each of the major markets. The evaluation included a description of the TDM activity, an examination of the cost, the agency or group responsible for implementation, the estimated impact of the strategy, and a discussion of the advantages and disadvantages.

The description provided a discussion of the approach and activities to be conducted. The estimated costs were provided using the low, medium, and high ranges identified previously. The agency, community, or organization most likely to be responsible for implementing the strategy was identified. In addition to existing groups, the potential for forming new organizations, such as TMOs, was raised. The impact of the strategy was determined from the microcomputer spreadsheet model and presented in the low, medium, or high range discussed earlier.

This information was presented to the PMT and discussed extensively over the course of two meetings. The PMT provided valuable insight into the advantages, disadvantages, potential barriers, and political acceptability of the different TDM strategies. The results of these discussions were summarized in the evaluation section.

SHORT-TERM PROGRAM

TDM actions that could be implemented on a short-term basis for the overall corridor and for specific markets were identified and developed into a short-term TDM program. This program focused primarily on strategies that could be implemented within existing agency budgets and did not require substantial lead time before initiation of the activity. Longer-term components of the TDM program were identified for later development into a long-term program. The focus and approach of the TDM strategies were different for the different markets, reflecting the unique characteristics of each. The general approach and the more specific TDM elements identified in the I-35W program for the different markets are summarized in the following paragraphs.

Overall TDM Actions

A series of overall TDM activities focusing on coordination, education, and information measures; promotion of existing services; and enhancement of existing traffic management techniques were recommended for the I-35W corridor as a whole. These activities included appointing a corridor manager to coordinate all aspects of the different activities going on in the corridor, establishing an interagency group to oversee implementation of the TDM program, establishing a public information program, promoting existing transit services, enforcing existing HOV bypass ramps, and expanding the Highway Helper program. The MN/DOT was identified as the lead agency for most of these activities.

Minneapolis CBD

The TDM program recommended for the Minneapolis CBD built on the existing measures and strength of the downtown market, while expanding and enhancing institutional arrangements to ensure successful implementation and private sector participation. Approximately 45 percent of the downtown workers use transit and 25 percent commute by carpool or vanpool. A high level of transit services is provided in the corridor, focusing on the downtown market, and a variety of rideshare activities has been focused downtown.

Specific elements of the TDM program for the CBD market included improving transit services and expanding employer-based transit and rideshare promotions. A major focus of the recommendations was a more active and committed role for major employers and the private sector. The formation of a TMO was recommended to provide the strength and institutional support for the TDM activities.

CBD Fringe

The CBD fringe represented the most diverse of the markets examined. The approach recommended in the program focused on specific strategies for the University of Minnesota and the other large employers in the area. The existing level of transit services and other TDM activities at these locations varies, as does the potential for improvements. Specific recommendations for the CBD fringe market included improving the special express bus network and local service to the University, examining the potential for selected transit improvements to other major employers, and promoting major employer-based rideshare and transit programs.

I-494 Corridor

The I-494 corridor has the lowest level of existing transit services and the lowest ridesharing activities. However, in terms of institutional arrangements, it provides one of the best for private sector involvement. In 1987, a TMO was formed for the I-494 corridor to examine and implement TDM strategies. In addition, five communities along the corridor have formed a Joint Powers Organization to address transportation and land use issues of mutual concern. These two organizations provide an excellent opportunity for a coordinated public and private approach to TDM activities.

The recommendations for the I-494 corridor focused on these two organizations. Specific strategies included improving reverse commute, crosstown and local circulation transit services, and employer-based transit and rideshare activities.

CONCLUSION

The methodology used in the development of the I-35W TDM program represents one approach to developing a TDM plan for a major urban transportation corridor. The process provides for a rigorous examination of the effectiveness of potential TDM strategies, the identification of the major travel markets, and the identification of the most effective strategies for each market. The microcomputer spreadsheet model developed as part of the process is an excellent tool for examining the impact of possible TDM strategies. The approach provides a relatively quick and low-cost process.

The methodology and the microcomputer spreadsheet model should be considered by other areas facing the same types of issues. The relative ease of application and the focused approach they provide may recommend them for use in other situations. TDM activities will continue to be a major focus in many metropolitan areas as one approach to dealing with increasing traffic congestion problems. TDM is not the answer to congestion problems. Rather, TDM strategies provide an additional set of tools for addressing traffic congestion. The approach to developing a TDM program outlined here may help other cities facing these problems.

ACKNOWLEDGMENTS

The work described in this paper was undertaken by the RTB. Katherine F. Turnbull was the planning manager at the RTB at the time of the development of the I-35W TDM program and was responsible for its development. Cindy Fish and David Jacobson, RTB planners, were responsible for many of the data collection activities. Assistance was also provided by the Metropolitan Council and a team of consultants: Richard H. Pratt, Richard H. Pratt, Consultant, Inc.; Richard Kuzmyak, Comsis, Inc.; and Eric Schreffler, Harold Katz and Associates.

The framework for the TDM microcomputer spreadsheet evaluation model was designed by Richard H. Pratt, who also furnished the experience-based parameters. The pivot point mode choice formulation and coefficient were provided by Gordon Schultz of Comsis, and Mark Roskin of Comsis implemented the spreadsheet model.

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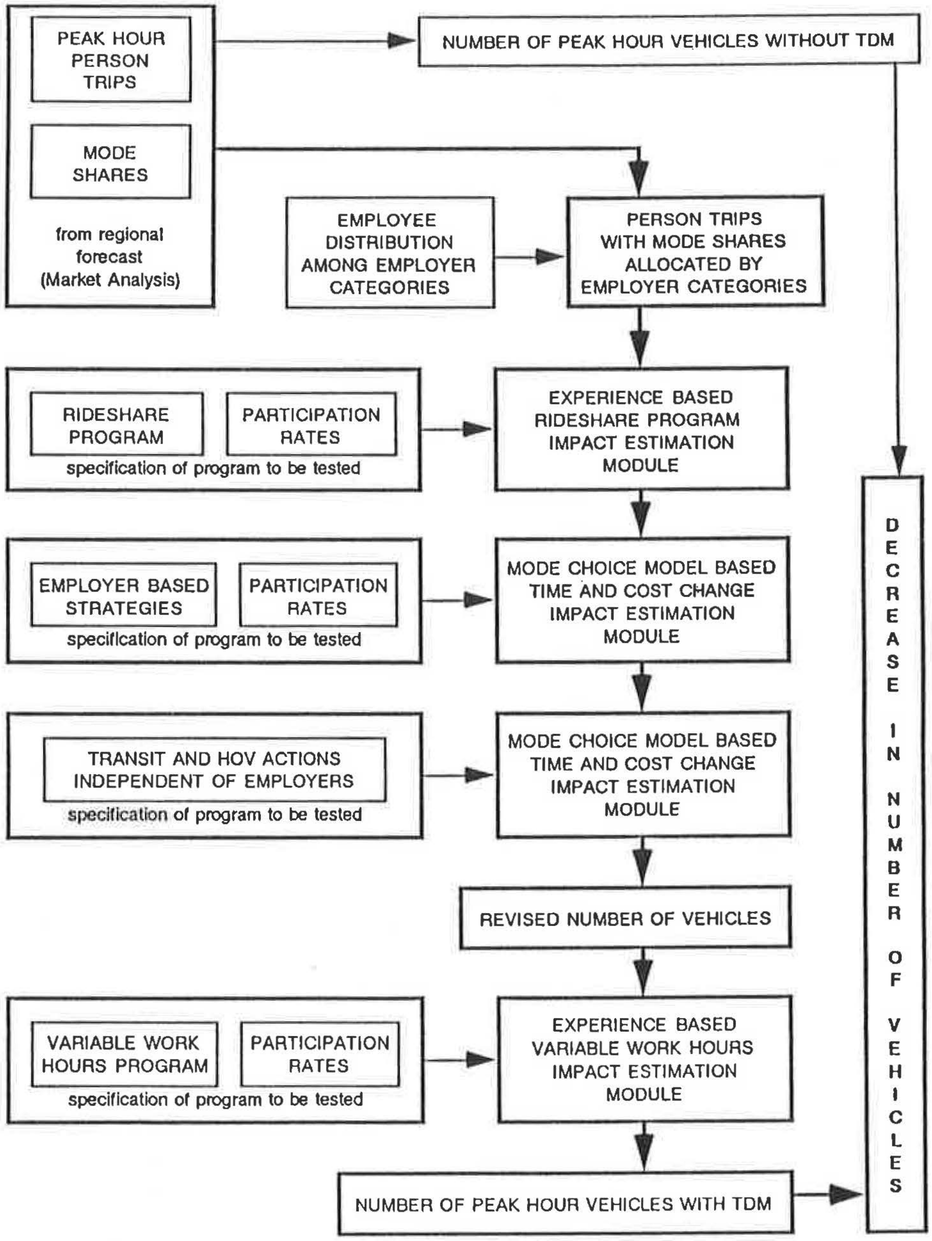

FIGURE 6 TDM spreadsheet evaluation model.

forecasts. The 2010 forecast was used, because its traffic estimates were closer to existing conditions than those of the 1980 traffic simulation.

The HBW trips were factored to represent a.m. peak-hour travel. The term "person-trips" is used in Figure 6, indicating that drive-alone trips, group-ride trips, and bus trips are all included. An initial step uses the trip data and mode share data to compute the base number of peak-hour vehicles without new TDM strategies.

The other initial step allocates the person-trips to different employer categories. Sixteen categories were used, identified as new development or present development, office or non-office, and four categories of employer size. This breakdown was made because employers in different categories exhibit different degrees of TDM participation and different employee response rates and may be affected differently by any regulations or ordinances that are applied.

The person-trip data were then successively modified by estimating the effect on mode shares of rideshare programs, employer-based strategies, and transit and HOV actions independent of employers. All programs, except the rideshare and variable work hour programs, have to be specified in terms of the time and cost savings offered or penalties imposed.

Employer participation rates are crucial to the analysis of rideshare programs and other employer-based strategies, but they are not relevant in the case of transit service improvements, HOV facilities, or actions independent of employers. The employer participation rates can be voluntary rates based on experience, or can be set to 100 percent for specific employer categories to represent mandatory participation.

The modified mode shares are applied to the person-trip data to produce a revised number of vehicles, which is then adjusted to reflect the effect of variable work hour programs on the proportion in the peak hour. The result is an estimate of the number of peak-hour vehicles that would remain when the TDM action or actions being analyzed are in force. That number is then compared with the base number to calculate the estimated decrease in peak-hour vehicle trips in the commuter market being analyzed.

The TDM spreadsheet model was used to evaluate the potential effects of each of the general TDM strategies under consideration for each of the I-35W commuter markets. In addition to examining the results for each TDM strategy, combinations of strategies were examined and evaluated.

Although the evaluation model provided results in terms of the estimated decrease in the number of peak-hour vehicle trips, a range was used when the information was presented to the PMT and others. This range provided a general indication of the magnitude of the TDM strategy and prevented focusing on one specific number. The effects of the strategies were presented in the following groupings:

- Low impact: 0- to 20-vehicle peak-period reduction,
- Medium impact: 21- to 100-vehicle peak-period reduction,
- High impact: 101- to 300-vehicle peak-period reduction, and
- Very high impact: Over 300-vehicle peak-period reduction.

A general cost estimate was provided for each strategy using the following categories:

- Low: under $500,000 annual cost,
- Medium: $500,000 to $1 million annual cost, and
- High: over $1 million annual cost.

In addition, three levels of impact were examined on the basis of participation rate and level of involvement by employers. The major differences between the categories relate to the level of private sector participation and whether participation is voluntary or required through community ordinances or other legislative action. The voluntary level assumes that the private sector is participating out of civic support or because the benefits are viewed as important. The second level assumes some requirements placed on the private sector, and the third level assumes a stronger set of requirements. Details on the different levels are as follows:

1. Voluntary
   - Ridesharing: 4 to 37 percent participation rate,
   - Transit subsidies: 1 to 7 percent participation rate, and
   - Variable hours for office only: 4 to 37 percent participation rate.
2. Partial mandatory: same as voluntary except
   - Parking management and pricing strategies: 15 percent participation rate, and
   - Ridesharing requirements for new development: 76 to 100 percent participation rate.
3. Fuller mandatory: same as voluntary except
   - Ridesharing requirements for old development: 4 to 100 percent participation rate, and
   - Parking management and pricing strategies for old development: 30 percent participation rate.

The ranges identified relate to the size of businesses. Where ranges were used, the lower percentage pertains to firms of under 50 employees, whereas the higher percentage pertains to firms of 50 employees or more.

IDENTIFICATION AND EVALUATION OF TDM STRATEGIES

On the basis of this examination of existing TDM activities, experience with the use of TDM strategies in other parts of the Twin Cities, and a review of relevant national TDM examples, a set of TDM strategies was identified for evaluation. These strategies included new TDM activities, not previously used in the Twin Cities or in the I-35W corridor, and the fine tuning or changing of existing TDM elements. The TDM strategies fell into five general categories: transit, ridesharing, parking management, variable work hours, and traffic management. The approach used with each of these elements in the modeling process is described briefly, along with their effects identified from the model. This description is followed by a more detailed discussion of the evaluation process.

Transit

Potential transit service improvements were identified in a general way for the overall corridor and for each of the major markets. The types of transit services suggested as viable options included improvements to existing services and a variety of new services. Two levels of transit improvements were examined in the modeling process: a 15 percent improvement and a 30 percent improvement. The 15 percent transit improvement assumed a 1- to 2-min decrease in the usual walk-and-wait time of a trip. Potential service improvements included additional trips on existing express routes and improved frequency on local services. The 30 percent transit improvement assumed a 2- to 4-min decrease in the usual walk-and-wait time of a trip. Potential service improvements included additional trips on express and local services, new express service, and new park-and-ride facilities. In addition, the effects of 25- and 50-cent employer subsidies were evaluated.

In general, transit improvements had a greater impact as a TDM strategy than many of the other elements. The impact varied by market, reflecting both the current level of service and the viability of potential improvements.

Ridesharing

The ridesharing strategies examined built on the existing program, which focuses on ride matching, overall marketing, and corridor-specific promotions. Two approaches were evaluated: a passive program and an active program. The major difference between the two was the role of the private sector. The active program reflected a high level of involvement and

Preliminary Evaluation of the Coastal Transportation Corridor Ordinance in Los Angeles

CHARLES BLANKSON AND MARTIN WACHS

The Coastal Transportation Corridor Ordinance attempts to regulate traffic congestion in a busy Los Angeles community by requiring new real estate developments to mitigate future trips and to contribute to a trust fund for improving traffic flow within the affected area. To conduct a preliminary evaluation of the trip reduction portion of the ordinance, a sample of eight buildings housing 117 firms was selected. Three buildings housing 44 firms were subject to the ordinance, and a control group of five buildings housing 73 firms was not affected by the ordinance. Differences in ridesharing facilities, services, and subsidies were observed, and 1,216 workers in the two groups of buildings were surveyed to determine their travel patterns. The results show that developers affected by the ordinance are significantly more likely to include preferential parking for carpoolers in their projects and some bicycle parking facilities as well. The companies affected by the ordinance offer a substantially smaller proportion of their employees free parking at work, and, among employees who pay to park, those in the buildings covered by the ordinance pay higher rates. The provision of these facilities and the combination of parking fees and other promotional efforts have had a very small initial effect on workers' decisions to drive to work alone. The proportion of workers driving to work alone is similar in the experimental and control groups. Although twice as many workers in buildings affected by the ordinance carpooled to work, they were a small fraction of the workforce. A sizable proportion of workers in the study area generally leave work outside the peak period, probably to avoid late-afternoon congestion.

American attitudes toward transportation planning have recently undergone significant change. For three decades after the end of World War II, public policy emphasized the construction of new highway and transit facilities to remove the backlog of needs resulting from the combined effects of depression, a war economy, continued urban growth, and accelerating automobile ownership. For the most part, transportation policymakers agreed that their primary goal was to accommodate growth by constructing facilities that would have adequate capacity to handle future demand. Land-use patterns and economic development were understood to be the sources of traffic, yet there was general agreement that transportation policy should aim to accommodate forecast land-use and economic growth rather than regulate them to control traffic.

Views of transportation policymakers have been changing under pressure from increasing growth and traffic congestion, growing limits on transportation budgets, and increasing opposition to highway construction by environmental coalitions and community groups. Now, policymakers frequently argue that "We can't build our way out of our problems," and that attempts to accommodate growth solely by increasing transportation system capacity impose greater costs on communities than are warranted by their benefits. In the 1970s, this shift in emphasis gave rise to transportation system management, the augmentation of capacity through low-capital-cost approaches such as traffic signal synchronization and reserved lanes for high-occupancy vehicles. In the early 1980s, transportation demand management was also emphasized, including efforts to promote ridesharing and transit use by workers through a variety of subsidy and incentive programs. In the late 1980s, this growing movement toward management rather than facility construction has emphasized changes in land-use policy and the spatial redirection of economic growth to control traffic at its source.

In Los Angeles, several regulatory programs, ballot initiatives, and municipal ordinances have been directed toward limiting traffic by controlling land use and real estate development. They have all been enacted so recently that relatively few evaluative studies have yet taken place. Tracking progress under these programs and learning from them is important, so that policymakers proposing new programs and amendments to older ones are informed by past successes and mistakes. One of the recent Los Angeles programs is evaluated in the following sections.

TRAFFIC REDUCTION IMPROVEMENT PROGRAM AND THE COASTAL TRANSPORTATION CORRIDOR ORDINANCE

In 1983, the Los Angeles City Council approved the citywide Traffic Reduction Improvement Program (TRIP). This blanket or framework ordinance enables the council, by a majority of two-thirds, henceforth to designate any community or neighborhood a "traffic impact area." When an area is so designated, a set of procedures is invoked, resulting in special land-use controls and development impact fees within the designated areas. These controls and fees are intended to mitigate the impacts of trips generated by new developments there. The designation of a traffic impact area requires the city to spend 1 year devising a transportation-specific plan for the impacted area, during which development permits may be issued only with the explicit approval of the council. When the year-long planning effort is complete, the council adopts, by separate ordinance, the transportation-specific plan devised

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during the planning period. Although the plans differ because of the specific areas to which they apply, they have many characteristics in common.

The first such plan to be enacted by the city was the Los Angeles Coastal Transportation Corridor Specific Plan Ordinance, which was passed in 1985 (*1*). This ordinance covers an area of approximately 24 mi^2, shown in Figure 1, bounded by Los Angeles International Airport on the south, the San Diego Freeway (I-405) on the east, the border of the City of Santa Monica on the north, and the Pacific Ocean on the west. The area presently has 40 million ft^2 of office, light industry, and hotel space. Plans for the area indicate that this amount of development may double in the coming 5 to 10 years. The present workforce of the area is over 100,000, and this, too, may double if developers' current plans are implemented.

The ordinance resulted from great pressure from a variety of homeowner and community groups and citizens active in opposing new development. The development community and the local city council representative responded, and many months of negotiation among these groups followed. In the end, as is often the case, homeowner groups labeled the ordinance too lenient on developers and opposed its implementation, and some developers complained that the ordinance was too restrictive.

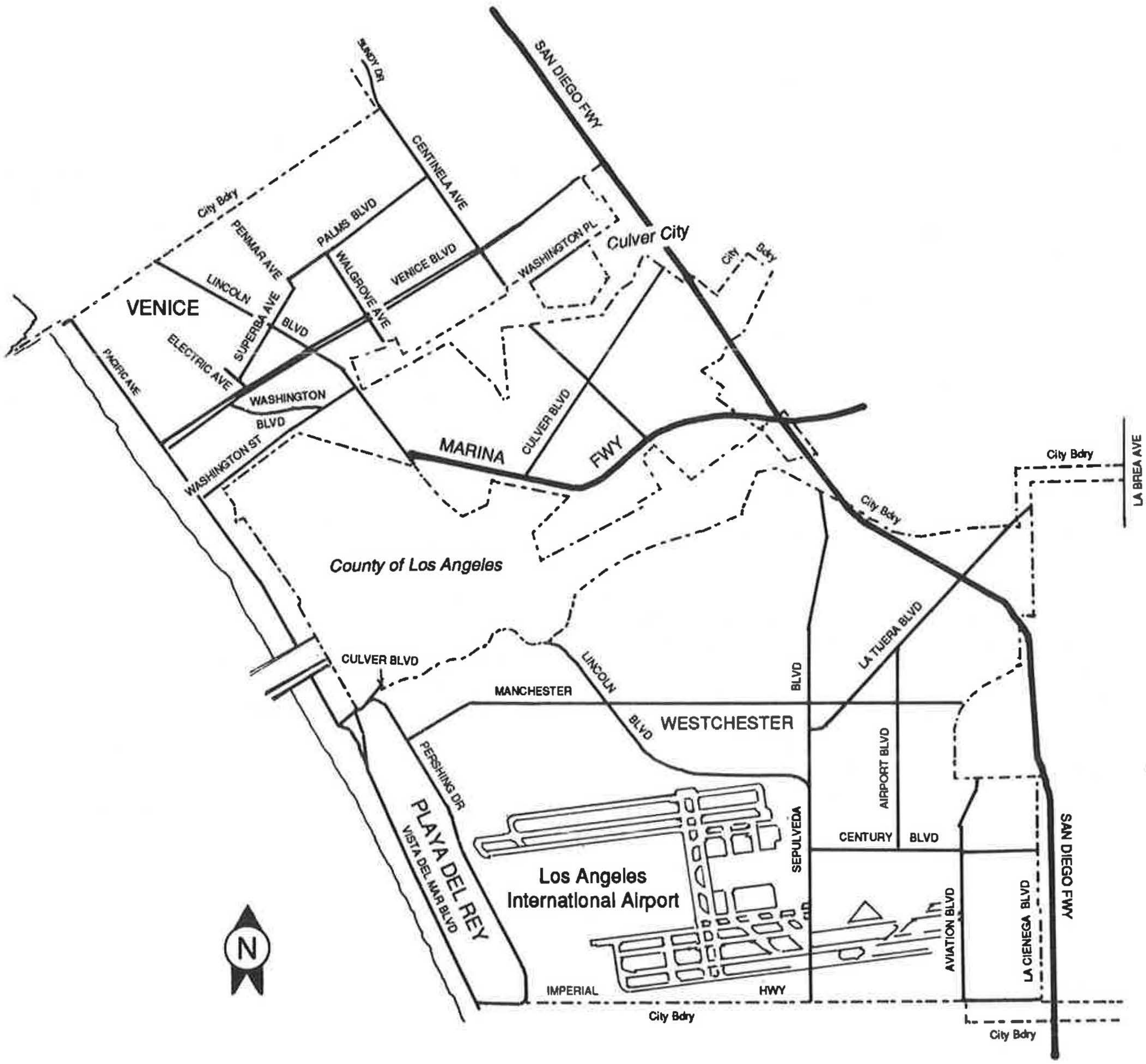

FIGURE 1 Map of the Coastal Transportation Corridor.

The concerns giving rise to the ordinance were reflections of a great deal of empirical information showing that the area's severe traffic congestion would worsen as development continued. The busiest intersection in the city of Los Angeles is at Century Boulevard and Aviation Avenue, within the study area. This intersection carried a traffic volume of 120,267 vehicles per day in 1985. The seventh busiest intersection in the city, Sepulveda Boulevard and Imperial Highway, with a daily volume of 102,770 in 1985, is also in the area affected by the ordinance. The San Diego Freeway, which forms the eastern boundary of the area, is the second most heavily traveled freeway in Los Angeles, carrying daily volumes of around 250,000 (*2*). At the time the ordinance was enacted, the growth trend in traffic was particularly alarming. Between 1973 and 1980, for example, daily trips on Lincoln Boulevard had risen by 200 percent, traffic on Sepulveda Boulevard had increased by 240 percent, and volume on the San Diego Freeway had grown by 210 percent (*3*).

MAJOR ELEMENTS OF THE ORDINANCE

The Coastal Transportation Corridor Ordinance requires that any new nonresidential development that would generate more than 100 trips in the afternoon peak hour must include measures that will reduce trip generation by at least 15 percent. The mitigation measures, which might include ridesharing programs, flexible work schedules, transit pass subsidies, or provision of bicycle facilities, are the responsibility of the real estate developer, who passes them along to the tenants through rental agreements.

Secondly, the developer must agree to pay, before construction of the project, a one-time fee based on the remaining unmitigated afternoon peak-hour trips produced by the project. The fee, which was initially set at $2,010 per afternoon peak-hour trip, is deposited in a trust fund specific to the impact area, which may be used by the city for the construction of projects included in the impact area's transportation-specific plan. Projects that are part of the plan include street widenings, installation of computerized traffic signals, construction of remote parking facilities served by shuttle buses, and extensions or expansions of public transit routes, all of which have been enumerated in the transportation-specific plan for the impact area.

A developer can propose a demand management program to reduce generated trips by more than the required 15 percent, and application can in such cases be made for a proportionate reduction in the required fees. For example, should the developer propose to reduce trips by 20 percent rather than the required 15 percent, the fee may be reduced by an amount equal to that which would be paid for 5 percent of the trips. However, if the developer accepts such a fee reduction and the trip reduction program eventually falls short of the required goal, he must later pay triple damages, in the form of a fee equal to three times what would have been paid before construction of the project.

Developers may also receive, in lieu of credit, a reduction in the impact fee assessment for any improvements they make in the regional or subregional transportation system, with approval of the Department of City Planning and the Department of Transportation of the City of Los Angeles. The ordinance also provides that large development projects must be broken into phases, with later phases being approved for construction only after earlier phases have been successful in achieving required trip mitigations.

The ordinance only applies to new development, and no fees or trip-reduction requirements apply to existing developments in the area. Furthermore, the ordinance exempts residential construction, government facilities, and neighborhood-serving commercial projects such as gasoline stations and car wash facilities, as well as religious facilities, schools, and grocery stores.

Because it is a government facility, Los Angeles International Airport is exempted from the requirements of the ordinance despite the fact that it is in the affected area. The airport occupies over 3,500 acres; with a 1986 workforce of 35,000 employees, it is the largest single employer in the study area and by far the largest trip generator. Many critics of the ordinance believe that the exemption of the airport renders the ordinance ineffectual.

Critics of the ordinance also argue that the trip generation rates published as part of the ordinance are not valid. The rates, derived from tables published by the ITE, are based on 1-day counts of facilities throughout the United States. The sample of buildings giving rise to the tables is not necessarily a random one or specifically comparable to buildings in southern California. The rates used in the ordinance do not take into consideration regional variations in trip generation, seasonal variations, or variations that might result from differences in climate or weather.

Another problem with the ordinance is its limited provisions for monitoring and enforcement. The only formal mechanism for monitoring the efforts of the developers to implement their trip-reduction programs are annual reports submitted to the city of Los Angeles by the developers themselves.

METHOD OF EVALUATING THE PROGRAM

The actions of developers responding to the imperatives of the ordinance were compared with those of a control group of similar developments nearby, which are not affected by the ordinance. Travel behavior of workers employed in buildings affected by the ordinance was compared with travel behavior of workers in the control buildings. The intent, of course, was to determine whether or not the programs provided by developers are affected by the ordinance, and whether the ordinance is having any measurable impact on employee travel choices.

Information about the programs offered by developers was gathered in direct personal interviews with the developers during 1988 and 1989. Information on travel patterns of employees working in various buildings was obtained by questionnaires distributed to employees during the summer and fall of 1988, with the cooperation of their employers and building managers. The sample, whose characteristics are presented in Table 1, included three buildings that were subject to the ordinance, which together included 44 separate firms. From among the employees of those firms, 620 completed questionnaires regarding personal characteristics and travel choices. A control group of five buildings containing 73 firms was used to obtain travel data and personal information about 596 employees.

TABLE 1 STUDY FIRMS

Building #	Respondents	# of Firms	Average # of Respondents per Firm
		A. Experimental Group	
1	121	40	3
2	266	1	266
3	233	3	78
Total:	620	44	
		B. Control Group	
4	160	30	5
5	89	20	4
6	117	21	6
7	131	1	131
8	99	1	99
Total:	596	73	
Overall Total:	1216	117	

TABLE 2 FACILITIES PROVIDED

	Reserved Parking for Ridesharers	Bicycle Racks	Lockers	Showers
Experimental				
Bldg. #1	No	No	No	No
2	Yes	Yes	No	No
3	Yes	Yes	No	No
Control				
Bldg. #4	No	No	No	No
5	No	No	No	No
6	No	No	No	No
7	Yes	No	No	No
8	No	No	No	No

In most of these instances, data on the employees were obtained directly through the questionnaires. In one case, an employer had recently completed a survey of its own and provided the survey results. Because the survey administered by the employer did not include a few of the questions on the questionnaire, the numbers of respondents differ somewhat from one question to another. The response rate varied from one firm to another, but the range of responses was between 25 and 38 percent of the employees of the eight buildings.

Chi-squared tests were done on all the findings to determine whether the differences observed between the experimental and control groups were statistically significant at the 0.05 level.

PROVISION OF FACILITIES, SERVICES, AND SUBSIDIES BY DEVELOPERS AND EMPLOYERS

Table 2 presents the facilities provided by the developers of the eight buildings in the sample. Reserved parking for ridesharers was provided in two of the three experimental buildings, whereas only one of the five control buildings offered reserved parking for ridesharers. Similarly, developers of two of the three buildings affected by the ordinance but none of the five control buildings had elected to include bicycle racks. Interestingly, none of the eight buildings included showers or lockers for bicycle commuters; developers may have regarded those facilities as unlikely to attract sufficient use to warrant

TABLE 3 PARKING SUBSIDY AT EXPERIMENTAL AND CONTROL BUILDINGS

	Experimental Group (%)	Control Group (%)
Pay to Park at Work?		
Yes	38.0	23.2
No	62.0	76.8
Amounts Paid		
<$20	10.6	70.4
$20 to $40	75.6	8.8
>$40	13.9	20.8

NOTE: For the question "Do you pay to park at work?" 324 answers were received for the experimental group and 538 for the control group. For the amounts, 123 answers were received for the experimental group and 125 for the control group.

their inclusion. Subsidized parking at worksites is common in the ordinance area. An inquiry was made to determine whether employers in buildings affected by the ordinance were providing subsidized parking for employees as frequently as employers in the control group. The results of this inquiry are presented in Table 3, which clearly indicates a substantial difference. Although 77 percent of the employees in the buildings not affected by the ordinance received free parking at work, only 62 percent of the employees in the affected buildings had their parking fully subsidized. This difference is significant, although the majority of the employees were parking free even in buildings covered by the ordinance.

Table 3 also indicates that among those paying to park at work, workers in buildings affected by the ordinance typically paid much more. Although 70 percent of the employees paying to park in the control buildings were paying less than $20.00 per month, only 11 percent of the employees in the experimental buildings paid that little, whereas three-fourths of them paid between $20.00 and $39.00 per month. Perhaps Table 3 indicates a shift toward employee-paid parking at worksites affected by traffic control ordinances such as the Coastal Transportation Corridor program.

EMPLOYEE TRAVEL PATTERNS

The Coastal Transportation Corridor Ordinance has two purposes. First, it aims to reduce automobile traffic by encouraging ridesharing, including transit use, vanpooling, carpooling, bicycling, and walking to work in buildings that come under the ordinance. Second, it seeks to upgrade traffic arteries in the impact area by charging developers fees that will be used to improve facilities in the corridor. Only the first of these questions is addressed here. By comparing the experimental population with the control group, the presence of substantial differences in their travel patterns can be estimated.

Before comparing travel patterns of the two groups, their demographic characteristics must be described in general terms. The samples in the experimental and control buildings did not differ significantly from one another in their major demographic characteristics. Of the workers in both the experimental and control buildings, 70.2 percent were in administrative and clerical positions, 20.4 percent in professional jobs, and 4.3 percent in janitorial and catering services. Nearly 70 percent of the respondents were under 40 years old, and 23 percent were between the ages of 40 and 59. The age distribution was judged to be typical of the Los Angeles commuter work force, because it is similar to the distribution of respondents to the 1988 commuter survey performed by Commuter Computer (*4*). Approximately 59 percent of the respondents were females, which was a substantially higher proportion than in the regional commuter survey, in which only 47 percent were women. Over half of the respondents earned between $20,000 and $49,999, and only about 10 percent earned less than $20,000 per year. Approximately 97 percent were employed full-time, which was defined as 5 days per week and 8 hr per day.

Because the ability to rideshare is dependent on the need for a car at work, respondents were asked whether they regularly needed a car at work. Although 68 percent said that they needed their cars as part of their work, 32 percent of these answered that they used their cars only for personal business while at work, and only 25 percent said that they used their cars at work virtually every day of the week. By contrast, 14 percent said that they typically used their cars at work only 1 day per week, and 15 percent said that 2 days per week was typical.

Table 4 indicates that the one-way distance between home and work was distributed similarly for workers in the buildings covered by the ordinance and those in the control group. In both instances, just under two-thirds of the employees traveled less than 15 mi between home and work, whereas about one-third traveled more than 15 mi. Because the work-trip lengths and demographic characteristics were similar for the

TABLE 4 PERCENTAGES OF EMPLOYEES TRAVELING VARIOUS DISTANCES FROM HOME TO WORK

	Experimental n=620	Control n=596
1 - 5 miles	23.1	24.8
6 - 15 miles	42.4	38.6
16 - 30 miles	17.1	18.3
31 miles & over	15.9	15.7
Non-response	1.6	2.7

TABLE 5 MODE SPLIT PERCENTAGES

	Experimental n=620	Control n=596
Drive alone	86.8	87.9
Public Bus	2.0	2.3
Carpool	7.4	3.5
Drop Off	2.9	3.5
Park & Pool	0.7	0.2
Motorcycle	0.2	1.3
Others	0.2	1.2

TABLE 6 TIMES OF ARRIVAL AT AND DEPARTURE FROM WORK

	Experimental Group (%)	Control Group (%)
Time of Arrival at Work		
Before 6:30 a.m.	5.0	4.4
Between 6:30 and 6:59 a.m.	6.0	6.2
Between 7:00 and 7:29 a.m.	11.6	9.6
Between 7:30 and 7:59 a.m.	21.0	13.3
Between 8:00 and 8:29 a.m.	25.5	21.1
Between 8:30 and 8:59 a.m.	23.4	31.5
Between 9:00 and 10:00 a.m.	6.0	8.4
After 10:00 a.m.	0.8	1.7
Not Regular	0.8	3.5
Time of Departure from Work		
Before 4:00 p.m.	8.6	10.1
Between 4:00 and 4:29 p.m.	7.4	7.7
Between 4:30 and 4:59 p.m.	14.1	12.6
Between 5:00 and 5:29 p.m.	28.1	20.0
Between 5:30 and 5:59 p.m.	12.8	13.8
Between 6:00 and 6:29 p.m.	16.8	24.7
After 6:30 p.m.	11.0	9.1
Nonresponding	1.3	2.2

two groups, any differences observed in travel patterns were assumed to be attributable to the program itself.

Table 5 presents a comparison of the mode choices for the journey to work between the two populations. Little difference was observed between the two samples in the proportion of workers who drive to work alone. In the buildings affected by the ordinance, more than twice the proportion of employees carpool to work, but these seem to have a small effect on the proportion driving to work alone. Only 13.2 percent of the experimental group employees did not drive alone, versus 12.1 percent of the control group employees. The ordinance has not appeared to make any substantial difference in the proportion of workers driving to work alone.

Table 6 indicates how those affected and those not affected by the ordinance differed in terms of their arrival and departure times. First, the table indicates that most workers in the study area arrive at work during the peak period. Only 11.8 percent of the experimental group employees and 12.3 percent of the control group employees arrive at work outside peak hours (i.e., before 7:00 a.m. and after 10:00 a.m.). Second, relatively larger proportions of experimental group employees (19.6 percent) and control group employees (19.2 percent) leave work outside peak hours (i.e., before 4:00 p.m. and after 6:30 p.m.).

The amount of information about alternative travel modes received by employees through their employers was of interest, because implementation of the ridesharing requirements is dependent on employee awareness of alternatives to driving alone. The results of this investigation are presented in Table 7. The table indicates that among those ridesharing to work, the majority of employees of companies in the control group had learned about their current option from a fellow employee. Although those in the experimental companies were three times as likely as those in the control group to learn about their options from their employers, those who heard about ridesharing from their employers constituted less than 3 percent of the sample. In both samples, not a single ridesharer reported having learned about opportunities for ridesharing from a ridesharing coordinator.

CONCLUSION

The results of this study are preliminary. They are based on a small sample of buildings, and the study was undertaken early in the history of implementing the Coastal Transportation Corridor Ordinance. The results thus far indicate that developers affected by the ordinance are significantly more likely to include preferential parking for carpoolers in their projects and to include some bicycle parking facilities. The buildings affected by the ordinance offer a substantially smaller proportion of their employees free parking at work, and those who pay to park pay higher rates. The provision of these facilities, and the combination of parking fees and other promotional efforts, seems to have had a very small initial effect on workers' decisions to drive to work alone. The proportion of workers driving to work alone is similar in the experimental and control groups; although twice as many workers in buildings affected by the ordinance carpooled to work, they were a small fraction of the workforce. Although most workers in

TABLE 7 SOURCE OF INFORMATION ABOUT PRESENT COMMUTE MODE BY PERCENTAGE

	Experimental n=386	Control n=596
Through Employer	2.3	0.7
Fellow Employee	6.7	2.0
Freeway Messages/Adverts	1.0	0.3
Fliers	0.5	0.5
Transportation Coordinators	0	0
Other	5.7	6.7
(Drive Alone)	83.7	89.6

the study area arrive at and leave work during the peak periods, an increasing number seem to leave work outside peak hours, perhaps to avoid the late-afternoon congestion.

In sum, promising differences in the behavior of real estate developers and employers affected by the ordinance were observed, but the differences are small. As yet, no substantial changes in travel behavior can be attributed to the ordinance, except for a tendency toward slightly higher rates of carpooling among workers at firms affected by the ordinance.

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Staggered Work Hours for Traffic Management: A Case Study

GENEVIEVE GIULIANO AND THOMAS F. GOLOB

The Staggered Work Hours Demonstration Project was conducted in downtown Honolulu during a 4-week period in 1988. During the project, official office hours for state, city, and county employees were shifted 45 min later in an attempt to alleviate the early peak-period congestion into downtown Honolulu. Approximately 3,500 of the 7,100 employees working in the Civic Center area participated in the project. This evaluation focuses on the project's effects on traffic flow, employee commuting experience, employee attitudes, work performance, and productivity. Three types of data were collected: (a) floating-car measurements of travel times and speeds on major corridors into the downtown area on two dates before and two dates during the project; (b) an employee panel survey of reported commuting experiences on the same four dates, as well as attitudes concerning project effects on activity schedules; and (c) a postproject survey of managers concerning work performance and morale. Results indicate a significant overall effect on travel conditions. Average estimated time savings were in the range of 3 to 4 min, or less than 10 percent of the average commute. However, the effects were not uniform, and nonparticipants benefited more than participants. Many participants also experienced inconveniences associated with household activities. Project results suggest that staggered work hours can improve travel conditions, but a permanent project should be as voluntary as possible to minimize problems of equity and inconvenience.

Traffic congestion has become a major public issue in U.S. metropolitan areas. Several recent opinion surveys have shown that, in a ranking of community problems, urban residents list traffic first or second (after crime). Faced with inadequate financial resources for major transportation system improvements and often with environmental constraints that preclude major improvements, public decision makers—pressured to take some action—are increasingly turning toward strategies that attempt to control or reduce congestion by managing travel demand. Travel demand management is aimed at reducing peak-period vehicle trips through strategies such as increased ridesharing and transit use, flexible work schedules, and telecommuting.

Transportation demand management (TDM) is a derivative of transportation system management (TSM). TSM was popularized in the 1970s, when transportation planners focused on increasing the efficiency or productivity of the transportation system in response to the energy crisis and air quality concerns (*1–3*). TSM includes both supply- and demand-oriented strategies, such as ramp metering, signal coordination, and provision of high-occupancy-vehicle lanes. Demand management strategies have become particularly attractive in heavily congested urban areas where the more conventional supply side or traffic engineering options have already been extensively implemented and reduction of peak vehicle trips is perceived to be the only short-term solution available.

Alternative work schedules are among the most widely implemented TDM strategies. They focus on shifting employee work schedules to eliminate or spread out peak-period work trips. Three types of alternative work schedules can be distinguished:

1. Staggered work hours—groups of employees work on fixed schedules with sequential or staggered start and end times,
2. Compressed work week—employees work full-time over a fewer number of work days, and
3. Flexible work hours—employees have some choice in establishing their work schedules.

Several studies of alternative work schedules have been conducted. Some of the studies (*4–7*) have documented the extent to which specific strategies have been implemented; others (*8–10*) have analyzed employee preferences among strategies. Simulation studies of traffic impacts associated with flexible hours have also been conducted, both separately (*11,12*) and relative to other TSM alternatives (*13–15*). Research on the impacts of alternative work schedules at home and in the workplace is more limited. Most existing research focuses on employee productivity issues, such as the feasibility of flexible or staggered shifts within different industries (*5,16*).

Actual impacts of alternative work hours programs remain unclear. Although impacts on traffic flow have been estimated, little empirical documentation is available (*17,18*). Employee attitudes toward various work schedule alternatives and the effect these alternatives may have on household schedules and activities continue to be largely unknown (*19,7*). Finally, more recent research (*10,20*) suggests that alternative work schedules may not be complementary to other TDM strategies, such as carpooling and transit use.

A 4-week demonstration project conducted in Honolulu, Hawaii, provided the opportunity to conduct an in-depth analysis of staggered work hours. The project's effects on traffic flow, employee commuting experiences, employee attitudes, and work productivity are summarized in the following paragraphs. The demonstration project and methods and data used in the analysis are discussed. Project results are followed by conclusions and policy implications. Details of the analyses are reported by Giuliano and Golob (*21*).

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HONOLULU STAGGERED WORK HOURS DEMONSTRATION PROJECT

Honolulu is an extremely congested city. A recent study (*22*) of urban freeway congestion ranked Honolulu 14th among U.S. urban areas in terms of annual vehicle-hours of delay, whereas in terms of population size it is ranked 48th. The state of Hawaii conducted the Staggered Work Hours Demonstration Project in downtown Honolulu to determine whether a large-scale shift in work hours among downtown workers could reduce traffic congestion.

The project took place during a 4-week period from February 22 through March 18, 1988. During the project, official office hours for state, city, and county employees were shifted from 7:45 a.m.–4:30 p.m. to 8:30 a.m.–5:15 p.m. The shift to a later schedule was selected because of the early peaking characteristic of downtown-bound traffic. Participation in the project was mandatory for all public employees. Nonparticipation required approval through a formal exemption process. Participation by private-sector downtown employers was encouraged but not required. Eighteen major corporations participated; certain employees of these companies were able to choose their project work schedule. Thus, for private-sector employees, participation meant changing work hours on a voluntary basis (i.e., flexible work hours); the change could be to either an earlier or later schedule, not necessarily to the designated hours of 8:30 a.m.–5:15 p.m.

The project's participation goal was 11,000 employees, or 18 percent of the total estimated downtown work force of 60,000. This goal was based on downtown traffic volume patterns, the proportion of work trips during peak hours, and the peak-hour mode split. An 18 percent participation rate was expected to result in significantly improved peak-period traffic conditions.

METHODS AND DATA

This evaluation of the project focuses on the project's impact on (a) travel and traffic conditions and (b) employees and the workplace. The purpose of the project was to alleviate traffic congestion; thus, the extent to which this objective was realized is of primary interest. However, overall project effectiveness depended on the response and attitudes of employees and managers to the shift in work hours.

Traffic Impact Measurement

Traffic impact measurement requires controlling for seasonal variability as well as day-to-day differences. Seasonal variability was controlled by selecting the month immediately preceding the project as the basis of comparison, minimizing potential differences caused by holidays and tourism patterns. Day-to-day differences were addressed by conducting floating-car observations of travel times and speeds along the three major directional corridors leading to downtown. Trips were made along an identical route, with one car commencing every 15 min, and recording actual times at a series of checkpoints along the route. The floating-car data were used to measure changes in peak travel conditions along the route. Information on the three routes is presented in Table 1. Floating-car observations were conducted on all routes on February 3 and 17 (before the project) and March 2 and 16 (during the project). Additional observations were conducted after the project on March 30 for Routes 1 and 3.

Impacts on Employees

A variety of issues concerning employee behavior must be examined to properly evaluate project impacts, including extent of participation in the project, worktrip travel characteristics, impacts on household activities, and attitudes toward the project.

A panel survey of employees was conducted to obtain information on these issues. This type of survey gathers information from respondents at more than one point in time. It is the most effective method for obtaining longitudinal data (*23*). In this case, accurate reporting of travel experiences was critical because it was likely that (a) travel time differences because of the project might be small and therefore difficult both to perceive retrospectively and to statistically measure and (b) employee attitudes toward the project could affect retrospective reporting. It was also important to be able to observe any changes in attitudes over the course of the project.

The panel had four waves, each coinciding with the floating-car observation days. All four waves contained identical questions concerning commuting experiences on the survey day (e.g., arrival and departure times, mode, and stops before and after work). The panel design thus permitted multiple "before" and "during" comparisons for each individual's commute trip. The first wave also elicited background information on demographic, socioeconomic, and residential location characteristics. In addition, the last wave included questions about attitudes and perceptions of the project.

Respondents were selected on a uniform 20 percent, or 1 in 5, basis both from the public sector and from private-sector companies that had elected to participate in the project. Surveys were distributed and collected at the worksite. The survey response rate was high; all four waves were completed in 69 percent of the 2,297 surveys distributed.

It was expected that implementation of staggered work hours would affect working conditions and productivity, as well as employee attitudes, tardiness and absenteeism, and overall work performance. Therefore, information on workplace effects was gathered through a random survey of public and private management personnel. The mail-back survey was distributed immediately after the close of the project. A total of 371 surveys was distributed, from which 281 valid responses were received.

PROJECT RESULTS

Project impacts are discussed in six general categories: (a) project participation, (b) travel conditions and commuting experiences, (c) perceptions of traffic conditions, (d) perceived project impacts, (e) workplace impacts, and (f) attitudes toward the project.

TABLE 1 FLOATING-CAR ROUTES

Route	Description	Residential Area	Starting Point	Ending Point	Length in Miles	Peak Period
1. Mililani	Mililani via Kamehameha Hwy. H-1 Freeway, Moanalua Freeway	Leeward	Kamehameha Hwy. at Kuahelani Ave.	Vineyard Blvd. off-ramp	15.1	5:15-8:15 a.m.
2. Hawaii Kai	Hawaii Kai via Kalanianaole Hwy., H-1 Fwy.	East Honolulu	Kalanianaole Hwy. at Keahole St.	Ward Ave. overpass	9.3	6:00-9:00 a.m.
3. Kailua	Kailua via Pali Hwy.	Windward	Kalanianaole Hwy. at Castle Hospital	Off-ramp to Punchbowl and H-1 Fwy.	9.1	5:30-8:30 a.m.

Project Participation

For the purpose of analysis, project participation was defined as working the prescribed schedule of 8:30 a.m.–5:15 p.m. Although participation was mandatory for public employees, exemption was possible if personal hardship could be demonstrated (for example, childcare or carpool arrangements). Employee survey data showed that about half of the eligible public employees actually changed their hours to the later schedule. Private-sector participation was voluntary and permitted changes to both earlier and later schedules. Just over 8 percent of the private employees surveyed changed to the later schedule, and 11 percent switched to an earlier schedule. Table 2 presents the changes in work hours for four groups of commuters: participants, nonparticipants (did not change work hours), early changers (changed to a schedule at least a half-hour earlier than usual), and late changers (changed to a schedule at least a half-hour later than usual, but not 8:30 a.m.–5:15 p.m.). (The source of these various schedule changes is unknown.) The remainder of the sample showed no consistent pattern over the four waves. On the basis of the participation rates, it is estimated that approximately 3,500 of the 7,100 public employees working in the Civic Center area participated in the project, along with about 500 private-sector participants, giving a total of 4,000 participants, representing 6 to 7 percent of the downtown workforce.

Participation also varied by area of residence. The highest participation rates occurred among workers living close to downtown and the lowest among workers living farthest from downtown in the windward area (northern edge of Oahu) and the East Leeward area (northeast suburbs) (see Figure 1). This pattern reduced the potential impact of the project on traffic conditions because short trips were overrepresented.

Characteristics of Participants and Nonparticipants

The demographic and socioeconomic characteristics of participants and nonparticipants were also examined. As expected, nonparticipants differed from participants in terms of characteristics that made participation more difficult. Nonparticipants had more children, used childcare services, and tended to be younger and female. Participants were more likely to be in professional or technical occupations and from households with fewer workers. Participants were also more likely to be car drivers, whereas nonparticipants were more frequently carpoolers or bus users (see Table 3). Differences in these five characteristics are statistically significant at the $p = 0.05$ level.

In the fourth wave survey, nonparticipants were asked to explain why they did not participate. The most frequently cited reason among both sectors was "My regular work sched-

TABLE 2 WORK HOUR CHANGES BY SECTOR

GROUP	PUBLIC		PRIVATE	
	Number	%	Number	%
1-Participants	610	49.6	74	8.4
2-Non Participants	489	39.7	552	62.7
3-Early Changers	10	.8	97	11.0
4-Late Changers	72	5.9	23	2.6
Varying Hours	49	4.0	134	15.2

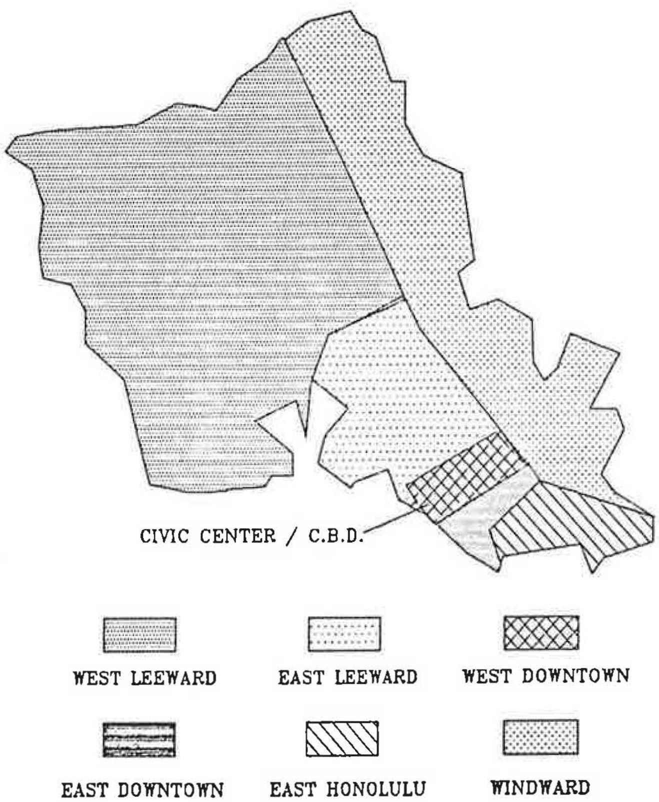

FIGURE 1 Residential areas on the basis of zip code clusters.

ule is more convenient." However, other reasons cited by public-sector employees with comparable frequency were clearly related to the mandatory nature of the project. These include ridesharing, childcare, and children's school arrangements, as well as other obligations before or after work (see Table 4). These results suggest that project participation was more difficult (and frequently impossible) for workers with time pressure or schedule constraints.

Participation Impacts

Patterns of project participation had clear (and somewhat unanticipated) impacts on the worksite. First, the project resulted in a significant concentration of arrivals and departures at public worksites. For example, Figure 2 shows arrival times of state employees by 15-min intervals for each survey wave (February 3 and 17 and March 2 and 16). Although the official nonproject start-work time is 7:30 a.m., roughly 30 percent of the arrivals occurred earlier than 7:15 a.m. before the demonstration project began. Arrivals during the project were far more concentrated around the 8:30 a.m. start time. These results suggest that many employees regularly arrived at work early, probably to avoid traffic congestion or because of schedule constraints of other household members. This concentration of arrivals (and departures) resulted in localized congestion problems at some sites. The project had much less impact on arrival times at private-sector worksites, as shown in Figure 3; an increase in later arrival times resulted in a more even distribution of arrivals during the project.

Second, participation in the project required substantial changes in work schedules for many public-sector employees. Forty percent of public-sector participants shifted from work schedules starting at 7:30 a.m. or earlier, and about 10 percent of participating city and county employees switched from start-work times of 7:00 a.m. Many city and county offices had ongoing flexible hours programs, and these programs were suspended during the demonstration project. Private-sector participants working the 8:30 a.m.–5:15 p.m. schedule during the project switched from previous 7:45 or 8:00 a.m. start times. Thus, the magnitude of the change required for participation was significantly greater for public-sector employees.

Third, some public-sector participants had the added problem of finding a parking space. Most public employee parking is provided on a first-come, first-served basis. At sites where parking is less convenient or available, participants found the most convenient parking already taken. Other participants were unable to use their regular express bus service because express service stops operation on most routes after 5:00 p.m.

Finally, survey results indicated that the project had no significant impact on mode split.

Travel Conditions and Commuting Experience

Project impacts on transportation system performance were measured in two ways: (a) analysis of floating-car data and (b) analysis of reported travel times of downtown commuters.

TABLE 3 SELECTED CHARACTERISTICS OF PARTICIPANTS AND NONPARTICIPANTS

	Participants	Non Participants
1. Children In Household		
One or more younger than 6 years	13.1%	23.0%
One or more 6-18 years	31.9%	38.4%
2. Childcare		
Use childcare services outside own home	17.7%	28.0%
3. Age of Respondent		
16-24	2.9%	4.6%
25-34	20.9%	27.9%
35-44	40.8%	36.6%
45-54	22.6%	20.1%
≥55	12.7%	10.8%
4. Sex of Respondent		
Male	42.0%	33.0%
Female	58.0%	67.0%
5. Mode to Work		
Car Driver	63.8%	55.7%
Car Passenger	17.4%	21.6%
Bus	14.5%	19.9%

TABLE 4 REASONS FOR NONPARTICIPATION CITED BY PUBLIC-SECTOR NONPARTICIPANTS

	Reason	Percent of Nonparticipants
1.	My work commitments did not allow it.	13.1
2.	My regular work schedule is more convenient.	32.3
3.	I must share a ride with others.	29.2
4.	I could not adjust my child-care arrangements.	25.8
5.	I must take children to/from school.	26.5
6.	I have other obligations before/after work.	25.4
7.	I could not have taken my regular bus.	5.3
8.	I had other problems with bus schedules.	7.5
9.	Other (please specify).	17.8

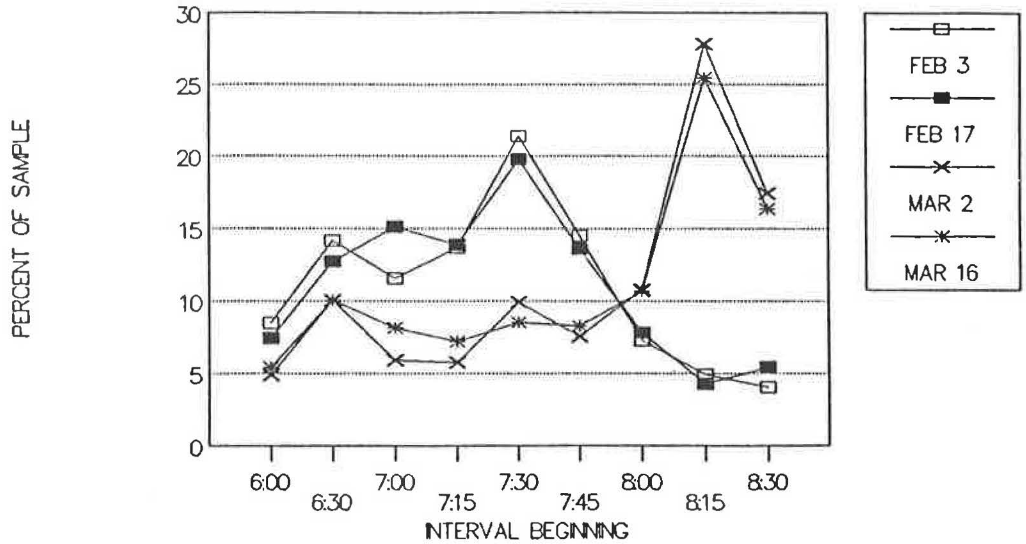

FIGURE 2 Arrival time at work, state employees.

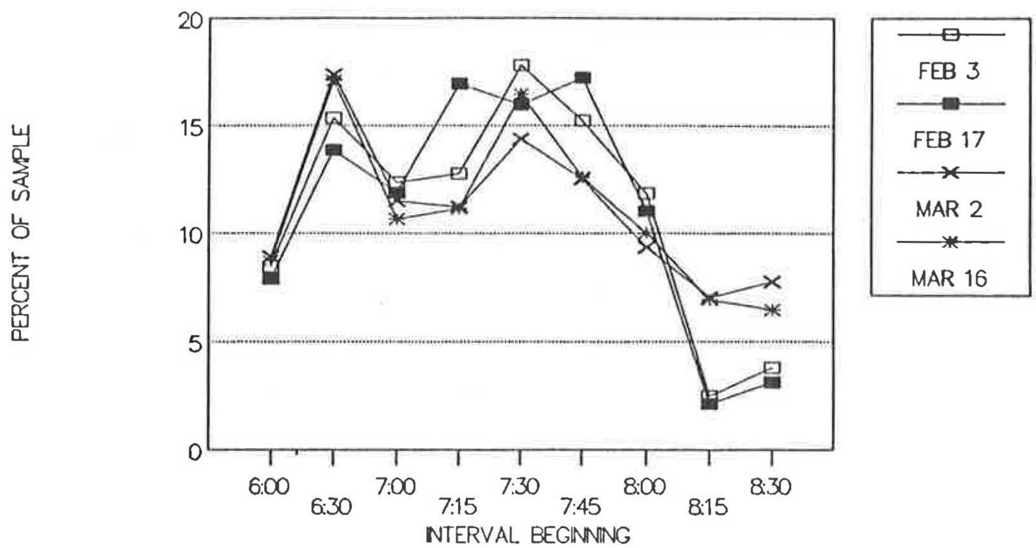

FIGURE 3 Arrival time at work, private-sector employees.

Floating-Car Results

Analysis of floating-car data from three different routes showed that statistically significant changes in peak traffic patterns occurred during the project. Table 5 presents travel time savings due to the project for all the routes. The time savings were estimated by comparing various combinations of travel time differences between the five different survey days. The analysis for Route 1 (Mililani) showed possible time savings attributable to the project of 5 to 13 percent (2 to 7 min) between 5:30 and 7:15 a.m. No systematic time differences were observed for departures after 7:30 a.m. Comparisons of average travel times before and during the project on Route 2 (Hawaii Kai) yielded travel time savings estimates for the period 6:45–7:30 a.m. of comparable relative magnitude to that of Route 1—about 9 to 12 percent. However, because Route 2 is much shorter than Route 1, the time savings estimate is smaller in absolute terms—3 to 4 min. For later travel times, possible savings declined until the 8:00–8:15 a.m. interval, where they became negative; additional travel time of about 2 min is attributable to the project in this time interval. Estimates of project impacts on Route 3 (Kailua) are similar to those of Route 2. Possible time savings are generally positive between 6:00 and 7:45 a.m., and negative thereafter. Possible savings range from 7 to 18 percent; possible losses range from 0 to 10 percent.

In the two cases for which sufficient data were available (Routes 2 and 3), time savings in earlier time intervals were found to be somewhat offset by travel time losses in later time intervals. However, in each case, the magnitude of the loss is not as great as the magnitude of the savings. These results suggest a spreading out of the peak and indicate that this spreading out can lead to travel time losses in some time intervals, even though the net effect of the change is positive.

Interestingly, the magnitude of these results is consistent with that of previous simulation studies (*24,11*).

Reported Travel Times

Reported travel times were analyzed for the four employee segments defined in Table 2. The analysis was restricted to automobile commuters and was conducted by comparing matched pairs of responses from the same individual for the four possible before/during combinations (Wave 1 versus Wave 3; Wave 2 versus Wave 3; Wave 1 versus Wave 4; Wave 2 versus Wave 4). Matched-pair comparison controls were used for individual differences in route choice, driving behavior, etc. The results of this analysis were largely consistent with those from the floating-car data.

Project participants experienced mixed travel conditions that varied by residential area. In particular, participants who had shifted from start times earlier than 7:30 a.m. experienced no significant savings or losses on the workbound trip. For some residential areas (notably the Windward and East Leeward areas), such participants experienced significant average possible travel time losses of 8 to 15 min (see Table 6). Participants who had shifted from a 7:30 a.m. or later preproject starting time experienced travel time savings, but such savings were statistically insignificant for most residential areas (*21*).

Nonparticipants realized average workbound travel time savings ranging from 2 min to almost 7 min (9 to 19 percent)

TABLE 5 TRAVEL TIME SAVINGS BECAUSE OF PROJECT, BY ROUTE

ROUTE	DEPARTURE TIME	MINUTES	PERCENT
1-Mililani	5:30 to 7:30 a.m.	2 to 7	5 to 13%
2-Hawaii Kai	6:45 to 7:30 a.m.	3 to 4	9 to 12%
	7:45 to 8:15 a.m.	0 to -2	0 to -9%
3- Kailua	6:00 to 7:45 a.m.	0 to 6	0 to 18%
	7:45 to 8:15 a.m.	0 to -2	0 to -10%

TABLE 6 MEAN WORKBOUND TRAVEL TIME DIFFERENCES FOR PARTICIPANTS, BY SHIFTING ARRIVAL TIME INTERVALS

Sample	Residential Area	(3) Mar. 2 vs. (1) Feb. 3	(4) Mar. 16 vs. (1) Feb. 3	(3) Mar. 2 vs. (2) Feb. 17	(4) Mar.16 vs. (2) Feb.17
1. 7:30-7:45 a.m. to 8:15-8:30 a.m.	East Honolulu	(NS)	(NS)	(NS)	(NS)
2. Pre-7:30 a.m. to 8:15-8:30 a.m.	East Honolulu	+16.0 Min. (+64.6%)	(NS)	+17.0 Min. (69.2%)	(NS)

TABLE 6 (*continued on next page*)

TABLE 6 *(continued)*

1. 7:30-7:45 a.m.					
to	Windward	(NS)	(NS)	(NS)	(NS)
8:15-8:30 a.m.					
2. Pre-7:30 a.m.					
to	Windward	+10.4 Min.	+11.2 Min.	+9.3 Min.	+8.3 Min.
8:15-8:30 a.m.		(+30.2%)	(+37.5%)	(+25.0%)	(+24.9%)
1. 7:30-7:45 a.m.	East	-7.4 Min.			
to	Leeward		(NS)	(NS)	(NS)
8:15-8:30 a.m.		(-18.9%)			
2. Pre-7:30 a.m.	East	+8.3 Min.	+9.5 Min.	+9.7 Min.	+15.0 Min.
to	Leeward				
8:15-8:30 a.m.		(+28.0%)	(+43.4%)	(+28.9%)	(+65.2%)
1. 7:30-7:45 a.m.	West	-15.5 Min.	-15.2 Min.	-9.1 Min.	-9.3 Min.
to	Leeward				
8:15-8:30 a.m.		(-25.4%)	(-23.5%)	(-15.0%)	(-14.6%)
2. Pre-7:30 a.m.	West	+10.6 Min.	+10.0 Min.		
to	Leeward			(NS)	(NS)
8:15-8:30 a.m.		(+25.9%)	(+24.7%)		
1. 7:30-7:45 a.m.	West & East				
to	Downtown	(NS)	(NS)	(NS)	(NS)
8:15-8:30 a.m.					
2. Pre-7:30 a.m.	West & East				
to	Downtown	(NS)	(NS)	(NS)	(NS)
8:15-8:30 a.m.					

NOTES: Differences in terms of project period minus preproject period.
NS= differences not statistically significant.

for those arriving at work between 7:30 and 8:15 a.m. However, nonparticipants who retained either earlier or later schedules experienced no significant changes in travel conditions. Finally, the workers who shifted from 7:30 a.m. to an earlier schedule (mainly private-sector employees) experienced mean travel time savings of 4 to 8 min.

For homebound (afternoon) trips, there was no change in the mean travel times for project participants. However, nonparticipants experienced a mean travel time savings similar to that of the workbound trip (about 10 percent) (*21*). This distribution of gains and losses between participants and nonparticipants affected attitudes toward the project, as further discussed in the following section.

Perceptions of Traffic Conditions

The perceptions of traffic conditions on the part of downtown employees are also important in evaluating the potential of staggered work hours. Employees were asked to express qualitative comparisons of traffic conditions during and before the demonstration project. They were asked about their trips both to and from work in terms of a 5-point scale ranging from much worse to much better.

Participants and nonparticipants had significantly different perceptions of differences in traffic conditions. Statistical analysis showed that nonparticipants were more likely to perceive traffic conditions on the trip to work as better or much better during the project, while perceptions of participants were more balanced between positive and negative perceptions.

Differences in the perceptions of traffic conditions on the trip from work to home were stronger between participants and nonparticipants. Nonparticipants' perceptions were skewed toward the positive side of the scale, whereas participants' perceptions were skewed toward negative responses (see Figure 4).

These perceptions are consistent with the reported travel time changes discussed in the preceding section. Nonparticipants likely enjoyed better travel conditions as participants shifted out of the "peak of the peak" travel intervals. For participants, travel impacts depended on their previous schedule. Those who shifted from the 7:45 a.m. start time were likely to have realized some benefit, whereas those who shifted from earlier start times, and thus did not previously travel at the height of the peak, did not realize any travel time gains. In addition, those from specific residential areas (Windward and East Leeward) who shifted from start times earlier than 7:45 a.m. realized significant travel time losses.

There were also differences among the perceptions both of participants and nonparticipants in the public and private sectors (*21*). Private-sector participants were more likely than public-sector participants to perceive better traffic conditions during the project for the trip both to work and from work. Among nonparticipants, private-sector perceptions were also more positive than those of public-sector participants.

Differences in perceptions between private- and public-sector participants and nonparticipants are not surprising. The voluntary nature of the project for private employees enabled them to optimize their work schedule. Thus, the individuals who changed hours were those who could benefit from the change. Those who could not benefit had no incentive to change and were not required to do so. The increased opportunity to choose one's work schedule probably added a positive subjective element to private-sector employee perceptions.

Perceptions of the two remaining groups of employees—those who changed to earlier hours (early changers) and those who changed to later hours (late changers)—were also consistent with reported travel time data (*21*). The early changers perceived better traffic conditions for both the workbound and homebound trips. They shifted to a less congested part of the peak and thus realized perceptible travel time savings. For the late changers, perceptions were balanced; in other words, there was no perceived change in traffic conditions during the project.

Perceived Project Impacts

The project changed circumstances at work both for participants and nonparticipants. It was anticipated that the change

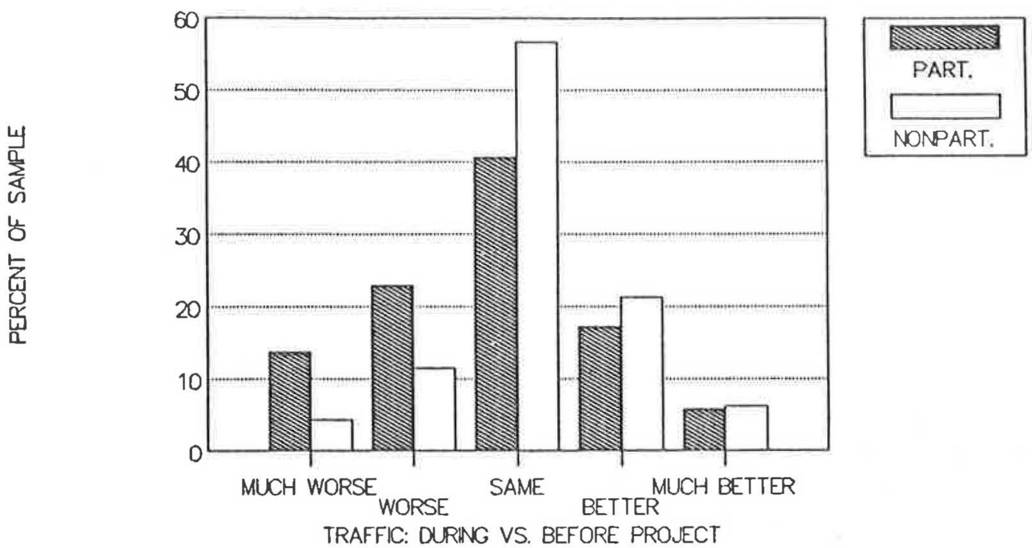

FIGURE 4 Perceptions of homebound traffic, all sectors.

in work schedules would affect employees' work performance, as well as participants' household activities and responsibilities. Of particular concern were family and childcare-related activities, given the large number of two-worker households in Hawaii.

Project participants reported problems in several aspects of nonwork activities, including taking care of personal business (63 percent), scheduling social activities (56 percent), and doing things with household members (56 percent). Arranging for childcare and children's school activities was also identified as a problem. These results are not surprising, given the temporary nature of the project, and do not necessarily suggest that such problems would be experienced if these changes were of longer duration.

Project participants also reported problems in getting to and from work. These problems were related either to finding a parking space or to using peak-only express bus service, as discussed previously.

Participants reported no significant problems with work-related activities, such as getting work done or meeting with clients or coworkers.

Employee attitudes were also analyzed by sector. Public-sector participants had significantly more negative attitudes about other activities during the project than did private-sector participants. That is, although participants in all sectors reported worse conditions in performing household, social, and work activities, state, city, and county participants were more likely to report much worse conditions. Because private-sector participants worked the same schedule as public-sector participants, the differences in attitudes between the two groups merit further explanation. It is possible that the mandatory nature of the project for public-sector employees made it necessary for many employees to work on the project schedule even though it was inconvenient or difficult for them to do so. In contrast, private-sector participants chose the new schedule willingly, probably only when it was convenient for them to do so. Public-sector participants also experienced a more extreme shift in work hours than did private-sector participants and thus potentially had more adjustments to make in nonwork activities. Finally, the more negative attitude of public-sector employees may reflect underlying discontent with the mandatory nature of the project.

Work Performance and Productivity

It was also anticipated that the project would affect job performance. The disruptive effects of changing employee work schedules, potential morale problems, and changing government office hours could pose problems for management. When asked to rate their employees' overall performance during the project, private-sector managers reported no change from usual conditions. Public-sector managers were more likely to report the same or worse conditions, with city-county responses significantly more negative than state responses.

Differences in ratings of employee morale were even more striking. Virtually all private-sector managers reported the same or a better level of employee morale during the project. State and city-county responses were just the opposite, as shown in Figure 5. Almost half of the city-county managers reported worse or much worse employee morale during the project, whereas 37 percent of state managers reported worse or much worse employee morale.

Analysis of specific aspects of work activities, including managing, communications, scheduling, and making contacts, revealed that few of these were affected by the project. A large majority (80 percent or more) of managers reported no change in work performance during the project. However, a general pattern of more negative than positive responses was evident, with public-sector management more likely than private-sector managers to report negative experiences. Most frequently identified management problems included coordinating interdepartmental work (21 percent), making contacts with mainland offices (21 percent), scheduling work assignments (17 percent), and communicating with employees (16 percent). The results indicate that inter- and intraorganization coordination was affected by the project. Public-sector responses were significantly more negative about communicating with other offices outside downtown, suggesting that the shift in work hours within downtown offices caused

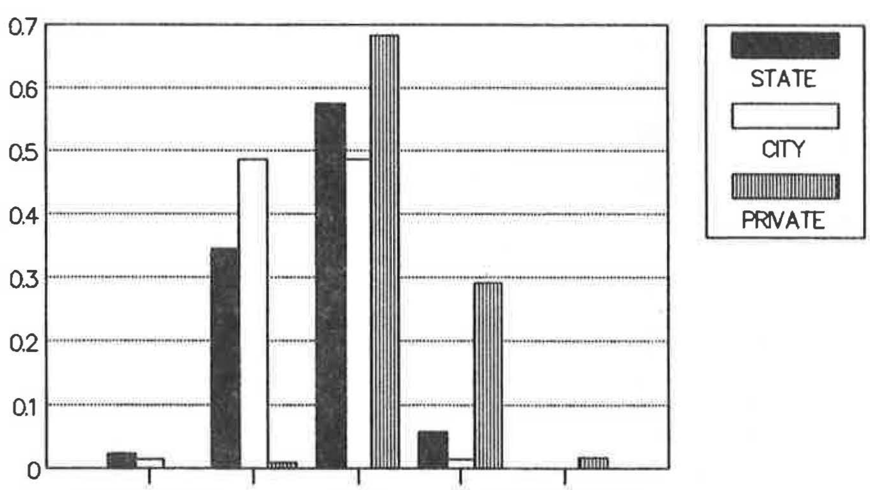

FIGURE 5 Perceptions of project influence on employee morale by sector.

some difficulties in maintaining activities that required linkages with units in locations that did not shift hours. Problems associated with contacting mainland offices were expected because the shift in business hours increased the time difference in business hours between Hawaii and the mainland.

Attitudes Toward Staggered Work Hours

Any consideration of implementing a permanent staggered work hours program requires an assessment of employee attitudes toward such a proposal. Managers were asked to rate their employees' attitudes toward the project before, during, and after its implementation. Private-sector responses were heavily skewed toward positive ratings, whereas public-sector responses were heavily skewed toward negative ratings. State ratings were significantly more negative than city-county ratings before the project. After the project, private-sector responses remained positive, state responses became significantly less negative (but not positive), and city-county responses remained negative. The shift in state employee attitudes is shown in Figure 6. These results conflict with other studies of employee attitudes toward staggered hours programs. Previous research indicates consistently positive assessments of such programs (*25*).

Both managers and employees were also asked their opinion about various staggered work hours alternatives that might be implemented in the future. All sectors and participants, as well as nonparticipants, were opposed to mandatory alternatives of staggered work hours. Attitudes toward voluntary alternatives were more mixed. Management responses to alternatives that allow employees to work on different schedules were bimodal—less than 10 percent were neutral. Private-sector managers were most likely to respond positively; state and city-county managers were more often negative, with city-county responses most negative. Voluntary staggered work hours alternatives were perceived positively by employee participants and nonparticipants. State and private participants were more positive than city-county participants about voluntary programs.

Evaluation of the project and possible future staggered work hours alternatives showed strong negative feelings toward mandatory programs. Public-sector employees were particularly opposed to any future mandatory program and had generally negative views of the project. These negative attitudes are only partially explained by the experiences of project participants because nonparticipants were also opposed to mandatory alternatives. In contrast, private-sector employees reported favorable attitudes to the project and had no strong feelings either for or against possible future mandatory programs.

These differences in attitudes between private- and public-sector employees reflect their differing experiences. For private-sector employees, the project was a voluntary program. They benefited from having the choice of changing their schedules in ways most favorable to their own particular circumstances. In contrast, public-sector employees were faced with a mandatory change in schedule that in many cases entailed a shift of an hour or more. Such shifts are bound to be disruptive, at least in the short run. Moreover, the difficulties of the work schedule shift were compounded for some participants by a longer, more congested commute.

These results indicate that mandatory changes in work schedules are strongly opposed by employees and their managers. Voluntary staggered work hours programs give more flexibility to employees and, understandably, are supported by them. This flexibility creates additional complexity for management in scheduling and coordinating work, however, and the management response to voluntary alternatives is therefore more mixed. The results also suggest that the negative reaction to mandatory programs may go beyond the problems and inconveniences generated by the project and may reflect more fundamental dissatisfaction with the project and its implementation.

CONCLUSIONS AND POLICY IMPLICATIONS

The Staggered Work Hours Demonstration Project is typical of many efforts either made or proposed to solve traffic prob-

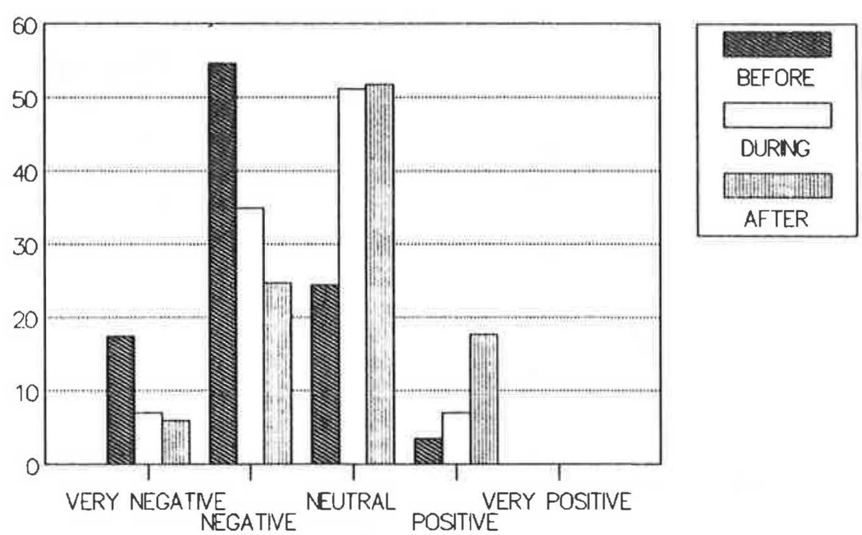

FIGURE 6 Attitudes of state managers toward project.

lems. Travel conditions were affected, but problems were encountered in doing so. In this case, the problems stem from (a) the magnitude of travel impacts attributable to the project, (b) the distribution of these impacts, and (c) the mandatory nature of the project.

The project had a significant positive overall effect on traffic conditions. Average estimated travel time savings were in the range of 3 to 4 min, or 7 to 9 percent of the average 45-min commute. There is some question as to whether a change of this magnitude is either meaningful or perceptible to most commuters. An average of 3 to 4 min means that some commuters saved more time than 3 to 4 min and others saved less. It is possible that the change was not noticeable for many commuters. Thus, the benefit of the project may for the most part have gone unnoticed.

The distribution of travel time impacts among the various employee segments and among geographic areas was also a source of problems. Nonparticipants generally benefited more than participants—and without the inconvenience of having to change their work schedule. Participants who shifted from earlier hours (i.e., those who had been taking advantage of preexisting flexible hours programs) incurred the greatest disruption in their schedule and were most likely to experience deteriorated travel conditions. Moreover, these participants were more likely to have the longest commutes. Thus, a significant minority of participants incurred particularly large costs as a result of the project. In contrast, private-sector employees, who were able to choose their work schedule, were more likely to have benefited from the project because they were able to adjust their commute to avoid the worst traffic periods.

These problems became more onerous for public employees because of the mandatory nature of the project. Employees were faced with a work schedule change over which they had little control. Although an extensive public relations program was conducted, employee resistance was not substantially reduced. The exemption process may have added to the negative reaction. Exemption required a formal application process. Guidelines for granting exemptions were uniform in the state and city, but design and implementation of exemption procedures were left to individual departments. Although the vast majority of those who applied for exemptions got them, survey results indicated that there were differences in the way exemption policies were applied among offices (*21*).

Unintended Consequences

The project also had impacts that were not anticipated. These included (a) effectively penalizing employees who had flexible work hours before the project, (b) localized congestion, and (c) parking and bus use problems.

As discussed previously, participants who switched from arrival times of before 7:30 a.m. incurred longer commutes as a result of the project and were most negatively affected by the travel shifts that took place. The research showed that flexible hours exist (de facto) in the public sector. Although the majority of employees start work at 7:45 a.m., about 40 percent of all public-sector employees start at other times. Arrival times at work are even more spread out, but the project had the effect of concentrating both arrival and departure times. This concentration led to localized congestion problems at major employment site access points and parking lots. Some public-sector participants had the added problem of finding a parking space, while others were unable to use express bus service. These impacts indicate that nonparticipants were more likely to benefit from the project than participants.

Lessons Learned

The project evaluation provides valuable insights for future traffic management policy. Results indicate that, given a choice, employees prefer earlier rather than later work schedules. This preference is shown by the pattern of schedule changes that occurred in the private sector, as well as by the pattern of arrival times among public-sector employees before the project. Preferences for earlier schedules have been documented previously (*7*). Thus, a voluntary program would likely result in few shifts to a later schedule and could have the effect of simply shifting the peak rather than spreading out the peak. Additional incentives (for example, a wage differential for the late shift) would probably be necessary to achieve a more even distribution of traffic. Factors that effectively penalize late arrivals—such as first-come, first-served parking—would have to be eliminated. Modified staggered hours programs are also a possibility. For example, employees could sign up for available schedule alternatives, much like driver schedules are allocated in the transit industry.

A more serious consideration for such programs is latent demand. It is possible that the benefits of any alternative work hours program would soon be eroded by new trips or by shifts of existing trips to the peak period. Latent demand may be particularly significant in heavily congested areas, where capacity constraints limit peak-period trips. Although latent demand is an issue that applies to any transportation improvement, it is more relevant in this case in view of the magnitude of the travel time savings identified.

A related issue is that of balancing costs and benefits. It is important to determine whether the costs incurred by participants are justified by the resulting transportation system performance improvements. Clearly, a mandatory program is inappropriate.

Finally, this research provided valuable information on the degree to which an individual's work schedule is embedded within the household activity schedule. When the work schedule changes, it affects all members of the household and requires adjustments in other activities. Social activities, childcare, children's activities, and household chores may need to be reorganized and rescheduled. The project also illustrated the dependence of workers on the schedule of other institutions and services. Thus, spreading out the normal work day is dependent on extending hours of childcare services, banks, medical offices, etc., as well as extending work-trip–oriented transit services.

The project demonstrated that staggered work hours can help alleviate traffic congestion. However, impacts are not uniform; some commuters will save time, but others will not. Impacts on the transportation system are sufficiently small that they could easily be eroded by latent-demand–related travel shifts, whereas impacts on the individual may be significant. Shifting work schedules is just one possible strategy for traffic management, and this research suggests that it can result in a complex set of costs and benefits.

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Evaluation of Employer-Sponsored Ridesharing Programs in Southern California

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On the basis of a survey of employer-sponsored ridesharing programs in Southern California, firm size appears to be the most important explanatory variable for program choice and effectiveness. Larger firms were significantly more likely to offer direct ridesharing incentives to employees and to report direct benefits to the employer from ridesharing, and they were somewhat more likely to implement staggered work shifts and compressed work weeks but not flexible work hours. Significant economies of scale occur in providing personalized matching assistance to employees. The employees of larger firms were significantly more likely to rideshare, apart from other firm, program, and policy factors. These results strongly suggest that public policy on ridesharing, to produce less costly, more effective, and thus more efficient results, should focus on larger firms. Personalized matching assistance was highly effective in increasing the level of ridesharing, but direct ridesharing incentives were not. Alternative work schedules may hinder the formation of ridesharing arrangements, at least in some cases. The regional coordination of ridesharing promotional efforts may be necessary from a public policy perspective, but it is not sufficient, by itself, to ensure an efficient level of ridesharing. Employer-sponsored ridesharing programs may be the single most effective strategy to promote efficient levels of ridesharing on a regional basis. Most firms do not actively promote ridesharing on a voluntary basis, however. The participation of both the private and public sectors is necessary to maintain regional mobility through transportation demand management strategies such as ridesharing.

Although traffic congestion most often is experienced at the local level, traffic mitigation measures generally can be implemented efficiently only when coordinated at the regional level (*1*). Traffic congestion is an indication of spatial or temporal imbalance between transportation supply, measured as transportation system capacity, and demand, derived from proximate land uses and their associated economic activities (*2*). This disequilibrium condition can be addressed over the longer term only by coordinating transportation and land use planning and investment decisions, a process that is typically successful only at the regional level (*3,4*).

Ridesharing as a public policy tool was introduced during World War II to conserve rubber and other natural resources vital to the ongoing war effort (*5*). Largely neglected after the war, ridesharing was revived as a conservation measure in the aftermath of the first Arab oil embargo in 1974. At least in theory, ridesharing may also be a useful transportation demand management technique aimed at mitigating traffic congestion at the local and regional levels. Whether ridesharing is suitable in modern urban and suburban environments, and if so, under what conditions, is controversial, however (*6*).

Regional ridesharing programs have not been particularly successful in increasing the level of ridesharing regionally (*7*, p. 1,*8*,*9*). The best-documented regional ridesharing programs have reduced regional vehicle miles of travel (VMT) directly or indirectly by 1 percent or less overall (*8*). This level of historical regional ridesharing program performance is insignificant compared with the average annual rate of growth in VMT (2 to 3 percent) in rapidly growing communities, which would be those most interested in ridesharing as a traffic mitigation measure. Occasionally, regional ridesharing efforts have had more significant results, as during the 1984 Summer Olympics, but those results were short lived, even during the 2 weeks of the games (*9*).

Employer-sponsored ridesharing programs, at least in some instances, have produced far more spectacular results. One comprehensive employee transportation demand management program, CH2MHill in Bellevue, Washington, reduced solo driving from 85 to 60 percent in just a few months. This program featured an on-site ridesharing coordinator, computerized carpool matching services, public transit subsidies, parking fees for solo drivers, and free parking for carpools and vanpools (*10*). A similar program sponsored by the Nuclear Regulatory Commission in north Bethesda, Maryland, reduced solo driving from 54 to 42 percent in a period of just 6 months (*11*). The Lawrence Livermore Laboratories in Livermore, California, with a program having more emphasis on ridesharing services and less on parking management, reduced solo driving from 85 to 36 percent after 5 years (*12*). Driving alone at Lawrence Livermore Laboratories has reverted to about 51 percent, however, presumably because its new management places less emphasis on ridesharing (*10*). Some observers argue that these isolated instances of success are insufficient to justify the expectations from and major commitments to transportation demand management (TDM) programs currently underway in California and other high-growth areas (*13*).

Why are employer-sponsored ridesharing programs more successful than regional ridesharing programs, at least occasionally? Because regional ridesharing programs alone apparently are not sufficient to produce significant results, are they necessary at all? These and other questions are considered in the context of a detailed analysis of the results of a 1985 survey of large firms in Southern California, all clients of Commuter

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Transportation Services, Inc. (CTS), the regional ridesharing agency for Southern California, except Orange County.

DATA AND RESEARCH METHODOLOGY

The influence of employer-sponsored ridesharing programs and alternative work schedules on employee mode choice was analyzed using a choice-based sample of Southern California firms. A mail survey of all CTS client headquarters worksites was conducted in early 1985. This sample plan controlled for external ridesharing assistance to each firm. At the time the survey was taken, CTS pursued a completely undifferentiated marketing strategy in providing its basic computerized ridesharing matching services to clients.

The sample included about 7 percent of all Southern California's manufacturing firms and 5 percent of its service firms with at least 100 employees. The average size of sample firms was 862 employees.

Of the 863 surveys mailed, 432 were returned. Responding firms had 372,206 employees among them, or just less than 10 percent of the entire Southern California regional work force. More than 90 percent of the responding firms had at least 100 employees (*14*).

Variables developed from the survey and used in the analysis are listed in Table 1. Endogenous variables include employee mode split, aggregated by firm, and the level of personalized matching assistance, types of direct ridesharing incentives, and alternative work schedules offered to employees. Exogenous variables include firm size, industry type, and location. Program organization and management attitudes toward ridesharing are also considered.

Employer location is identified at the regional level. Regional centers are defined on the basis of level of employment, a proxy for the average localized density of development. Downtown Los Angeles is the region's primary center, with more than 225,000 employees. Secondary centers include high-density inner suburban communities such as Glendale, Long Beach, Pasadena, Santa Monica, and the Wilshire corridor. Tertiary centers include all other areas of Los Angeles County. Areas outside Los Angeles County are treated separately. Site characteristics are identified on the basis of land tenure

TABLE 1 VARIABLES USED IN THE ANALYSIS

Variable	Mean	Definition
BUS_PRG	0.02	1 if buspool program offered, 0 else
VAN_PRG	0.13	1 if vanpool program offered, 0 else
CAR_PRG	0.12	1 if carpool program offered, 0 else
LN_PSH	4.13	ln (total annual program staff hours)
LN_HPE	0.67	LN_PSH/LN_EMP
LN_PSD	7.00	ln (total annual program staff dollars)
LN_DPE	1.15	LN_PSD/LN_EMP
STG_HRS	0.23	1 if staggered work shifts offered, 0 else
CMP_HRS	0.14	1 if compressed work weeks offered, 0 else
FLX_HRS	0.27	1 if flexible work hours offered, 0 else
STG_HRSN	0.10	1 if staggered work shifts offered in the absence of other work hours policies, 0 else
STG_HRSY	0.13	1 if staggered work shifts offered in the presence of other policies, 0 else
LG_DRV	1.64	logit (employee drive alone mode split)
LG_POL	-2.05	logit (employee ridesharing mode split)
LG_TRN	-3.62	logit (employee public transit mode split)
LN_EMP	5.92	ln (total number of on-site employees)
PUB_IND	0.18	1 if public agency, 0 else
SRV_IND	0.35	1 if service firm, 0 else
PRI_CEN	0.06	1 if primary center location, 0 else
SEC_CEN	0.18	1 if secondary center location, 0 else
NLA_CEN	0.16	1 if outside Los Angeles County, 0 else
MLT_OWN	0.09	1 if multiple tenant owner, 0 else
MLT_RNT	0.14	1 if multiple tenant renter, 0 else
SNG_RNT	0.17	1 if single tenant renter, 0 else
LOW_MAN	0.41	1 if lower/non-management, 0 else
TCH_DPT	0.24	1 if technical department, 0 else
REG_ONL	0.72	1 if regulatory compliance reasons only, 0 else
INT_CON	0.65	1 if internal program constraints reported, 0 else
EXT_CON	0.57	1 if external program constraints reported, 0 else
COM_BEN	0.12	1 if community ridesharing benefits reported, 0 else
FRM_BEN	0.16	1 if employer ridesharing benefits reported, 0 else
EMP_BEN	0.19	1 if employee ridesharing benefits reported, 0 else

and tenancy. Property owners generally have a larger stake in their location than renters, making them more sensitive to site-specific transportation problems. Multiple-tenant facilities are generally higher in density than single-tenant facilities and thus have higher land and parking costs, making them more likely candidates for employer ridesharing programs. Most other variable definitions are relatively straightforward or are addressed at appropriate places in the text. The analysis used a variety of techniques, including cross-tabulation for categorical variables, multiple regression analysis for impact analysis, and comparisons of elasticity measures for sensitivity analysis.

EMPLOYER-SPONSORED RIDESHARING PROGRAMS

The survey categorized employer-sponsored ridesharing programs along two dimensions: program content and the level of resource commitment on an annual basis. Program content, a discrete measure, was identified according to types of ridesharing incentives offered to employees; these included no direct incentives, carpool incentives, vanpool incentives, and buspool incentives (*15*). Resource commitment, a continuous measure, was identified as the annual number of staff-hours and dollars spent on staff time devoted to promoting ridesharing (*16*).

Only 252 of the responding firms (58 percent) estimated the total annual number of staff hours spent on ridesharing, and 184 firms (43 percent) estimated the total annual dollar cost of such staff time. These firms spent an average of 339 staff-hours at a total cost of $5,197 per year on the promotion of employee ridesharing. This amounted to an annual average of 0.31 hr and $5.07 per employee. Clearly, employer-sponsored ridesharing programs in Southern California were not particularly expensive to administer.

Personalized Matching Assistance

The provision of personalized matching services to employees was found to exhibit significant economies of scale in production and distribution. Staff hours increased by an average of only 57 percent and staff dollars by 68 percent with every 100 percent increase in the number of employees served (Table 2). Thus, although total program costs increased with firm size, costs per employee declined. Larger firms were able to provide more services at the same cost per employee or the

TABLE 2 PERSONALIZED MATCHING ASSISTANCE BY FIRM AND PROGRAM CHARACTERISTICS

Independent Variables	Dependent Variables LN_PSH	LN_HPE	LN_PSD	LN_DPE
CONSTANT	+0.30 (0.34)	+0.70 (6.94)	+2.79 (4.65)	+1.64 (15.7)
BUS_PRG	+1.30 (2.50)	+0.17 (2.00)	+0.65 (1.17)	+0.14 (1.43)
VAN_PRG	+0.65 (2.51)	+0.10 (2.27)	+0.66 (2.47)	+0.09 (1.98)
CAR_PRG	+0.17 (0.70)	+0.04 (0.97)	+0.26 (1.05)	+0.05 (1.09)
LN_EMP	+0.57 (6.50)	-0.02 (1.14)	+0.68 (7.52)	-0.08 (5.20)
PUB_IND	-0.39 (1.59)	-0.06 (1.50)	-0.56 (2.25)	-0.06 (1.49)
SRV_IND	-0.20 (0.95)	-0.04 (1.11)	-0.41 (1.85)	-0.06 (1.67)
PRI_CEN	+0.88 (2.26)	+0.12 (1.90)	+0.63 (1.65)	+0.10 (1.43)
SEC_CEN	-0.10 (0.44)	-0.01 (0.34)	-0.06 (0.26)	-0.01 (0.25)
NLA_CEN	+0.54 (2.30)	+0.09 (2.16)	+0.32 (1.38)	+0.06 (1.36)
MLT_OWN	+0.67 (2.03)	+0.10 (1.80)	+1.15 (3.29)	+0.18 (3.01)
MLT_RNT	+0.27 (1.08)	+0.03 (0.76)	+0.31 (1.18)	+0.04 (0.82)
SNG_RNT	-0.36 (1.48)	-0.07 (1.61)	-0.19 (0.72)	-0.01 (0.32)
LOW_MAN	-0.24 (1.41)	-0.05 (1.63)	-0.35 (1.98)	-0.06 (2.04)
TCH_DPT	+0.61 (2.94)	+0.09 (2.66)	+0.71 (3.21)	+0.10 (2.49)
REG_ONL	-0.20 (1.07)	-0.03 (1.09)	-0.46 (2.38)	-0.09 (2.56)
INT_CON	+0.16 (0.80)	+0.04 (1.17)	+0.12 (0.57)	+0.02 (0.66)
EXT_CON	+0.29 (1.46)	+0.06 (1.70)	+0.41 (2.06)	+0.09 (2.48)
EMP_BEN	-0.14 (0.73)	-0.02 (0.53)	-0.11 (0.57)	-0.02 (0.53)
Log likelihood	-416.83	30.14	-281.17	40.13
N	252	252	184	184

NOTES: Based on tobit regression analysis. Predicted values for all dependent variables were constrained to equal or exceed 0 using maximum likelihood estimation procedures. t-scores are in parentheses.

same level of services at a lower cost than smaller firms could. Other factors influencing the provision of personalized matching assistance included the type of direct ridesharing incentives offered, industry type, firm location, ridesharing program management and organization, and management attitudes toward ridesharing (Table 2).

Direct Ridesharing Incentives

Only 27 percent of the responding firms offered direct ridesharing incentives to employees. This was perhaps not too surprising, because fully 72 percent of responding firms listed compliance with regional air quality regulations as their sole reason for developing a ridesharing program in the first place (Table 1). Larger firms were significantly more likely than smaller firms to offer direct ridesharing incentives to employees (Table 3). In fact, the largest firms (2,000 or more employees) were almost 10 times as likely as the smallest firms (fewer than 250 employees) to offer direct ridesharing incentives. The likelihood of offering carpool incentives did not increase much for firms with more than 250 employees, but the likelihood of offering vanpool incentives increased rapidly with firm size for all categories. Only firms with 1,000 or more employees offered buspool incentives. Clearly, the level of direct ridesharing incentives offered to employees was strongly influenced by firm size, presumably because of economies of scale in the provision of ridesharing services.

Reported Benefits from Ridesharing

Ridesharing benefits may accrue to employees, employers, and the community (*15*). Employee ridesharing benefits include lower commuting costs, reduced wear and tear on commute vehicles, and less commuting stress. Employer ridesharing benefits include reduced employee parking requirements, less employee tardiness and absenteeism, improved employee morale and productivity, and enhanced employee recruitment and retention. Community ridesharing benefits include reduced air pollution, energy consumption, and traffic congestion.

Only 38 percent of the surveyed firms reported any benefits from ridesharing. Among these, 19 percent cited employee benefits; 16 percent, employer benefits; and 12 percent community benefits. Larger firms were significantly more likely than smaller firms to report direct employer benefits from ridesharing, but not employee or community benefits (Table 4). Employer ridesharing benefits should increase systematically with firm size if significant economies of scale are realized in the provision of ridesharing services to employees. Employee and community benefits, which are external to the firm, would not necessarily be related to firm size.

Both employer and community benefits from ridesharing increased significantly with the level of direct ridesharing incentives employers offered, although employee ridesharing benefits did not (Table 5). The meaning of this relationship is not entirely clear but may have something to do with general management and labor relations. If employees benefit from

TABLE 3 DIRECT RIDESHARING INCENTIVES OFFERED, BY FIRM SIZE

	Direct Ridesharing Incentives Offered				
Firm Size	No Incentives	Carpool Incentives	Vanpool Incentives	Buspool Incentives	Total Firms
<250 employees	154 (92%)	7 (4%)	6 (4%)	0 (0%)	167 (39%)
250-499 employees	82 (73%)	17 (15%)	14 (12%)	0 (0%)	113 (26%)
500-999 employees	47 (65%)	13 (18%)	12 (17%)	0 (0%)	72 (17%)
1,000-1,999 employees	24 (57%)	7 (17%)	9 (21%)	2 (5%)	42 (10%)
2,000+ employees	10 (26%)	7 (18%)	14 (37%)	7 (18%)	38 (9%)
Total Firms	317 (73%)	51 (12%)	55 (13%)	9 (2%)	432 (100%)
Chi-square[1]	81.55	15.69	36.31	58.69	
Degrees of freedom	4	4	4	4	
Level of significance	0.001	0.01	0.001	0.001	

1 Chi-square calculated for each *column* treated separately as the dependent variable.

TABLE 4 RIDESHARING BENEFITS REPORTED BY FIRM SIZE

	Ridesharing Benefits Reported[1]				
Firm Size	Employee Benefits	Employer Benefits	Community Benefits	Any Benefits[2]	Total Firms[3]
<250 employees	22 (13%)	18 (11%)	15 (9%)	47 (28%)	167 (39%)
250-499 employees	26 (23%)	15 (13%)	17 (15%)	46 (41%)	113 (26%)
500-999 employees	14 (19%)	8 (11%)	6 (8%)	26 (36%)	72 (17%)
1,000-1,999 employees	9 (21%)	13 (31%)	7 (16%)	22 (52%)	42 (10%)
2,000+ employees	9 (24%)	17 (45%)	7 (18%)	25 (66%)	38 (9%)
Total Firms	80 (19%)	71 (16%)	52 (12%)	166 (38%)	432 (100%)
Chi-square[4]	5.62	34.81	5.68	23.36	
Degrees of freedom	4	4	4	4	
Level of significance	0.30	0.001	0.30	0.001	

1 Multiple response possible.
2 Employee, employer, *or* community benefits from ridesharing reported.
3 *Rows* do not add to 100% because some firms reported no benefits from ridesharing.
4 Chi-square calculated for each *column* treated separately as the dependent variable.

ridesharing, then program participation might be expected to increase, perhaps engendering additional program costs to the firm.

Indirect Measures: Alternative Work Schedules

Alternative work schedules include staggered work shifts, flexible work hours, and compressed workweeks. Staggered work shifts, through appropriate scheduling by the employer, thin out employee peak-period travel at a single location by separating the arrival and departure times for each major shift's employees. Staggered work shifts are applied most often at large installations, such as military bases, hospitals, universities, and major manufacturing employment centers. Flexible work hours have a similar effect, but employees are allowed to choose start and end times to suit their own convenience, within specific employer guidelines. The potential result is congestion relief along major travel corridors leading to the employment site. Compressed workweeks increase the number of hours worked per day and decrease the number of days worked each week. The direct result is an absolute reduction in the total number of trips made and a shift in work arrival and departure times away from at least one daily peak-travel period. The overall result may be a reduction in total regional VMT and in peak-period traffic congestion.

Of the responding firms, 43 percent offered alternative work schedules of one type or another (Table 6). Twenty-seven percent allowed flexible work hours, 22 percent staggered work shifts, and 14 percent compressed their workweek. Larger firms were more likely to offer staggered shifts and compressed work weeks, but the firm size difference was only slight for flexible work hours. The expectation had been that larger firms could more easily accommodate individual flexible work hours and still have adequate office coverage during normal business hours than could smaller firms. These results suggest that flexible work hours may have somewhat wider applicability as a TDM strategy than previously thought, at least for firms with 100 or more employees.

The relationship between the level of direct ridesharing incentives offered and alternative work schedules was not significant, with the weak exception of staggered work shifts (Table 7). The choice of program type was almost completely independent. Staggered work shifts and compressed workweeks were both moderately related to the level of ridesharing benefits reported, however (Table 8). More specifically, firms reporting direct employer ridesharing benefits were more likely to offer all types of alternative

TABLE 5 RIDESHARING BENEFITS REPORTED BY DIRECT RIDESHARING INCENTIVES OFFERED

Direct Ridesharing Incentives Offered	Ridesharing Benefits Reported[1]				
	Employee Benefits	Employer Benefits	Community Benefits	Any Benefits[2]	Total Firms[3]
No Incentives	57 (18%)	31 (10%)	29 (9%)	97 (31%)	317 (73%)
Carpool Incentives	10 (20%)	13 (25%)	4 (8%)	25 (49%)	51 (12%)
Vanpool Incentives	12 (22%)	21 (38%)	14 (26%)	37 (67%)	55 (13%)
Buspool Incentives	1 (11%)	6 (67%)	5 (56%)	7 (78%)	9 (2%)
Total Firms	80 (19%)	71 (16%)	52 (12%)	166 (38%)	432 (100%)
Chi-square[4]	0.82	48.74	28.80	35.86	
Degrees of freedom	3	3	3	3	
Level of significance	0.95	0.001	0.001	0.001	

1 Multiple response possible.
2 Employee, employer, *or* community benefits from ridesharing reported.
3 *Rows* do not add to 100% because some firms reported no benefits from ridesharing.
4 Chi-square calculated for each *column* treated separately as the dependent variable.

work schedules, significantly so in the case of staggered shifts and compressed workweeks.

Parking Management

Parking pricing and supply clearly are critical factors influencing employee mode choice (*17*). Parking management was not considered explicitly in this analysis, however. Virtually all of the responding firms (98 percent) offered free or subsidized parking to some or all of their employees. Of those few firms that did use parking pricing or supply control mechanisms, many charged relatively little for employee parking, and most (81 percent) did not have adequate records on which to base accurate parking cost estimates. Thus, for most surveyed firms, parking management consisted of providing free parking to all employees. Carpool and vanpool preferential parking spaces were identified in the analysis as direct ridesharing incentives.

EMPLOYEE MODE CHOICE

The true test of the effectiveness of employer-sponsored ridesharing programs should be in terms of their effects on employee mode choices (*18*). Unfortunately, data on disaggregate discrete employee mode choices are not available from the survey. Each firm, however, was asked to estimate aggregate employee mode split, including the percentage of employees commuting to work by driving alone, carpooling, vanpooling, buspooling, taking public transit, or using other modes of travel such as bicycling and walking. Overall, 291 of the responding firms (67 percent) supplied such an estimate. On average, 75 percent of their employees drove alone, 16 percent carpooled or vanpooled, 5 percent took public transit, and 4 percent used other modes of travel for their daily commute.

A comparison of employer policy measures, such as ridesharing programs, directly with employee mode choices, controlling simultaneously for the complex social, economic, policy, and environmental factors influencing these daily decisions is not possible given the limitations of the CTS data. A second-best alternative is to treat employee mode split, aggregated by firm, as a proxy for the sum of individual employee mode choices at each firm. This aggregate employee mode split variable (or variables) can be analyzed using the weighted least squares regression technique proposed by Theil (*19*). Dependent variables analyzed here include the drive alone, ridesharing (carpool and vanpool), and public transit mode splits for each firm, transformed into log-likelihood ratios, or logits, as follows:

$$\text{logit} = \ln \frac{P}{1 - P} \qquad (1)$$

TABLE 6 ALTERNATIVE WORK SCHEDULES POLICIES BY FIRM SIZE

Firm Size	Alternative Work Schedules Policies[1]				Total Firms[3]
	Staggered Work Shifts	Flexible Work Hours	Compressed Work Weeks	Any Policies[2]	
<250 employees	31 (19%)	43 (26%)	17 (10%)	69 (41%)	167 (39%)
250-499 employees	19 (17%)	29 (26%)	14 (12%)	43 (38%)	113 (26%)
500-999 employees	19 (26%)	18 (25%)	8 (11%)	29 (40%)	72 (17%)
1,000-1,999 employees	16 (38%)	12 (29%)	12 (29%)	25 (60%)	42 (10%)
2,000+ employees	13 (34%)	14 (37%)	8 (21%)	19 (50%)	38 (9%)
Total Firms	98 (23%)	116 (27%)	59 (14%)	185 (43%)	432 (100%)
Chi-square[4]	12.97	2.30	11.95	6.98	
Degrees of freedom	4	4	4	4	
Level of significance	0.05	0.70	0.05	0.20	

1 Multiple response possible.
2 Staggered work shifts, flexible work hours, *or* compressed work weeks policies reported.
3 *Rows* do not add to 100% because some firms reported no alternative work schedules policies.
4 Chi-square calculated for each *column* treated separately as the dependent variable.

where P is equal to the percentage of a firm's employees using a particular mode of travel. Weights are applied to the left- and right-hand sides of each equation in summing error terms, to control for differences in sample size and in the likelihood that employees will choose a particular mode. The results are shown in Table 9.

Controlling for a variety of other firm, program, and policy factors, firm size still was associated with a significant increase in employee ridesharing, which occurred about equally at the expense of driving alone and public transit use. Firm size may be related indirectly to spatial interactions that are external to the firm. For example, firms may be so large that they directly create development density by virtue of their location decisions. This would apply principally in underdeveloped or low-density areas. A stronger hypothesis is that large firms are more likely to prefer high-density locations than are small firms. This is an agglomeration or external economies argument. Large firms may also use space more efficiently than smaller firms, creating the effect of high-density development. This is an internal economies argument. In any case, the employees of large firms were significantly more likely to rideshare than the employees of small firms in this analysis.

Personalized matching assistance was associated with a significant increase in the level of ridesharing at individual firms. This supports the notion that the ridesharing coordinator plays a pivotal role in determining the success of employer transportation programs (*16*). By contrast, direct incentives were not associated with significant increases in ridesharing. The use of such incentives as preferential carpool and vanpool parking to encourage ridesharing, at least in the absence of parking pricing and supply control measures (*20,21*), is brought into question by these results.

Alternative work schedules were associated with increases, decreases, or no change in the level of ridesharing in this analysis, depending on the combination of alternative work schedules offered to employees. Compressed workweeks and flexible work hours were associated with increases in driving alone and decreases in ridesharing and public transit use. Staggered shifts in the presence of compressed work weeks or flexible work hours had the opposite effect—increases in ridesharing and public transit use and decreases in driving alone. Staggered shifts in the absence of compressed workweeks and flexible work hours were not significantly related to the employee mode choice. Alternative work schedules that employees may choose but then must adhere to apparently increase ridesharing by making potential carpool partners more dependable and predictable, useful characteristics when one is expected to arrive at work on time. Alternative

TABLE 7 ALTERNATIVE WORK SCHEDULES POLICIES BY DIRECT RIDESHARING INCENTIVES OFFERED

Direct Ridesharing Incentives Offered	Alternative Work Schedules Policies[1]				
	Staggered Work Shifts	Flexible Work Hours	Compressed Work Weeks	Any Policies[2]	Total Firms[3]
No Incentives	61 (19%)	84 (26%)	39 (12%)	129 (41%)	317 (73%)
Carpool Incentives	15 (29%)	10 (20%)	9 (18%)	21 (41%)	51 (12%)
Vanpool Incentives	20 (36%)	19 (35%)	10 (18%)	30 (55%)	55 (13%)
Buspool Incentives	2 (22%)	3 (33%)	1 (11%)	5 (56%)	9 (2%)
Total Firms	98 (23%)	116 (27%)	59 (14%)	185 (43%)	432 (100%)
Chi-square[4]	9.33	3.23	2.19	4.33	
Degrees of freedom	3	3	3	3	
Level of significance	0.05	0.50	0.70	0.30	

1 Multiple response possible.
2 Staggered work shifts, flexible work hours, *or* compressed work weeks policies reported.
3 *Rows* do not add to 100% because some firms reported no alternative work schedules policies.
4 Chi-square calculated for each *column* treated separately as the dependent variable.

work schedules that are too flexible may discourage ridesharing by allowing daily travel decisions to vary sufficiently to reduce dependability (*22,23*). Compressed workweeks had a very negative impact on public transit use. This may have been related to the span and frequency of public transit service in Southern California. The longer work days associated with compressed workweeks might make public transit use outside normal peak travel periods too inconvenient. Ridesharing is less dependent than public transit on external agents for service delivery and would be less negatively affected by the time of day the commute is made.

Service firms and public agencies showed higher levels of public transit use than did manufacturing firms. This difference may be related to patterns of industrial location and the availability of public transit service. Public agencies showed a lower level of ridesharing than did private firms. Public agencies often are tied to particular locations, are more likely to own land, and are less likely to perceive land ownership as an opportunity cost. The availability of more abundant land for parking may account for the higher propensity of public employees to drive alone (*24*).

Levels of transit use were much higher among employees of primary and secondary center firms than among those less centrally located. This was undoubtedly related to the supply of transit service. Firms located outside Los Angeles County showed less transit use but more ridesharing, providing some evidence that ridesharing may substitute for transit use in certain situations.

Site characteristics were only marginally related to employee mode choice. Employees of multiple-tenant owners were slightly more likely than other employees to drive alone. This may have occurred in response to public policy on parking. Federal regulations allow employee free parking as a nontaxable benefit. This policy may increase the supply of, and especially the demand for, employee parking spaces. Multiple-tenant owners may have insufficient parking because of dense development, and they may wish to retain as much as possible of this limited supply of parking for their employees to internalize the available tax breaks.

Overall, personalized matching assistance appears to have been effective in increasing the level of employee ridesharing at Southern California firms, while direct ridesharing incentives were not.

RIDESHARING PROGRAM COST-EFFECTIVENESS

Although a direct comparison of the costs and benefits of ridesharing would be useful, most employers have difficulty

TABLE 8 ALTERNATIVE WORK SCHEDULES POLICIES BY RIDESHARING BENEFITS REPORTED

Ridesharing Benefits Reported	Alternative Work Schedules Policies[1] Staggered Work Shifts	Flexible Work Hours	Compressed Work Weeks	Any Policies[2]	Total Firms[3]
No Benefits	50 (19%)	68 (26%)	25 (9%)	105 (39%)	266 (62%)
Employee Benefits	17 (29%)	17 (29%)	11 (19%)	27 (47%)	58 (13%)
Employer Benefits	20 (36%)	18 (32%)	15 (27%)	30 (54%)	56 (13%)
Community Benefits	11 (21%)	13 (25%)	8 (15%)	23 (44%)	52 (12%)
Total Firms	98 (23%)	116 (27%)	59 (14%)	185 (43%)	432 (100%)
Chi-square[4]	9.23	1.29	13.79	4.23	
Degrees of freedom	3	3	3	3	
Level of significance	0.05	0.80	0.01	0.30	

1 Multiple response possible.
2 Staggered work shifts, flexible work hours, *or* compressed work weeks policies reported.
3 *Rows* do not add to 100% because some firms reported no alternative work schedules policies.
4 Chi-square calculated for each *column* treated separately as the dependent variable.

estimating ridesharing benefits, at least in dollar terms (*25*). Many believe the benefits of ridesharing clearly outweigh the costs (*26*). In place of cost-benefit analysis, transportation system effectiveness analysis can be used if benefits are difficult or impossible to ascertain (*27*). The equations shown in Tables 2 and 9 can be used to evaluate the cost-effectiveness of typical employer-sponsored ridesharing programs in Southern California by comparing ridesharing program staff expenditures with the percentage of employees shifting from one mode of travel to another. Typical ridesharing program staff expenditures can be estimated for firms of different sizes using the Table 2 equations. The average relationship of such expenditures to employee mode choice can be estimated using the Table 9 equations.

Only typical firm and program characteristics—those most often found in the survey itself—are used. The typical surveyed firm was engaged in private manufacturing (47 percent), offered no direct ridesharing incentives to employees (73 percent), reported no benefits from ridesharing (62 percent), had no alternative work schedules (57 percent), offered free or subsidized parking to some or all employees (98 percent), was located in a tertiary employment center of Los Angeles County (60 percent), owned the site it occupied and occupied the site exclusively (60 percent), developed its ridesharing program to comply with regional air quality regulations only (72 percent), and reported significant constraints on program expansion, both internally, such as lack of management interest (65 percent), and externally, such as lack of employee interest (57 percent).

The variable definitions used in this analysis allow dramatic simplification of the equations shown in Tables 2 and 9. Specifically, the following equations can now be used:

$$\text{LN\_PSD} = 2.86 + 0.68 * \text{LN\_EMP} \quad (2)$$

$$\text{LG\_DRV} = 2.91 - 0.12 * \text{LN\_EMP} - 0.73 * \text{LN\_DPE} \quad (3)$$

$$\text{LG\_POL} = -3.65 + 0.17 * \text{LN\_EMP} + 0.96 * \text{LN\_DPE} \quad (4)$$

$$\text{LG\_TRN} = -3.11 - 0.09 * \text{LN\_EMP} + 0.02 * \text{LN\_DPE} \quad (5)$$

In order to illustrate the complex effects of firm size on employer-sponsored ridesharing program costs and cost-effectiveness, firms with 100, 1,000, and 10,000 on-site

TABLE 9 EMPLOYEE MODE CHOICE BY FIRM AND PROGRAM CHARACTERISTICS

Independent Variables	Dependent Variables LOG_DRV	LOG_POL	LOG_TRN
CONSTANT	+2.91 (3.67)	-3.65 (4.39)	-3.11 (3.26)
BUS_PRG	-0.32 (0.93)	+0.19 (0.56)	+0.23 (0.46)
VAN_PRG	-0.09 (0.45)	+0.22 (1.02)	-0.33 (1.28)
CAR_PRG	+0.02 (0.13)	+0.05 (0.29)	-0.29 (1.16)
LN_DPE	-0.73 (1.84)	+0.96 (2.31)	+0.02 (0.04)
CMP_HRS	+0.52 (2.27)	-0.35 (1.47)	-0.68 (2.39)
FLX_HRS	+0.39 (1.91)	-0.32 (1.54)	-0.39 (1.54)
STG_HRSN	-0.14 (0.56)	+0.15 (0.57)	+0.01 (0.03)
STG_HRSY	-0.50 (2.13)	+0.54 (2.21)	+0.36 (1.23)
LN_EMP	-0.12 (1.56)	+0.17 (2.05)	-0.09 (0.87)
PUB_IND	+0.29 (1.36)	-0.52 (2.39)	+0.65 (2.35)
SRV_IND	-0.25 (1.20)	-0.13 (0.59)	+1.36 (5.05)
PRI_CEN	-0.62 (2.23)	-0.16 (0.53)	+1.65 (5.30)
SEC_CEN	-0.22 (0.93)	-0.11 (0.44)	+0.91 (3.25)
NLA_CEN	-0.26 (1.45)	+0.38 (2.08)	-0.77 (2.89)
MLT_OWN	+0.32 (1.25)	-0.39 (1.44)	-0.12 (0.40)
MLT_RNT	-0.20 (0.86)	+0.19 (0.78)	-0.08 (0.28)
SNG_RNT	-0.04 (0.16)	-0.02 (0.07)	+0.47 (1.44)
R-Squared Adj.	0.05	-0.11	0.31
N	136	136	136

NOTES: Based on weighted least squares regression analysis. t-scores are in parentheses.

employees were evaluated. More than 90 percent of the surveyed firms had 100 or more employees, while all but three firms had fewer than 10,000 employees. These three size classes of firms thus illustrate the full range of economies of scale within the normal distribution of Southern California firms with ridesharing programs. The results are shown in Table 10.

Total ridesharing program costs to employers increased systematically with firm size, while costs per employee declined. The average level of effort of most firms was limited. For example, a firm with 1,000 employees typically spent a total of $1,785, or $1.79 per employee, on personalized matching assistance.

The average mode split for a firm with 1,000 employees and no personalized matching assistance was approximately 89 percent drive alone, 8 percent ridesharing, and 2 percent public transit use. Public transit use varies little with firm size. A firm with 10,000 employees would be expected to have more than twice the level of ridesharing as a firm with 100 employees, assuming no personalized matching assistance is provided and holding all else equal.

The average mode split for a firm with 1,000 employees that provides a typical level of personalized matching assistance was 78 percent drive alone, 19 percent ridesharing, and 2 percent public transit use. Once again, public transit use was little affected by the level of personalized matching assistance provided. The decrease in the percentage that drive alone produced by typical levels of personalized matching assistance was virtually identical across all firm size classes, varying only from 10.48 percent to 11.07 percent.

The absolute number of employees shifted out of driving alone by typical levels of personalized matching assistance was almost directly proportional to the size of the firm. That is, larger firms did not generate greater proportional shifts in mode split than smaller firms through typical levels of employer ridesharing program investment. Cost-effectiveness did improve with firm size, however, because per-employee ridesharing program costs decreased significantly with firm size. The cost per person diverted from driving alone to ridesharing was reasonable for all firm size classes, varying from as low as $7.72 per employee of large firms to $33.91 per employee of small firms (Table 10).

THE LIMITS OF EMPLOYER RIDESHARING PROGRAMS

These results on the overall cost-effectiveness of employer ridesharing programs are subject to two major limitations. First, the findings technically are valid primarily for within-group or within-range predictions only. Because most employer ridesharing programs in Southern California are fairly limited in scope at this time, extrapolating from these results to a future in which much more ridesharing promotion is being accomplished can be problematic. Second, the nature of the equations used suggests that costs per person placed into ridesharing increase continuously with the level of program effort. Personalized matching assistance helps to reduce transaction and information costs associated with the formation of

TABLE 10 EMPLOYER RIDESHARING PROGRAM COST-EFFECTIVENESS FOR A TYPICAL SOUTHERN CALIFORNIA FIRM[1]

Firm size (number of employees)	100	1,000	10,000
Ridesharing program staff expenditures[2]			
Per firm	$373	$1,785	$8,544
Per employee	$3.73	$1.79	$0.85
Mode split without personalized matching assistance provided[3]			
Drive alone	91.38	88.94	85.81
Ridesharing	5.35	7.75	11.15
Public transit	2.87	2.35	1.91
Totals	99.60	99.04	98.87
Mode split with personalized matching assistance provided[3]			
Drive alone	80.67	78.46	74.74
Ridesharing	16.30	19.11	24.23
Public transit	2.96	2.40	1.95
Totals	99.93	99.97	100.92
Shift in mode split with personalized matching assistance provided[4]			
Drive alone	- 10.61	- 10.48	- 11.07
Ridesharing	+10.95	+11.36	+13.08
Public transit	+0.07	+0.05	+0.04
Number of employees shifted with personalized matching assistance provided[4]			
Drive alone	- 11	- 105	- 1,107
Ridesharing	+11	+114	+1,308
Public transit	0	+1	+4
Cost effectiveness with personalized matching assistance provided ($/person placed out of driving alone)	$33.91	$17.00	$7.72

1 The typical surveyed firm offered no direct ridesharing incentives (73%) or alternative work schedules (57%) to employees, reported no benefits from ridesharing (62%), was engaged in private manufacturing (47%), was located in a tertiary center of Los Angeles County (60%), and owned the site it occupied, which it occupied exclusively (60%).

2 From Equation 2.

3 From Equations 3, 4, and 5. Percentages may not sum exactly to 100%, due to random as well as systematic errors in the parametric estimation of equations. As long as reasonable (e.g., normal) assumptions are made concerning the hypothetical attributes of firms and programs, systematic errors will remain slight.

4 These numbers may not sum exactly to zero. See note 3.

ridesharing arrangements but does not alter the relative price advantages of different modes of travel (*28*). Thus, personalized matching assistance, by itself, should have a dramatic initial impact, which tapers off with increases in effort beyond a certain point. The question is, How much personalized matching assistance is enough, or, conversely, How much personalized matching assistance is too much?

The following equations were used to evaluate this problem:

$$P = \frac{e^{2.91 - 0.12 \cdot \ln(E)}}{1 + e^{2.91 - 0.12 \cdot \ln(E)}} \quad (6)$$

$$P' = \frac{e^{2.91 - 0.12 \cdot \ln(E) - 0.73 \cdot \ln(\$)/\ln(E)}}{1 + e^{2.91 - 0.12 \cdot \ln(E) - 0.73 \cdot \ln(\$)/\ln(E)}} \quad (7)$$

and

$$C = \frac{\$}{E * (P - P')} \quad (8)$$

where

P = percentage of a firm's employees who drive alone before program implementation;

P' = percentage of a firm's employees who drive alone after program implementation;

E = firm size, measured as the number of employees working on-site;

$\$$ = total annual dollar cost of staff time spent on providing ridesharing services; and

C = index of cost-effectiveness, measured as dollars per person shifting from driving alone.

Equation 8 can be rewritten as

$$P' = \frac{C * E * P - \$}{C * E} \quad (9)$$

Equations 7 and 9, when set equal, produce an objective function for determining the minimum percentage of a firm's employees driving alone after an employer has started a ridesharing program, given firm size and a maximum acceptable value (or limit) for program cost-effectiveness. Thus:

$$\frac{e^{2.91-0.12*\ln(E)-0.73*\ln(\$)/\ln(E)}}{1+e^{2.91-0.12*\ln(E)-0.73*\ln(\$)/\ln(E)}} - \frac{C*E*P*-\$}{C*E} = 0 \qquad (10)$$

If E is given, P is determined (Equation 6), and C and $\$$ can be obtained through the iterative solution of Equation 10. Equation 10 is fundamentally nonlinear and cannot be solved algebraically, except for arbitrarily large or arbitrarily small values of $\$$, which are not relevant here. Once P and $\$$ are known, C and P' can be estimated, and the percentage of a firm's employees shifted out of driving alone ($P - P'$) may be found. This system of equations can be solved iteratively for various levels of cost-effectiveness using a simple spreadsheet formulation to avoid the tedium of repetitious calculations.

The maximum potential of personalized matching assistance to influence employee mode choice in Southern California, in the absence of parking management strategies or direct ridesharing incentives and on the basis of the iterative solution of Equation 10, is shown in Figure 1. Each curve represents the maximum shift in the percentage of employees driving alone that can be obtained, probabilistically, for a given level of cost-effectiveness. Cost-effectiveness is measured here in dollars spent per person placed out of driving alone for firms of four different size classifications, ranging from 100 to 100,000 employees. Few if any individual employers have 100,000 employees at a single work site, but many urban and suburban employment activity centers, some of which have formed transportation management organizations to conduct ridesharing and related programs, approach or exceed this figure. As Figure 2 shows, personalized matching assistance has a clear but ultimately limited ability to shift employees away from driving alone and into ridesharing. Larger firms typically shift more employees into ridesharing for a given level of cost-effectiveness. At very low levels of cost-effectiveness (higher costs per person placed), these economies of scale tend to disappear. Within the typical range of costs found among current employer ridesharing programs, however, the provision of personalized matching assistance clearly has economies of scale.

Depending on the actual marginal social benefit to be derived from ridesharing, the acceptable cost per person placed out of driving alone as a result of personalized matching assistance has limits. Most employee parking spaces cost $1,000 or less per employee per year. Thus, parking pricing or other TDM strategies may be more efficient than personalized matching assistance, at least beyond a certain level of effort. This finding strongly supports the idea that combinations of TDM strategies, rather than individual strategies implemented in isolation, may have the greatest effect on employee mode choice at the lowest costs. In the great majority of cases, personalized matching assistance certainly should be one of those elements.

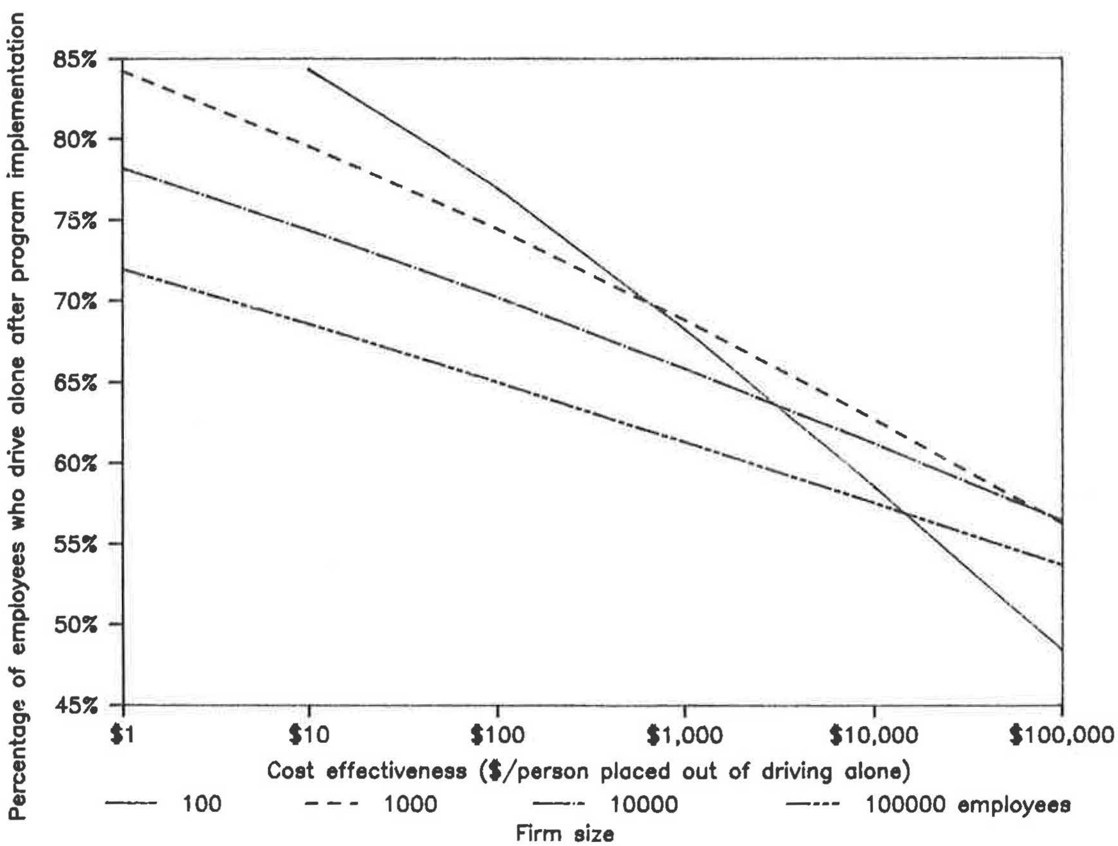

FIGURE 1 Employees driving alone after program implementation.

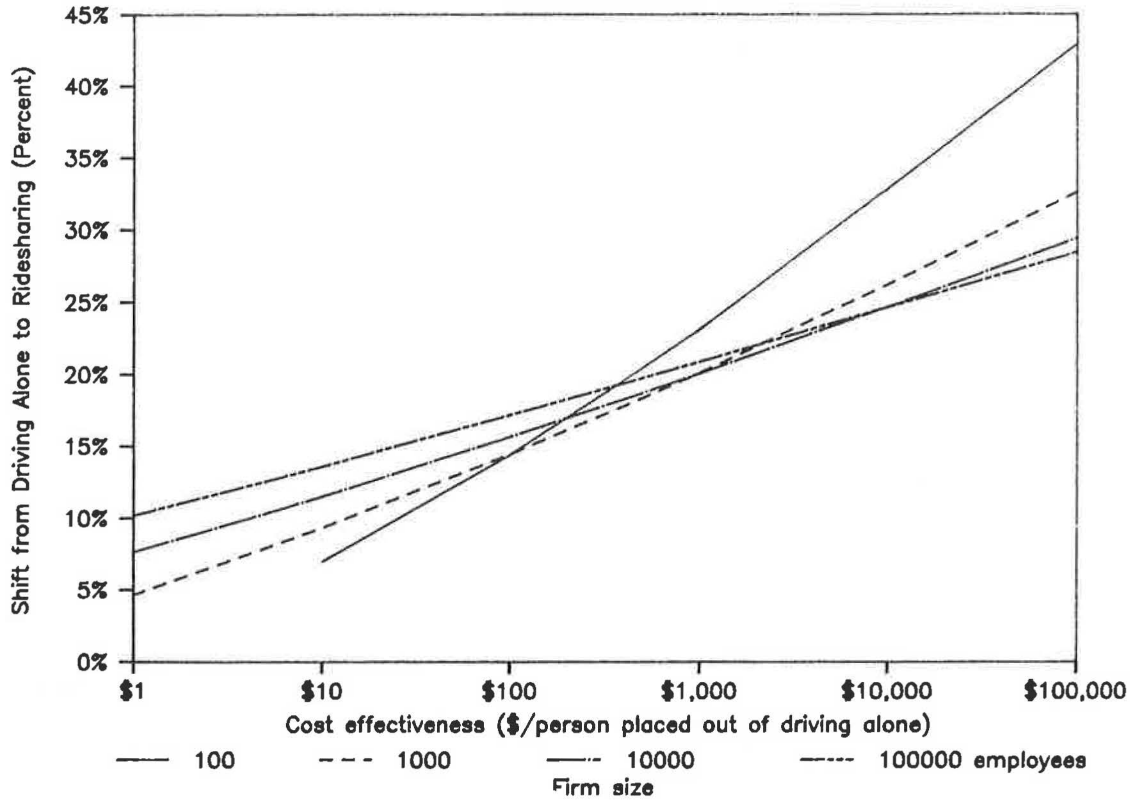

FIGURE 2 The effect of personalized matching assistance on employee mode choice.

CONCLUSIONS

The analysis presented here is increasingly relevant as urban areas develop along the lines of generalized dispersion with polycentric concentration of employment and other activities, which is characteristic of Southern California (*29,30*). Several points of interest to transportation researchers and public policy analysts are evident. Employer-sponsored ridesharing programs are limited to a large extent by both urban form and pricing constraints. Public policy based on such programs can have a significant impact on employee mode choice, however, if efforts center on appropriately identified markets and well-focused implementation strategies. The marginal costs of these programs tend to be small, while the marginal benefits appear to be great. Differences among employers must be taken carefully into account in designing employee ridesharing programs that are successful and cost-effective. The importance of firm size cannot be overstated in this regard. Ridesharing programs, with the economies of scale enjoyed by individual large firms, could be designed to serve groups of firms in large urban and suburban employment centers. Transportation management organizations may have the greatest potential of all because of their potential size and other institutional advantages (*31*). Evaluation results are still pending for many transportation management organizations, but this analysis appears to confirm the theoretical justification for their existence.

Regulatory efforts at the local and regional levels may be useful in inducing individual firms to participate in employee ridesharing programs (*32*). The tacit recognition of employee and community benefits may be achieved at a higher level through regulatory efforts that seek to internalize some of these program costs within a market framework. Clearly, a market for employee ridesharing programs already exists in Southern California, at least in limited form. Regulatory efforts geared toward expanding and improving on these existing market interactions probably will be more effective or efficient than those that are not (*33*). Regional ridesharing agencies might well concentrate less on the direct delivery of ridesharing services to commuters and more on brokering higher level institutional services to employers, developers, and local public agencies.

Personalized matching assistance is effective because it meets the needs of commuters. Direct ridesharing incentives, such as preferential parking for vanpools and carpools, are not effective, at least not in situations where free parking for all employees is the rule rather than the exception. Alternative work schedules may help or hinder the formation of ridesharing arrangements, depending on the form such programs take. These results strongly suggest that transportation demand management cannot be carried out piecemeal and achieve its full potential. Only those programs that are coordinated, both internally and externally, will yield significant results over the long term.

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Predicting Consumer Demand for Alternative Transportation Services Among Suburban Commuters

KEVIN J. FLANNELLY AND MALCOLM S. MCLEOD, JR.

A survey of suburban commuters revealed that their interest in ridesharing and related transportation system management (TSM) strategies, other than flextime, was minimal. The disincentive of high parking costs did not appear to be sufficient to attract riders to standard transit services. Enhanced service, however, provided an important incentive for transit use, even when disincentives were comparatively low. Improvements in service, including express buses, reduced access time, and guaranteed seating, can induce automobile commuters to use alternative transit or paratransit. Moreover, decentralization of service from its downtown focus could open up a sizeable market for alternative transit both among carpoolers and solo drivers. Interest in alternative transit with improved service characteristics is directly related to commute time. Thus, increases in traffic congestion may stimulate demand for alternative transit, even at higher fares. The balance between service and fare that will optimize ridership can be easily deduced for various markets. Demand-response transit services appear to provide a feasible and profitable transit alternative, particularly if they are linked to a computerized, real-time, booking and dispatching network.

Commuters from East Honolulu experience high levels of traffic congestion along the only direct route to the island's major employment centers. A survey conducted in an eastern suburb of the island of Oahu was designed to explore the potential of various transportation system management (TSM) strategies for easing this congestion.

Of particular interest was commuters' inclination to use alternative forms of transit. Fares, service characteristics, hour of travel (with respect to peak-hour congestion), ease of access, miles traveled, and commute time all have been shown to influence mode split and transit ridership (*1–3*). The purpose of the East Honolulu survey was to assess the relative contributions and interactions of these factors on choice of mode and potential demand for alternative transit modes.

METHODOLOGY

Under the aegis of the Hawaii Department of Transportation, all 8,900 or so households in the section of East Honolulu shown in Figure 1 were contacted by mail. Because virtually the entire study area is owned by a single developer, a commercial list of all mailing addresses in the specific area of interest was readily available.

Center for Psychosocial Research, 777 Kapiolani Boulevard, Suite 1824, Honolulu, Hawaii 96813.

A total of 3,322 households in the target population completed and returned their questionnaires, a response rate of 37 percent.

Most of the households in the sample (79.2 percent) consisted of three or four people, and only 13.5 percent were larger. The majority of respondents (60 percent) were between 30 and 50 years of age; 6.7 percent were younger, and 33.4 percent were older. Roughly two-thirds of respondents (64.4 percent) were male, and more than 99 percent had a driver's license.

The questionnaire elicited data on the demographic characteristics of respondents, their present commuting habits, and their attitudes toward and interest in using different travel alternatives. Most of the questions, especially those dealing with travel alternatives, required participants to rate their opinions and judgments on a scale of 0 to 10 (*4,5*). This rating scale allows people to assign a specific, subjective value to their attitudes, and it is particularly valuable in asking people about their probable future behavior (*6*). The ratings can be assumed to reflect respondents' own subjective probability of choosing a given behavioral alternative under the conditions stated in the question (*4*). For example, a rating of 10 assigned to the likelihood of using an express bus at a \$1.00 fare is assumed to indicate 100 percent certainty that the respondent would use an express bus at a \$1.00 fare. A rating of 0 to the same item expresses certainty that the respondent will not take an express bus; that is, that the respondent's subjective probability of using an express bus is zero under given conditions.

To obtain demand estimates from these data, each rating was multiplied by 10. The average, or mean, likelihood score (the rating times 10) for any question provides an estimate of the percentage of people that are likely to use a given alternative. The standard deviation of the mean was used to calculate an error of estimate (the standard error of the mean). Although the relationship between respondents' likelihood scores and their actual behavior has yet to be validated, this method should prove to be more accurate than other commonly used scaling methods for determining consumer preferences.

Continuous variables, such as travel time and the attitudinal and behavioral ratings, were analyzed by parametric techniques, such as analysis of variance (ANOVA). Unweighted means ANOVA was used for most purposes to correct for disparities in sample sizes when participants were classified into subgroups such as carpool or solo driver. Wherever pos-

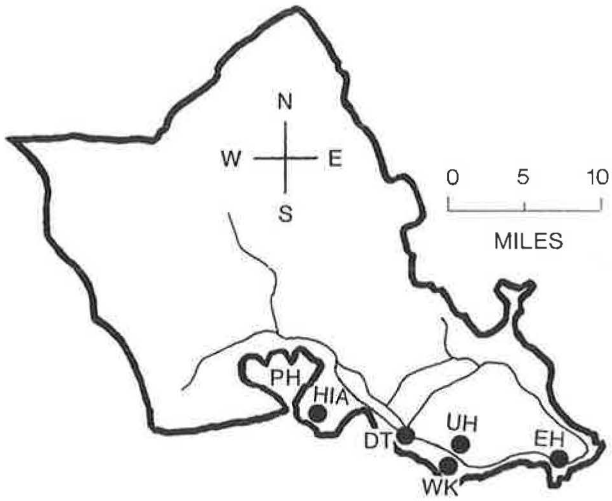

FIGURE 1 Primary highways on the island of Oahu and major work sites of commuters from the East Honolulu study area.

sible, the data were analyzed by factorial designs so that the effects of several independent variables could be examined simultaneously. Other parametric and nonparametric statistics were used as necessary.

RESULTS

Among those defined as commuters (that is, those living farther than 1 mi from work), 44.1 percent traveled roughly 10 mi from home to work in downtown, an area approximately 1 mi square that includes the capitol district, where most state and municipal offices are located. An additional 13.1 percent worked within 2 mi east or west of downtown (8 to 12 mi travel from East Honolulu), excluding Waikiki, part of which is within this 2-mi range. About 7.7 percent worked in Waikiki, a major tourist center southeast of downtown. Another 12.7 percent of commuters worked in the area east of Pearl Harbor that includes commercial and industrial establishments near the Honolulu International Airport. These work sites are 5 to 6 mi west of downtown, 15 mi or more from East Honolulu. Slightly more than 10 percent of the commuters traveled more than 20 mi each way to work.

Because of the topography of Oahu, all but 1.2 percent of the responding commuters made their daily commute along the same corridor into downtown, much of it on a single suburban arterial. Hence, in addition to the 44.1 percent working downtown, 29.1 percent of the commuters traveled the same route to get to jobs west of downtown, and another 19.2 percent traveled 7 mi or more with other inbound traffic each morning to reach job locations east of downtown. Given this situation, the geographical relationship of commuters' work sites with respect to downtown might be expected to have a more profound influence on commuter behavior than commute distance alone, although the two factors are closely related in this case (7).

Approximately 92.7 percent of commuters responding to the survey traveled to and from work by car, 6.4 percent took a bus, and fewer than 1 percent walked or rode a bicycle or motorcycle. Roughly 67.1 percent of all commuters said they drive alone, while 11.7 percent were in two-person carpools, 7.5 percent were in three-person carpools, and 6.4 percent were in carpools of more than three people. Nearly 80 percent (78.9 percent) of the people who carpool did so only with family members, another 12.9 percent were in carpools with people who are not family members, and the remaining carpoolers commuted with both family and nonfamily members.

Work Location and Mode Choice

No significant differences were found among modes with respect to commute distance. The average one-way travel distances of bus and car commuters were comparable (10.9 and 11.4 mi, respectively), and no differences in commute distance were found between carpools and solo drivers or among the different types of carpools (family, nonfamily, mixed). The location of work sites with respect to downtown, however, was found to exert a significant influence on choice of travel mode.

Generally, the closer people work to downtown, the greater their likelihood of riding the bus ($p < 0.001$). Only 4.4 percent of people that work 2 to 4 mi east of downtown commuted by bus, whereas 8.9 percent of people working within 2 mi of downtown were bus riders. Bus ridership was highest (12.0 percent) among those who work in downtown, but it was lowest (1.6 percent) among commuters working west of downtown.

A similar trend was found in carpooling. Among car commuters who worked more than 4 mi east of downtown, some 70.4 percent drove alone. This proportion decreased approaching downtown from the east, reaching a low of 67.9 percent solo drivers among car commuters who work downtown. Once past downtown, the percentage of solo drivers rose significantly again, to 77.7 percent ($p < 0.001$). No relationship was found between work location and types of carpools.

Transit Service

Clearly, for the commuters surveyed, mode choice was more a function of destination than of distance. The results further suggest that the decision to use transit is also a function of service. Among the commuters that rode the bus, 77.6 percent worked downtown and 9.4 percent worked at the University of Hawaii, the only two work sites having express bus service from East Honolulu. The sparsity of ridership among people working west of downtown may reflect the lack of express service to these work sites, the need to change buses to travel west of downtown on some bus routes, or both.

The importance of express bus service in the decision to use mass transit is made more evident by comparing bus ridership to the university, which is about 2 mi due east of downtown, with that to Waikiki, which stretches from about 1.5 to 3 mi southeast of downtown and does not have express bus service. Only 2.1 percent of the commuters surveyed traveled by bus to work in Waikiki ($p < 0.001$), even though the price of parking in Waikiki was about 35 percent higher than at the university (see section on parking costs). To put these percentages in perspective, only 1 out of every 55 respondents who commuted to Waikiki traveled by bus compared to 1 out of 8 commuters to the university. Only downtown Honolulu, itself, had a higher ratio of bus riders to commuters (1 out of 7).

Travel Time

Analysis of covariance, used to control for distance, revealed that automobile travel was 25 percent faster than bus travel ($p < 0.001$), according to respondents' estimates of their typical commuting times. Even though the majority of bus riders in the sample used express bus service, on average, a bus commuter traveled 1 mi in 4.4 min (approximately 13.6 mph), whereas a car commuter covered the same distance in 3.5 min (17.1 mph). The data do not permit separation of in-vehicle and out-of-vehicle travel time.

No direct relationship was found between travel distance and travel speed, but travel speed of car commuters was found to vary with respect to job location. The morning commute is slowest for people working downtown, and speed increases in proportion to the distance of job sites from downtown, in either direction. People who drove through downtown in the morning to get to job sites west of it had higher average speeds (25 to 35 mph, depending on distance) than those who drove to downtown (18 mph) or to areas within 2 mi east of downtown (18.5 mph).

The faster speeds of workers commuting through downtown in the morning is explained in part by the difference in home departure times of commuters. An inverse linear relationship was found between departure time and commute distance ($r = -0.26$, $p < 0.001$), with car commuters leaving 4 to 5 min earlier for each mile they have to travel. This suggests that people who worked west of downtown left earlier than other commuters in order to avoid traffic congestion. Presumably because of the slower speed of bus travel, bus riders left for work an average of 20 min earlier than car commuters ($p < 0.002$).

The distribution of departure times among car commuters in relation to work location with respect to downtown is shown in Table 1. On average, carpoolers left for work 20.4 min earlier than did solo drivers ($p < 0.003$). Further analyses showed that almost 43 percent of morning commuters traveling toward downtown left home early enough to avoid the peak-hour traffic east of downtown between roughly 7:00 and 8:00 a.m. About 39 percent, mainly those working east of downtown, traveled during peak westbound (that is, inbound) traffic, and the remaining 18 percent traveled after the peak hour.

Parking Costs

Only 37 percent of automobile commuters paid to park at or near their places of work, and in most areas of the island, 70 to 97 percent of workers parked free. In the major commercial districts of Waikiki and downtown Honolulu, however, about half of the car commuters paid for parking. Both the percentage of people who paid for parking and their average monthly cost for parking (the monthly price) were significantly higher in these two areas compared to all other work sites ($p < 0.001$).

As seen in Table 2, parking was one-third less expensive in areas adjacent to downtown (within 2 mi east or west of downtown), excluding Waikiki, and the percentage of car commuters that actually paid for parking was less than half that for downtown. Islandwide, the proportion of car commuters that paid for parking at work was inversely related to the distance of their work sites from downtown (biserial $r = -0.30$, $p < 0.001$). This relationship also holds for the price of parking ($r = -0.40$, $p < 0.001$)

Attitudes Toward TSM Strategies

The time of day that commuters traveled to work and the location of their work sites with respect to downtown each had significant effects on the attitudes of respondents toward various TSM strategies. Both of these effects were far more pronounced than were effects of respondents' mode of travel.

A three-way ANOVA on morning time of travel (before, after, or during the peak traffic hour), work location (east of, west of, or in downtown), and travel mode (bus, carpool, or solo driver) found that time of travel and work location, but not mode, had major effects on the proportion of commuters interested in having flextime or staggered work hours and in using express bus service. The results for work location and time of travel are presented in Tables 3 and 4, respectively. From 35 to 42 percent of commuters were interested in having flextime or staggered work hours on their jobs, depending on where they worked ($p < 0.05$) and the time they usually left

TABLE 1 AVERAGE MORNING DEPARTURE TIMES FOR SOLO DRIVERS AND CARPOOLERS

Job-Site Location	Solo Driver	Carpooler
> 2 Miles East of Downtown	7:23	6:57
< 2 Miles East of Downtown	7:12	6:48
Downtown Proper	6:52	6:39
< 2 Miles West of Downtown	6:43	6:30
> 2 Miles West of Downtown	6:25	6:24

TABLE 2 PERCENT OF CAR COMMUTERS PAYING FOR PARKING AND AVERAGE MONTHLY COST

Location	Percent	Mean	S.E.M.
Downtown	51.7	$61.09	± 1.45
Downtown ± 2 Miles [a]	21.9	$41.18	± 3.30
Waikiki	46.0	$47.30	± 2.19
All Other Sites	24.5	$25.44	± 1.40

[a] Downtown ± 2 Miles = 2 Miles East or West of Downtown.

TABLE 3 PERCENT OF COMMUTERS LIKELY TO USE VARIOUS TSM STRATEGIES BY JOB LOCATION

	Work Location		
TSM Strategy	East of Downtown	In Downtown	West of Downtown
Flex-Time/Staggered Hours	36.2	39.1	41.2 [a]
Park & Ride for Express Bus	23.3	29.0	21.0 [a]
Park & Ride for Carpooling	16.1	15.3	16.6
Non-Family Carpooling	15.8	15.5	17.0

[a] Significant difference across categories.

TABLE 4 PERCENT OF COMMUTERS LIKELY TO USE VARIOUS TSM STRATEGIES BY TIME OF TRAVEL

	Time of Travel		
TSM Strategy	Before Peak	During Peak	After Peak
Flex-Time/Staggered Hours	39.1	42.1	34.7 [a]
Park & Ride for Express Bus	26.2	25.1	19.7 [a]
Park & Ride for Carpooling	16.3	16.4	12.9
Non-Family Carpooling	17.6	17.1	11.0 [a]

[a] Significant difference across categories.

for work ($p < 0.01$). Somewhat more car commuters (41.0 percent) than bus riders (32.3 percent) were interested in alternative work schedules, and solo drivers (40.4 percent) showed slightly higher interest than carpoolers (37.2 percent), but these differences are not statistically significant.

Both commuters' time of travel ($p < 0.005$) and their work location ($p < 0.001$) had statistically significant effects on the likelihood of using park-and-ride facilities for express bus service, with downtown commuters and those traveling before or during the peak hour showing the greatest interest. This level of interest was attributable, in part, to those who were already bus riders (mainly downtown bus riders), 40.1 percent of whom said they were likely to use the facilities ($p < 0.005$) compared to 24.6 percent of solo drivers and 25.7 percent of carpoolers. Yet, even among car commuters ($p < 0.001$), those who worked downtown showed more interest (28.3 percent) than those working elsewhere (21.9 percent).

Lack of interest in using park-and-ride lots for carpooling was virtually universal, reflecting commuters' general resistance to carpooling with people from outside the family. Postpeak commuters showed significantly less interest than other commuters ($p < 0.05$) in park-and-ride lots. Nevertheless, car commuters recognized the potential time savings of high-occupancy-vehicle (HOV) lanes, and respondents in carpools with four or more people rated the value of HOV lanes quite highly.

Alternative Transit

The major purpose of this study was to determine potential demand for alternate modes of transportation that differ in service characteristics from normal bus service. Based on previous research (*1,4,8*), the two service characteristics of interest were a guaranteed seat and the distance of pickup and drop-off points from a commuter's points of origin and destination (referred to hereafter as access). The survey asked people to rate their likelihood of using alternative public transit or paratransit service, based on access, whether or not they were guaranteed a seat, and three hypothetical fares.

Time of travel, work location, and mode had significant but sometimes marginal effects on interest in using alternative transit or paratransit services. Overall, prepeak commuters were most likely to favor using such service ($p < 0.01$). Differences in interest between commuters traveling during and after peak were not statistically significant. No differences were found between downtown workers and those working west of downtown, and both of these groups were significantly more likely to use paratransit than people working east of downtown ($p < 0.05$).

Solo drivers and carpoolers appeared to be equally likely to use paratransit under all conditions posed. The primary effect of mode was for bus riders, who revealed an interaction between mode and fare and service characteristics. The percent of bus riders interested in paratransit exceeded that of automobile commuters only at a one-way fare of $1.00. Some 52 to 67 percent of bus riders were likely to use such service for a $1.00 fare if they were guaranteed a seat and access was comparable to current conditions. Potential ridership among bus commuters would increase to 74 percent if door-to-door service were offered at a $1.00 fare.

At a one-way fare of $2.00, interest among bus riders dropped to 20 to 29 percent even with a guaranteed seat (depending on access).

A $2.00 fare would be substantially more than the prevailing cost of commuting by bus for commuters who purchased a $15 monthly bus pass, which allows unlimited travel on Oahu's bus system. Pass holders were estimated to make an average of 2.9 daily weekday trips, so a $2.00 one-way fare would be more than seven times the current average trip cost of bus commuters, almost all of whom used monthly passes.

Although these findings, like those of other researchers (*2*), indicate that work site affects mode choice, this could reflect the importance of existing service conditions and may not apply to transit having different service characteristics. Other variables, such as commute distance, might exert greater influence if geographical biases, such as centering service around downtown commuters, were eliminated. Because commute distance can exert a significant influence on transit ridership (*2*), independent of work location, analysis of covariance was used to partition out or statistically remove the variance attributable to commute distance. Bus commuters were excluded from these and subsequent analyses because of their small number.

This exercise substantially reduced the effects of time of travel and work location on commuters' professed likelihood of using alternative transit. Commute distance significantly ($p < 0.002$) affected potential transit ridership of both carpoolers and solo drivers in a positive but nonlinear fashion. The percentage of automobile commuters (no reliable differences between solo drivers and carpoolers were found) likely to use paratransit was lowest (13 to 14 percent) among those traveling less than 5 miles each way to work. Interest jumped to 20 percent with commutes longer than 5 mi but increased only another 2 percent between 5 and 20 mi. At commute distances of more than 20 mi each way, interest in paratransit rose sharply again.

The effects of service characteristics and fare were more profound ($p < 0.001$ for each factor). Regardless of commute distance, a guaranteed seat increased potential ridership by roughly 7 percent, but each 5-min increase in access time (beyond door-to-door service) decreased prospective ridership 5 to 6 percent on average. Fare had the most powerful influence on commuters' likelihood of using paratransit, in that each $1.00 increase in fare produced a 12 to 13 percent decrease in potential ridership, all other things being equal. Significant two-factor and three-factor interactions among service variables and fare were found, however, implying that the effects of other things are neither equal nor inconsequential.

Various combinations of distance, service factors, and fare can produce extremely high or extremely low ridership, as seen in Table 5. For simplicity, Table 5 shows only three of the five commute distances used in these analyses because scores were relatively stable for commutes between 6 and 20 mi long (includes groups 6–10, 11–15, and 16–20 mi). Significance levels are not noted in the table because all main effects are significant.

The table reveals that trade-offs among service characteristics and fare can yield similar levels of ridership at different commute distances. Increases in service can compensate for losses in ridership that would occur with increases in fare. For example, at commute distances longer than 20 mi, 32.9 per-

TABLE 5 PERCENT OF AUTOMOBILE COMMUTERS LIKELY TO USE PARATRANSIT ON THE BASIS OF SERVICE CHARACTERISTICS, FARE, AND COMMUTE DISTANCE

Commute Distance [a]	Seating [b]	Access	One-Way Fare $1.00	$2.00	$3.00
		Door/Door	33.2	18.3	8.4
< 5	Seat	5 Minutes	27.2	14.0	7.3
		10 Minutes	18.9	10.9	4.8
		Door/Door	24.6	13.0	5.0
< 5	No Seat	5 Minutes	20.5	10.5	4.7
		10 Minutes	13.3	7.8	4.3
		Door/Door	47.8	27.7	13.3
11-15	Seat	5 Minutes	41.1	22.0	9.3
		10 Minutes	30.8	15.7	6.4
		Door/Door	36.0	19.6	8.9
11-15	No Seat	5 Minutes	30.1	14.8	5.8
		10 Minutes	21.2	10.1	4.4
		Door/Door	50.0	33.3	20.3
> 20	Seat	5 Minutes	44.3	29.0	16.3
		10 Minutes	32.9	20.9	13.1
		Door/Door	39.7	25.9	14.3
> 20	No Seat	5 Minutes	34.8	21.5	11.1
		10 Minutes	24.5	14.0	7.7

[a] in Miles

[b] Seat = Guaranteed Seat; No Seat = No Guarantee of Seat.

cent of automobile commuters were willing to walk 10 min to board paratransit at a $1.00 fare, if they were guaranteed a seat. A similar percentage (34.8 percent) were willing to walk only 5 min at the same fare if they were not guaranteed a seat. At a $2.00 fare, both door-to-door service and a guaranteed seat would be required to attract roughly the same proportion (33.3 percent) of people commuting more than 20 mi. A comparable level of demand (33.2 percent) among people commuting less than 5 mi is achieved only with door-to-door service, a guaranteed seat, and a $1.00 fare.

Although commute time is related to commute distance, the correlation between the two was relatively low ($r = 0.29$) and the shared variance between these two factors varied from 3.6 to 13.6 percent, depending on mode. Because the time a commute takes to complete also encompasses some of the effects of the time of travel relative to the peak hour and mode, it would seem to be a potentially important variable affecting the likelihood of using alternative transit. Commute time produced a pattern of interest in paratransit quite similar to that found for commute distance: (a) low interest among those with commute times faster than 20 min, (b) a steep rise in interest among people whose commuting time was between 20 min and 30 to 40 min, (c) a gradual increase in interest up to 40 to 50 min, and (d) a second sharp increase among those commuting longer than 50 min. In part, the similar pattern of results produced by commute time and commute distance may reflect the correlation between them, although, as noted, the correlation was not high.

The relationships between commute time and the other variables tested (fare, access, and seating) also mirror those found for commute distance, and significant main effects of all variables and interactions among all variables were found. Despite these commonalities, interest in paratransit was greater at the highest levels of commute time than it was for commute distance, suggesting that travel time is more important than distance alone in attracting ridership. A more important difference between time and distance effects is that commute time has a significant interaction with seating that commute distance does not. Commute time is particularly sensitive to the value of a seat, and the value of a guaranteed seat increases systematically with commute time. Table 6 reveals how trade-offs among fare and service characteristics result in comparable levels of ridership at different commute times.

CONCLUSIONS

The results of the survey indicate a low rate of vehicle occupancy among commuters from the far eastern suburbs of Oahu. The vast majority of the people surveyed commuted by car, and the great bulk of these drove alone. Given these data, any efforts to increase vehicle occupancy seem worthwhile.

Only a small percentage of commuters were willing to carpool with people outside their own families, however (*4,7*). The existence of a parking facility that provides a central meeting place for carpoolers appears to offer virtually no incentive to carpool. Solo drivers were not interested in using these facilities, nor were current carpoolers, most of whom commute with family members. Even though people see the advantage of HOV lanes, the existing HOV/contraflow lane in Honolulu covers only a short distance (*9*). Apparently for most people, the time savings these lanes provide are outweighed by the burden of commuting with nonfamily members.

TABLE 6 PERCENT OF AUTOMOBILE COMMUTERS LIKELY TO USE PARATRANSIT ON THE BASIS OF SERVICE CHARACTERISTICS, FARE, AND COMMUTE TIME

Commute Time [a]	Seating [b]	Access	One-Way Fare $1.00	$2.00	$3.00
		Door/Door	25.5	17.0	12.1
< 20	Seat	5 Minutes	21.0	14.2	9.1
		10 Minutes	17.4	11.0	6.9
		Door/Door	21.4	13.7	8.3
< 20	No Seat	5 Minutes	18.7	11.4	5.2
		10 Minutes	12.8	7.9	3.9
		Door/Door	49.0	28.7	14.2
30-40	Seat	5 Minutes	41.9	23.1	11.0
		10 Minutes	31.1	17.0	8.0
		Door/Door	37.9	20.4	9.4
30-40	No Seat	5 Minutes	31.4	15.5	6.5
		10 Minutes	22.6	10.9	4.9
		Door/Door	49.7	31.6	17.6
> 50	Seat	5 Minutes	43.1	26.2	13.4
		10 Minutes	34.5	21.6	11.4
		Door/Door	38.1	22.8	12.5
> 50	No Seat	5 Minutes	32.3	18.9	8.8
		10 Minutes	24.0	13.2	6.7

[a] in Minutes

[b] Seat = Guaranteed Seat; No Seat = No Guarantee of Seat.

Although interest in flextime or staggered working hours was high among various categories of commuters, morning departure times are distributed so widely already that any rigid institutional regimen of staggered hours is likely to cause considerable conflict with the usual travel arrangements of many commuters, as was found during the staggered working hours demonstration project with municipal, state, and other employees working in downtown Honolulu (*10*). Any such approach must be truly flexible, allowing commuters to adjust their departure times as they see fit.

Mode of travel had a significant effect on departure times, and departure times (presumably reflecting work schedules, with adjustments for other external influences) had significant effects on interest in using different modes of travel. Acting concurrently with these influences, and presumably interacting with them to some degree, work location with respect to downtown also affected departure time, mode choice, and the likelihood of using available and potential transportation alternatives. These relationships probably reflect the fact that work location itself is related to a number of factors, including parking costs, commute distance, traffic congestion, and job density, that themselves provide incentives and disincentives for using various modes and that may limit potential carpool mates (*2*). Offered a transit alternative free of existing operating conditions and constraints, such as transit's focus on downtown travelers, commuter interest was strongly influenced by basic factors such as commute time, service, and fare. Correspondingly, the effects of mode, time of travel (with respect to the peak), and work location appear to have less influence on interest in transit.

Potential ridership appears to grow with increases in commute time and distance. The influence of commute time in determining potential ridership would seem to be particularly important for providers of paratransit services because it implies that interest in transit that offers improved levels of service will rise as traffic congestion worsens. Improved service characteristics, such as guaranteed seating, significantly enhance ridership (*4,7*). Guaranteed seating augmented potential ridership at all access times and could compensate for increased access times, at least up to 5 min. Beyond this distance, the time spent walking to pickup and drop-off points partially outweighed the value of a seat, but this trade-off depended upon commute time. For commuters traveling more than 50 min, for example, walking 10 min to boarding points to get a guaranteed seat was valued about as highly as having door-to-door service without guaranteed seating.

Of course, fare had a major effect on the likelihood that the commuters in this sample would use paratransit. All things being equal, potential ridership was affected more by fare than it was by either service variable studied, although this finding may not hold generally (*1*). Tables 5 and 6 make apparent the ability of the combined, positive effects of door-to-door service and guaranteed seating to overcome the negative effects of fare on potential ridership. Furthermore, the combined effects of these service variables become more marked as commute time increases.

Can the type of service needed to attract alternative transit or paratransit ridership be profitable? The market appears to be strong enough to support a profit-making enterprise that can avoid the limitations and pitfalls of traditional transit operations (*11–13*). Existing computer technology seems to offer the solution to providing such service at low cost.

The innovative but inefficient dial-a-ride concept, which was first introduced in the 1960s (*14,15*), should be combined with current computer technology to provide a real-time, demand-response transit alternative (*16–18*). A computer network, enabling communication between consumer and transit operators through a centralized system, could improve service at low cost. This network could not only offer immediate access to information about alternative transportation services but also would enable users to book a ride for any time, at any time, from the services available, on a trip-by-trip basis. Fares could be billed monthly by the central computer, saving on accounting costs (*16,17*).

Access to the system need not be limited to consumers and operators of present day alternative transit or paratransit systems. Because the network would connect homes to a central computer, people seeking a ride could match their travel plans with those of people offering a ride, by posting appropriate information on an electronic bulletin board. This system would permit single-trip carpooling in its most convenient form—dynamic ridesharing, without the time and information costs or personal commitment that often deter people from joining permanent carpools (*5,19,20*).

State and municipal governments should take measures to encourage varied forms of paratransit and make efforts to integrate information and transportation services.

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Estimating Transportation Corridor Mobility

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This report summarizes an investigation of some methods of quantifying peak-hour person- and vehicle-movement for different travel modes in major transportation corridors. Several procedures for estimating freeway, high-occupancy-vehicle (HOV) lane, and rail transit line operation are identified. These procedures are evaluated as to their data requirements, reasonableness of results, and ability to produce intuitively correct conclusions. The recommended equations allow comparison of peak-hour operation of freeway main lanes and adjacent HOV lanes or rail transit lines to estimate how much high-capacity, high-speed transportation alternatives increase person-movement.

Roadways, transit routes, and special transportation facilities are designed to provide maximum traffic flow at acceptable levels of service (LOS) during peak travel periods, that is, to address person-movement needs. Freeway transit facilities and high-occupancy-vehicle (HOV) treatments represent strategies to address congestion problems. Individual projects work together to provide a system of transportation facilities.

In many urban travel corridors, peak-period travel demand is too great to be accommodated without congestion for 2 or 3 hr during each peak period. In extreme examples, a freeway may operate only slightly better during the remainder of the daylight hours.

Roadway project evaluation has emphasized peak-hour and peak-period vehicle operating conditions. Of growing importance, however, is the potential for increased passenger movement in major travel corridors. Increasing bus and private-vehicle occupancy rates, and therefore person-movement capacity, has become possible using priority treatment techniques. Analytical procedures should measure how much these HOV treatment techniques contribute to the total person-movement capacity of a corridor.

Several peak-hour travel condition indicators are applied to major Texas urban freeways. Several mobility estimation procedures are analyzed for their applicability to peak-hour person-movement. The investigation was based on peak-hour freeway and HOV lane operating data. Analysis techniques focusing on peak-hour operation are consistent with other accepted highway and street evaluation procedures such as in the *Highway Capacity Manual* (*1*). The concepts involved in peak-hour traffic and transit operation are also much easier to quantify than those associated with peak periods, and more data are available on them. Peak-period operation, especially in situations in which congestion reduces travel speeds for 2 or 3 hr in each peak, is also an important comparative measure of corridor mobility.

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CANDIDATE CONGESTION MEASURES

Several methodologies are useful for relating traffic volume, person-movement, and travel time to congestion in major travel corridors. Peak-hour congestion measurement procedures can be demonstrated using data from existing busway and HOV lane projects throughout the United States and Canada. The priority lane and mixed-flow facility characteristics and operating statistics are presented in Tables 1 and 2. The Ottawa and Pittsburgh lanes are bus-only facilities in separate rights-of-way with no mixed-flow facility immediately adjacent. The data in Tables 1 and 2 were derived from a 1985 ITE survey (*2*). The operating statistics and some of the facility designs have changed, but they provide a wide range of project types and vehicle and person volumes with which to illustrate the application of various methodologies.

Person-Movement on Freeways and HOV Lanes

The usual way to measure person-movement on HOV lanes is to compare the number of people in priority lanes with that in mixed-flow lanes. A standard used to evaluate HOV lanes with this measurement is that if a HOV lane carries more people in the peak hour than an average freeway lane, the priority treatment is considered to be an improvement. This measure is an estimate of how well roadway supply is being used to provide person-movement.

The data presented in Table 3 compare the number of people carried at the peak hour in freeway lanes and in HOV lane projects in North America. Many of these HOV projects are adjacent to mixed-flow freeway lanes and, therefore, are subject to constant public scrutiny. Figure 1 shows how these data are typically presented. All of the freeway projects, with the exception of the Katy Freeway with carpools of three or more (3+) persons, have more than one freeway lane of people in the HOV lane during the peak hour. Public perception of the Katy Freeway HOV 3+ lane as an underused facility resulted in a lowering of the occupancy requirement to HOV 2+, and a commensurate increase to 2.4 freeway lanes of persons in the HOV lane. The Bay Bridge and Route 495 contraflow lane (Lincoln Tunnel approach) permit bypassing a toll plaza. The average mixed-flow traffic volume on those projects is relatively low and a significant number of buses use each project.

TABLE 1 PHYSICAL DESCRIPTION OF OPERATING TRANSITWAY FACILITIES, 1985 DATA

HOV Project and Location	Number of Lanes		Length (mi.)	Eligible Vehicles
	HOV	Frwy		
Exclusive in Separate R.O.W.				
Ottawa, Canada				
Southeast Transitway	1/direction	NA	1.5	Bus
West Transitway	1/direction	NA	2.9	Bus
Southwest Transitway	1/direction	NA	1.9	Bus
Pittsburgh, PA				
East Busway	1/direction	NA	6.8	Bus
South Busway	1/direction	NA	3.5	Bus
Facilities in Freeway R.O.W.				
Exclusive Facilities				
Houston, Texas				
I-10 (Katy) (1985)	1(reversible)	3	6.2	Bus, 3+
I-10 (Katy) (1988)[1]	1(reversible)	3	13.2	Bus, 2+
I-45 (North)	1(reversible)	3	9.6[2]	Bus, 8+
Los Angeles, I-10 (San Bernardino Fwy)	1/direction	4	11.0	Bus, 3+
Washington, D.C.				
I-395 (Shirley)	2(reversible)	4	11.0	Bus, 4+
I-66	2/direction	NA	9.6	Bus, 3+
Concurrent Flow				
Los Angeles, Route 91	1(EB only)	4	8.0	Bus, 2+
Miami, I-95	1/direction	3	7.5	Bus, 2+
Orange County, Route 55	1/direction	3	11.0	Bus, 2+
San Francisco, CA				
Bay Bridge	3(WB only)[3]	16[3]	0.9	Bus, 3+
US 101	1/direction	3	3.7	Bus, 3+
Seattle, WA				
I-5	1/direction	4	5.6	Bus, 3+
SR 520	1 (WB only)	2	3.0	Bus, 3+
Contraflow				
New York City, NJ, Rte. 495	1	3	2.5	Bus
San Francisco, CA, US 101	1	4	4.2	Bus

Source: Reference 2
NA - Not Applicable
R.O.W. - Right-of-Way

[1]Katy Transitway began operation with two-or-more person (2+) carpools in August 1986
[2]In the morning a 3.2-mile concurrent flow lane is also in operation (total HOV length = 12.8 mi.)
[3]Number of lanes at the toll plaza

Speed of Person-Volume (SPV)

Comparing person throughput on a freeway lane and HOV lane describes the relative (peak-hour) volume but does not necessarily estimate the effect of travel speed. To address this factor, the product of speed and person-volume per lane has been used to estimate the relative benefit of HOV lanes and freeway main lanes (*2*). Although the person-volume on freeways is generally related to vehicle-volume (assuming relatively constant vehicle occupancy rates for freeways in most North American cities), HOV lanes carry differing types of vehicles and varying numbers of occupants. A HOV lane with 2,000 peak-hour vehicles, each carrying two people, will move the same number of people as 100 buses with 40 passengers each. The LOS for these lanes will be significantly different, however.

One measure of LOS for roadway passengers takes into account both vehicle speed and person-volume. Multiplying speed by volume per lane, rather than total person-volume, more accurately describes the travel conditions for HOV and general-purpose lanes. This equation is as follows:

$$\text{SPV} = \text{Travel Speed (mph)} \times \text{Peak-Hour Person-Volume per Lane} \quad (1)$$

Weighting each of the facilities by the total number of people experiencing each condition yields a value for the corridor roadway system.

$$\text{SPV}_{\text{Corr}} = \frac{\text{SPV}_{\text{HOV}} \times \begin{matrix}\text{HOV Peak-Hour}\\ \text{Person-Volume}\end{matrix} + \text{SPV}_{\text{Fwy}} \times \begin{matrix}\text{Freeway Peak-Hour}\\ \text{Person-Volume}\end{matrix}}{\text{(Freeway + HOV) Peak-Hour Person-Volume}} \quad (2)$$

TABLE 2 PEAK-HOUR, PEAK-DIRECTION HOV LANE OPERATING CHARACTERISTICS

HOV Project and Location	Average Peak-Hour Volume[1]						Average Speed(mph)[1]	
	Bus		Van & Carpool		Freeway			
	Vehicle	Person	Vehicle	Person	Vehicle	Person	HOV Lane	Freeway
Exclusive in Separate R.O.W.								
Ottawa, Canada								
Southeast Transitway & Central Area Transitway	270	7,650	NA	NA	NA	NA	45	NA
West Transitway	135	6,800	NA	NA	NA	NA	29	NA
Southwest Transitway	125	4,250	NA	NA	NA	NA	29	NA
Pittsburgh, PA								
East Busway	105	4,895	NA	NA	NA	NA	31	NA
South Busway	75	2,785	NA	NA	NA	NA	26	NA
Facilities in Freeway R.O.W.								
Exclusive Facilities Houston, Tx.								
I-10 (Katy) 3+ HOVs	35	1,200	90	510	4,660	5,420	53	29
I-10 (Katy) 2+ HOVs	35	1,190	1,330	2,715	4,650	4,930	47	35
I-45 (North)	70	2,555	180	1,450	4,375	5,050	58	24
Los Angeles, I-10 (San Bern)	75	3,320	835	2,735	8,210	10,335	55	24
Washington D.C.								
I-395 (Shirley)	155	5,425	1,575	7,500	6,625	8,525	57	26
I-66	80	2,765	1,910	7,510	NA	NA	58	NA
Concurrent Flow								
Los Angeles, Route 91	20	500	1,370	3,050	8,000	8,960	53	27
Miami, I-95	10	350	1,335	2,400	5,850	7,240	50	39
Orange County, Route 55	5	80	1,250	2,730	6,100	6,710	60	31
San Francisco, CA								
Bay Bridge	195	6,505	1,945	7,940	6,655	7,900	22	5
US 101	80	2,785	305	940	5,875	8,990	56	37
Seattle, WA								
I-5	45	1,820	395	1,190	7,500	9,000	34	26
SR 520	55	2,300	255	1,060	3,485	3,905	16	7
Contraflow								
New York City, NJ, Rte. 495	725	34,685	NA	NA	4,475	7,380	21	4
San Francisco, CA, US 101	150	6,000	NA	NA	7,000	9,450	50	50

Source: Reference 2
NA - Not Applicable ND - No Data Provided
[1]Values are the average of morning and evening peak-hour where applicable

The HOV lane and freeway speed of person-volume (SPV) values are shown in Table 4. The highest HOV values are those for the Route 495 and the Shirley Highway HOV lanes. The corridor SPV values for these facilities and other HOV projects are significantly higher than the freeway SPV values. Exclusive facilities, both in separate rights-of-way and within freeway corridors, generally have higher HOV SPV values than concurrent-flow lanes. This attribute is consistent with the expectations of HOV priority treatments that require significant capital investment.

Most of the freeway values are between 40,000 and 70,000, which is consistent with average speeds of 20 to 30 mph and person-volumes of 1,500 to 2,500 per lane. In general, higher SPV values are possible with higher occupancy requirements on HOV lanes, because operating capacity is defined by vehicular volume. In the case of the Katy Freeway, however, decreasing the minimum vehicle occupancy for HOV lane eligibility increased person movement. With three or more occupants required on the HOV lane, the corridor SPV value was only 17 percent greater than the freeway value. When two-person carpools were allowed on the HOV lane, the SPV for the corridor became 95 percent greater than the freeway value.

Person-Movement Index (PMI)

Another easily calculating, yet descriptive, measure of person-movement is the person-movement index (PMI) (*3*), also described as the rate of person-movement (*4*). The PMI, defined as the product of vehicle occupancy and speed, is calculated as follows:

$$\text{PMI} = \begin{array}{c}\text{Peak-Hour}\\ \text{Vehicle Occupancy}\\ \text{(persons per vehicle)}\end{array} \times \begin{array}{c}\text{Peak-Hour}\\ \text{Travel Speed (mph)}\end{array} \qquad (3)$$

A higher vehicle occupancy rate or greater travel speed will yield a higher PMI value. As in the SPV calculation, weighting the freeway and HOV lane PMI values by the number of people each facility carries provides an estimate of the corridor system effectiveness. Thus,

$$\text{PMI}_{\text{Corr}} = \frac{\text{PMI}_{\text{HOV}} \times \begin{array}{c}\text{Peak-Hour HOV}\\ \text{Person-Volume}\end{array} + \text{PMI}_{\text{Fwy}} \times \begin{array}{c}\text{Peak-Hour Freeway}\\ \text{Person-Volume}\end{array}}{\text{(Freeway + HOV) Peak-Hour Person-Volume}} \qquad (4)$$

TABLE 3 PEAK-HOUR FREEWAY AND HOV LANE PERSON-VOLUME COMPARISON

HOV Project and Location	Average Peak-Hour Person Volume		Person Volume Per Lane		Ratio of HOV Lanes to Freeway Lane Person Volume
	HOV Lane	Freeway	HOV Lane	Freeway	
EXCLUSIVE IN SEPARATE R.O.W.					
Ottawa, Canada					
Southwest Transitway & Central Area Transitway	7,650	NA	7,650	NA	NA
West Transitway	6,800	NA	6,800	NA	NA
Southwest Transitway	4,250	NA	4,250	NA	NA
Pittsburgh, PA					
East Busway	4,895	NA	4,895	NA	NA
South Busway	2,785	NA	2,785	NA	NA
FACILITIES IN FREEWAY R.O.W.					
Exclusive Facilities					
Houston, Texas					
I-10 (Katy) 3+ HOVs	1,710	5,420	1,710	1,805	.95
I-10 (Katy) 2+ HOVs	3,900	4,930	3,900	1,645	2.37
I-45 (North)	4,005	5,050	4,005	1,685	2.38
Los Angeles, I-10 (San Bern)	6,055	10,335	6,055	2,585	2.34
Washington D.C.					
I-395 (Shirley)	12,925	8,525	6,465	2,130	3.03
I-66	10,275	NA	5,138	NA	NA
Concurrent Flow					
Los Angeles, Route 91	3,550	8,960	3,550	2,240	1.58
Miami, I-95	2,750	7,240	2,750	2,415	1.14
Orange County, Route 55	2,810	6,710	2,810	2,235	1.26
San Francisco, CA					
Bay Bridge	14,445	7,900	4,815	495	9.75
US 101	3,725	8,990	3,725	2,995	1.24
Seattle, WA					
I-5	3,010	9,000	3,010	2,250	1.34
SR 520	3,360	3,905	3,360	1,955	1.72
Contraflow					
New York City, NJ, Rte. 495	34,685	7,380	34,685	2,460	14.10
San Francisco, CA, US 101	6,000	9,450	6,000	2,365	2.54

Source: Reference 2
NA - Not Applicable

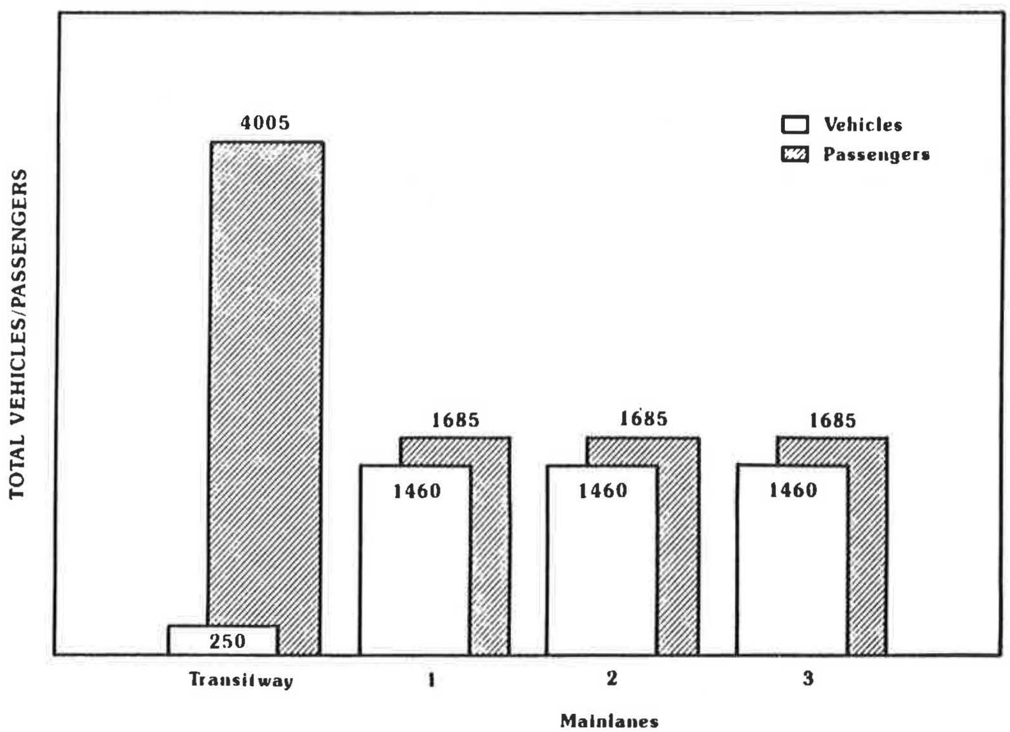

FIGURE 1 Average peak-hour person- and vehicle-volume on North Freeway (I-45) main lanes and transitway.

TABLE 4 SPEED OF PERSON-VOLUME VALUES FOR HOV LANES AND FREEWAYS

HOV Project and Location	Peak-Hour Person Volume Per Lane		Speed of Person Volume			Percent Increase Corridor vs Fwy[3]
	HOV Lane	Freeway	HOV[1] 1000	Freeway[1] 1000	Corridor[2] 1000	
Ottawa, Canada						
Southeast & Central Area Transitway	7,650	NA	344	NA	344	NA
West Transitway	6,800	NA	197	NA	197	NA
Southwest Transitway	4,250	NA	121	NA	121	NA
Pittsburgh, PA						
East Busway	4,895	NA	154	NA	154	NA
South Busway	2,785	NA	73	NA	73	NA
Exclusive Facilities						
Houston, Texas						
I-10 (Katy) 3+ HOVs	1,710	1,805	91	52	61	20
I-10 (Katy) 2+ HOVs	3,900	1,645	182	58	113	95
I-45 (North)	4,005	1,685	231	40	125	210
Los Angeles, I-10 (San Bern)	6,055	2,585	333	63	163	160
Washington D.C.						
I-395 (Shirley)	6,465	2,130	371	55	245	345
I-66	5,140	NA	296	NA	296	NA
Concurrent Flow						
Los Angeles, Route 91	3,550	2,240	189	60	97	60
Miami, I-95	2,750	2,415	138	94	106	15
Orange County, Route 55	2,810	2,235	169	69	98	45
San Francisco, CA						
Bay Bridge	4,815	495	104	3	68	2,455
US 101	3,725	2,995	207	111	139	25
Seattle, WA						
I-5	3,010	2,250	101	58	69	20
SR 520	3,360	1,955	55	13	32	150
Contraflow						
New York City, NJ, Rte. 495	34,685	2,460	743	11	615	5,730
San Francisco, CA, US 101	6,000	2,365	302	119	190	60

Source: Reference 2
NA - Not Applicable
ND - No Data Provided
[1]See Equation 1 [2]See Equation 2 [3]Represents difference between corridor SPV and freeway SPV

Table 5 presents PMI values for the freeway, HOV lanes, and total corridor examples. The bus-only facilities in Ottawa, Pittsburgh, and New York City have high PMI values because of the relatively high occupancy rates achieved without carpools. The Katy HOV 3+ and North Freeway transitways in Houston also had limited carpool use and, therefore, relatively high PMI values. Eight of the freeway PMI values are between 25 and 40, reflecting low main-lane vehicle occupancy rates and traffic speeds. HOV lanes are rarely successful if the freeway main lanes are uncongested, and vehicle occupancy rates are not significantly different in most major urban areas.

The conclusions derived from the corridor PMI calculation are somewhat counterintuitive. Allowing two-person carpools on the Katy transitway significantly increased total HOV person-movement but also decreased the average HOV vehicle occupancy ratio by 80 percent. The PMI values for both the HOV 2+ lane and the total system were significantly lower than those for HOV 3+ operation, indicating a decrease in project effectiveness. Because peak-hour person-movement increased 25 percent with no significant reduction in speed, however, the Katy transitway was more successful at moving people during the peak hour as a HOV 2+ project than as a HOV 3+ lane. When the shift to 2+ was made, motorists perceived the Katy transitway as underused (*5*). Apparently, some threshold vehicle-volume is necessary for a HOV project to seem useful; once above that level, more detailed analysis tools may be applied.

EVALUATION OF MOBILITY MEASUREMENT METHODOLOGIES

The freeway and HOV lane operational measures summarized use a variety of inputs but have in common the relative availability of data. Each has its advantages and limitations. The mixed-flow and HOV lane person-volume statistic (Table 3) is easy to calculate and illustrates a key benefit of HOV priority treatments—increasing the person-movement capability of a freeway or arterial corridor. The concept is also relatively easy to illustrate, as shown in Figure 1, and to explain to the general public. This benefit should not be overlooked; the success or failure of many priority treatment projects has been determined by the public perception of HOV lane use rates. Particularly in the case of concurrent (no barrier separation) flow lanes, the appearance of a relatively unused lane and easy convertibility from priority to mixed-flow vehicle usage requires a marketing effort to encourage use.

SPV values combine the two most significant performance measures of HOV lane operation (Table 4). Increased person-

TABLE 5 PMI VALUES FOR HOV LANES AND FREEWAYS

HOV Project and Location	Person Movement Index			Percent Increase Corridor vs Frwy[3]
	HOV Lane[1]	Freeway	Corridor[2]	
EXCLUSIVE IN SEPARATE R.O.W.				
Ottawa, Canada				
Southeast Transitway & Central Area Transitway	1,275	NA	1,275	NA
West Transitway	1,461	NA	1,461	NA
Southwest Transitway	969	NA	969	NA
Pittsburgh, PA				
East Busway	1,499	NA	1,499	NA
South Busway	1,008	NA	1,008	NA
FACILITIES IN FREEWAY R.O.W.				
Exclusive Facilities				
Houston, Texas				
I-10 (Katy) 3+ HOVs	726	33	199	500
I-10 (Katy) 2+ HOVs	133	37	80	115
I-45 (North)	932	28	428	1,445
Los Angeles, I-10 (San Bern)	367	31	155	405
Washington D.C.				
I-395 (Shirley)	429	33	272	715
I-66	298	NA	298	NA
Concurrent Flow				
Los Angeles, Route 91	136	30	60	100
Miami, I-95	102	48	63	30
Orange County, Route 55	135	34	64	90
San Francisco, CA				
Bay Bridge	146	6	97	1,410
US 101	537	57	197	250
Seattle, WA				
I-5	230	31	81	160
SR 520	177	7	86	1,050
Contraflow				
New York City, NJ Rte. 495	1,025	7	847	11,880
San Francisco, CA, US 101	2,016	68	825	1,110

Source: Reference 1

NA - Not Applicable
ND - No Data Provided

[1]See Equation 3
[2]See Equation 4
[3]Represents difference between total PMI and freeway PMI

movement at significantly higher speeds (relative to the mixed-flow lanes) is the purpose of designating HOV lanes, and the SPV measure directly quantifies this result. Combining the SPV values both for the freeways and HOV lanes into a total corridor measure provides a basis for determining the effect of priority treatment projects. Higher passenger volume or greater speed, or both, will raise the SPV value. The SPV formula is applicable both to mixed-flow and to priority treatment projects, with identical data requirements for each. The results are directly comparable and easier to explain than indicators based on different formulas. The values resulting from this calculation, however, are large (tens of thousands) and may be difficult for the public to understand. Also, they are not easy to compare with other measures.

Vehicle occupancy rate and vehicle speed are combined in PMI. This calculation is as uncomplicated as the SPV formula and may be somewhat easier to understand. HOV PMI values are significantly higher than freeway main-lane PMI values. PMI values for the two facilities can be combined to form a corridor PMI value to indicate HOV lane impact. Increasing person-movement by reducing the HOV minimum occupancy requirement, however, decreases the PMI value. As was indicated in Table 5, this counterintuitive relationship (PMI value is lower, even though the overall travel situation improves) is also apparent in the corridor PMI value. For example, total peak-hour person-movement on the Katy transitway increased from 1,710 (with HOV 3+) to 3,900 (with HOV 2+), indicating an improvement, but the PMI value decreased 80 percent. This large decrease was not offset by the increased person-movement (used to weight the freeway and HOV PMI values), and the corridor PMI decreased 60 percent. Weighting the PMI values by person-volume per lane would provide a more intuitively correct increase in the total PMI value but would not indicate the average travel condition for all commuters on both facilities.

RECOMMENDED MOBILITY MEASUREMENT PROCEDURE

Analytical procedures transportation professionals use to assess peak-hour operating conditions on streets and freeways typically focus on vehicle-volume and speed. The *Highway Capacity Manual* (*1*) and almost all other methodologies examine the flow of vehicles, because the physical limitations of capacity are related to vehicle characteristics and volume. To compare priority treatment techniques and mixed-flow freeway lanes, however, person-movement is more appropriate. HOV priority lanes operate at significantly higher speeds than mixed-flow lanes. This advantage can be incorporated into a methodology that can illustrate the relative effectiveness of mixed-flow and HOV lanes.

Peak-Hour Mobility Estimation Methodology

The SPV calculation offers the best combination of ease of data collection, applicability to both mixed-flow and HOV lane operation, and ability to reflect the effects of new conditions such as changes in minimum occupancy rules. The most negative feature of the calculations is that it produces values that are relatively large (typically greater than 40,000) and are not related to standard quantities such as those used in the *Highway Capacity Manual* (*1*). Thus, they may not be readily understood by transportation professionals or the general public. A par value could be used to normalize the results of individual equation elements so as to indicate congested freeways more clearly.

$$\begin{array}{l}\text{Par Value} \\ \text{for Freeway SPV} \\ \text{Calculation}\end{array} = 45\text{ mph} \times \begin{array}{l}\text{1,850 Vehicles} \\ \text{per Lane in the} \\ \text{Peak Hour}\end{array} \times \begin{array}{l}\text{1.2 Persons} \\ \text{per Vehicle}\end{array}$$

$$= 99{,}900 \text{ (use 100,000)}$$

The speed and volume values represent freeway operating conditions at the beginning of LOS E (*1*). Peak-hour LOS E or F operation represents significant travel delay and also is frequently associated with delay during other hours. Operation of mixed-flow freeway lanes at LOS E has been acknowledged as a general warranting condition for establishing HOV lanes (*6*).

A similar par value was generated to evaluate arterial street HOV lanes. Using the *Highway Capacity Manual* (*1*) value for signalized intersection delay at LOS E, an uncongested arterial vehicle speed of 35 mph, and an arterial street spacing of 1 mi, an LOS E speed of 25 mph was estimated, as follows:

$$\text{LOS E Stopped Delay} \times 1.3 = \begin{array}{l}\text{LOS E Total Delay per} \\ \text{Intersection}\end{array}$$

$$40\text{ sec} \times 1.3 = 52\text{ sec (0.9 min)}$$

$$\text{1 mi Street Spacing} \div 35\text{ mph} = \text{1.7 min Operating Time}$$

$$\text{1.7 min Operating Time} + \text{0.9 min of Delay} = \begin{array}{l}\text{2.6 min Total} \\ \text{Travel Time}\end{array}$$

$$\text{1 mi Street Spacing} \div \text{2.6 min Total Travel Time} = \begin{array}{l}\text{23 mph} \\ \text{(use 25 mph)}\end{array}$$

The planning analysis criteria in the *Highway Capacity Manual* (*1*) identify 1,200 to 1,400 veh/hr as the range that represents near-capacity conditions. A 50 percent green time value was assigned to the average of that volume (1,300 veh/hr) to estimate peak-hour LOS E traffic volume on an arterial. (The limiting condition for arterial street capacity is at the intersection of two principal arterials; each arterial would be expected, for planning purposes, to require 50 percent of the green time. This calculation is as follows:

$$\begin{array}{l}\text{Par Value for} \\ \text{Arterial SPV} \\ \text{Calculation}\end{array} = 25\text{ mph} \times \begin{array}{l}\text{1,300 Vehicles per} \\ \text{Lane in the} \\ \text{Peak Hour}\end{array} \times \begin{array}{l}\text{50 Percent} \\ \text{Green Time}\end{array} \times \begin{array}{l}\text{1.2 Persons per} \\ \text{Vehicle}\end{array}$$

$$= 19{,}500 \text{ (use 20,000)}$$

Corridor Mobility Index

The par values for freeway and arterial operation can be combined with the SPV calculation (Equations 1 and 2) to generate a corridor mobility index (CMI). For high-speed HOV lanes and rail transit lines:

$$CMI_F = \frac{\begin{array}{l}\text{Travel} \\ \text{Speed (mph)}\end{array} \times \begin{array}{l}\text{Peak-Hour Person} \\ \text{Volume Per Lane}\end{array}}{100{,}000} \quad (5)$$

For arterial street HOV Lanes:

$$CMI_A = \frac{\begin{array}{l}\text{Travel} \\ \text{Speed (mph)}\end{array} \times \begin{array}{l}\text{Peak-Hour Person} \\ \text{Volume Per Lane}\end{array}}{20{,}000} \quad (6)$$

The high-speed equation applies to HOV lanes within or adjacent to freeways, rail transit within exclusive rights-of-way, or busways within separate rights-of-way. Although the operational characteristics of busways and rail transit lines are not similar to those of HOV lanes or freeways, the capital and operating costs are. The alternatives analysis process followed for UMTA funding purposes balances the characteristics of these technologies. The commuting public also perceives HOV lanes, rail transit lines, and busways as comparable technologies.

The arterial street equation provides a lower par value to adjust for the difference in operating characteristics between freeway (or exclusive) facilities and priority treatments within street rights-of-way. Local-service transit bus routes, with multiple stops along an arterial street HOV lane, should be evaluated according to a lower standard than is used for express bus freeway service.

Interpretation of CMI Values

Table 6 presents CMI values for the bus and HOV priority lane projects in Canada and the United States. The range of

TABLE 6 PEAK-HOUR FREEWAY AND HOV LANE CMI VALUES

HOV Project and Location	Corridor Mobility Index (CMI) HOV[2] (1000)	Freeway[2] (1000)	Corridor[3] (1000)	Percent Inc Total vs Freeway[4]
EXCLUSIVE IN SEPARATE R.O.W.				
Ottawa, Canada				
Southeast Transitway & Central Area Transitway	3.4	NA	3.4	NA
West Transitway	2.0	NA	2.0	NA
Southwest Transitway	1.2	NA	1.2	NA
Pittsburgh, PA				
East Busway	1.5	NA	1.5	NA
South Busway	.7	NA	.7	NA
FACILITIES IN FREEWAY R.O.W.				
Exclusive Facilities				
Houston, Texas				
I-10 (Katy) 3+ HOVs	.9	.5	.6	20
I-10 (Katy) 2+ HOVs	1.8	.6	1.1	95
I-45 (North)	2.3	.4	1.2	210
Los Angeles, I-10 (San Bern)	3.3	.6	1.6	160
Washington D.C.				
I-395 (Shirley)	3.7	.6	2.5	345
I-66	3.0	NA	3.0	NA
Concurrent Flow				
Los Angeles, Route 91	1.9	.6	1.0	60
Miami, I-95	1.4	.9	1.1	15
Orange County, Route 55	1.7	.7	1.0	45
San Francisco, CA				
Bay Bridge	1.0	0	.7	2,455
US 101	2.1	1.1	1.4	25
Seattle, WA				
I-5	1.0	.6	.7	20
SR 520	.5	.1	.3	150
Contraflow				
New York City, NJ, Rte. 495	7.4	.1	6.1	5,730
San Francisco, CA, US 101	3.0	1.2	1.9	60

Source: Reference 2
NA - Not Applicable
ND - No Data Provided

[1]See Equation 1
[2]See Equation 11
[3]See Equation 2
[4]Represents difference between total CMI and freeway CMI

accuracy of travel time, vehicle speed, and person-volume data for the freeway main lanes and the HOV lane should be recognized explicitly. Because traffic volume and speed vary daily, the CMI values should be considered to have at least a 10 percent variability. Such factors are recommended because of the relative ease of data collection and potential for consistency in data collection technique.

Also, the travel speeds and ridership used in the calculations should be indicative of conditions throughout the corridor, if CMI values are to be representative of peak operation.

As defined in the par value calculations, a CMI value of 1.0 indicates a HOV lane with approximately the same combination of speed and person-volume as a congested (LOS Ė) freeway or arterial street traffic lane. All of the facilities in Table 6 were evaluated with the freeway par value of 100,000. Depending on the freeway main-lane values, HOV lanes with SPV values below 1.0 may be ineffective projects.

Only three projects in Table 6 have CMI values less than 1.0. One is no longer operational (Katy HOV 3+), and another has a CMI five times higher than the adjacent freeway main lanes (SR 520, Seattle). The busway projects in Ottawa and Pittsburgh have somewhat constrained operating conditions in that many of the buses stop at transit stations along the busway and access the busway arterial street-type at intersections, resulting in much lower speeds than could be obtained in express operation. Even so, the CMI values for all but one of these facilities exceed 1.0.

CMI values in excess of 2.0 seem to be associated with projects that according to other data are considered extremely successful; 9 of the 21 projects in Table 6 satisfy this criterion.

Another method of interpretation involves a comparison of the freeway main-lane values with the total corridor system (freeway and HOV lane). The corridor index values are a weighted average of the freeway and HOV lane index values, using total person-movement as the weighting factor. The CMI for the HOV lanes is 40 to 50 percent higher than that for the freeway, which would indicate effective projects. Projects that increase the freeway CMI value by more than 100

TABLE 7 CMI VALUES FOR SELECTED RAIL TRANSIT SYSTEMS

Rail Transit System	Peak-Hour Peak Direction Ridership[1]	System Average Speed (mph)[2]	Corridor Mobility Index[3]
HEAVY RAIL TRANSIT SYSTEMS			
Atlanta			
North Line	6,400	34	2.2
South Line	4,500	34	1.5
East Line	3,100	34	1.1
West Line	2,700	34	.9
Washington, D C			
Red Line	11,300	30	3.4
Orange Line	9,800	30	2.9
Blue Line	5,000	30	1.5
Yellow Line	4,200	30	1.3
LIGHT RAIL TRANSIT SYSTEMS			
Calgary			
South Line	5,200	20	1.0
Northwest Line	3,200	20	.6
Northwest Line	3,900	20	.8
Edmonton			
Northeast Line	3,200	22	.7
Portland			
MAX LRT Line	1,600	20	.3
San Diego			
South Line	2,000	29	.6

[1]Source: Reference 7
[2]Source: Reference 8
[3]See Equation 11

TABLE 8 PEAK-HOUR CMI VALUES FOR EVENING PEAK HOUR ON SELECTED URBAN TEXAS FREEWAYS

City and Freeway	Peak-Hour Data		Speed of Person Volume[1] (1000)	Corridor Mobility	
	Volume Per Lane	Travel Speed		Index[2]	Rank
DALLAS AREA					
E R L Thornton (I-30)	1,930	30	70	.7	8
Old D/FW Trnpk (I-30)	1,750	45	94	.9	3
N Central (US 75)	1,800	25	54	.5	13
Stemmons (I-35E)	1,520	35	64	.6	11
S. R L Thornton (I-35E)	1,875	45	101	1.0	1
N LBJ (I-635)	2,080	35	87	.9	6
HOUSTON AREA					
Gulf (I-45)	1,990	40	95	1.0	2
North (I-45)	1,925	25	58	.6	12
East (I-10)	1,485	50	89	.9	5
Katy (I-10)	1,610	35	68	.7	9
West Loop (I-610)	2,080	30	75	.8	7
Eastex (US 59)	2,200	25	66	.7	10
Southwest (US 59)	1,555	25	47	.5	14
Northwest (US 290)	1,900	40	91	.9	4

Source: References 9, 10, 11

Note: See Table 6 for North and Katy Freeway and Transitway combined CMI values

[1]Average vehicle occupancy = 1.2 persons

[2]See Equation 11

percent are clearly successful in moving significantly more people at greater travel speed than is possible with single-occupant vehicles on mixed-flow lanes.

Several rail transit line peak-hour passenger loads and average system operating speeds are presented in Table 7 as an illustration of the application of the CMI calculation to other travel modes. The relatively low speeds are a result of the station stops, as is the case for the Ottawa and Pittsburgh busway systems (see Table 6). The CMI values for most of the heavy rail transit lines appear to exceed the CMI value representing a congested freeway lane (1.0). The lower speed and ridership values for the newer light rail systems result in CMI values less than 1.0.

A comparison of SPV and CMI values for some Texas freeways for which volume and travel time characteristics are available is presented in Table 8.

Application of Corridor Mobility Index Values

Experience from operating HOV lane projects suggests that a level of vehicle use between 600 and 1,000 in the peak hour is necessary for general public acceptance of a HOV lane in a freeway corridor. Vehicle-volume values below this have often resulted in a negative public perception of the priority treatment. The methodology outlined in this paper probably will not change these perceptions. If a lane appears to be underused, technical analyses of ridership and travel speed may not alter that perception.

This corridor mobility analysis is not as detailed as some other methodologies. The factors used in this procedure, however, focus on the important aspects of express transit and carpool operation. The combination of travel speed and person-volume directly measures one of the most important factors to the traveling public—speed—and an important measure of project success examined by public officials and urban commuters—ridership. If public discussion on major transportation facilities includes quantitative analyses, the CMI may provide a relevant comparison between general-purpose travel lanes and HOV lanes, busways, or rail transit lines.

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Computerized Sketch-Planning Process for Urban Signalized Intersections

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Urban transportation networks in many cities will continue to experience enormous pressures because of population growth. As a result, the level of service at many signalized intersections will decrease to unacceptable levels within the next 20 years. Continuation of traffic growth over this period will necessitate intersection improvements. Previous research provided a simple methodology [the sketch-planning process (SPP)] based on cost-effectiveness to determine the optimum intersection improvement plan. During the automation of the SPP, many improvements were introduced. These improvements include (a) determination of the exact future year an intersection fails to meet requirements, (b) determination of the intersection's failure mechanisms, and (c) ability to simulate periodic signal timing optimization. A case study illustrates the automated SPP (ASPP) software package, which is a quick and efficient implementation tool that helps planners to rate intersection improvements. This package will enable planners to determine right-of-way needs before development makes costs prohibitive.

One of the most complex locations in an urban traffic system is the signalized intersection. As urban traffic increases, existing signalized intersections will experience additional pressure and the level of service offered by the intersections will deteriorate. This situation will require remedy through signalized intersection improvements.

Previous research on the sketch-planning process (SPP) (*1*) provided a simple methodology for decision making in rating urban intersection improvements.

The objective of the SPP process was to identify improvements that enable an intersection to meet minimum level of service requirements under a predicted growth rate. In addition, this process analyzed the economic impact of the identified improvements.

The computerized process uses commonly accepted simulation models to accurately determine the effects of an intersection improvement on future traffic conditions year by year. Results of these simulations allow the user to clearly determine which improvements will be needed and for how long these improvements will maintain the intersection within the service requirements. Staged improvement series are accepted as well as alternative improvement series.

A data base is maintained of the improvement type, costs, and results for each series of improvements. From the results, public costs such as fuel, oil, tires, maintenance, repair, and depreciation can be determined. Using this information, comparisons can be made between each series of improvements to determine the optimum plan. Once the optimum plan has been determined, the needed right-of-way can be purchased before development drives the cost up or makes the cost prohibitive.

Fiscal planning can be enhanced by using this process to improve several intersections. Plans for improving each intersection can be combined to obtain an overall plan that meets monetary, personnel, and equipment constraints.

Automation of SPP is more than a way to execute the process faster because several improvements were implemented. Without automation, such improvements would have been difficult to execute manually. Most of these improvements are the result of simulating the intersection year by year, including

- Determination of the exact future year an intersection fails to meet requirements,
- Introduction of the automatic checking of the measures of effectiveness (MOE) providing information about the failure mechanism on which to base improvement selection,
- Simulation ability for periodic signal timing optimization,
- Generation capability for improvement alternatives showing the specified date at which the intersection must undergo construction (e.g., addition of lanes), and
- Estimation of each year's delay costs on the basis of present worth.

In order to allow the simulation of an exclusive, shared left-turn pair, the *Highway Capacity Manual* (HCM) (*2*) volume allocation methodology was extended to include exclusive-left- and shared-left-turn-lane groups. TRANSYT–7F's shared-lane model is unable to simulate this situation (*3*). Modification of the HCM method allows nearly all geometrics of an intersection leg to be considered for analysis.

AUTOMATION OF THE SKETCH-PLANNING PROCESS

The SPP, which forms the basis of the automated SPP (ASPP), can be thought of as an algorithm in which each step of the procedure requires from the user an input, computation, or decision, or a combination of these. The 10-step SPP process is shown in Figure 1 (*1*).

SPP methodology involves a comprehensive evaluation of all aspects of intersection improvement. This evaluation entails user input of various parameters pertaining to existing and future traffic conditions, available right-of-way, and physical and economic constraints for improvement. Constraints used to identify an intersection as a candidate for improvement are volume/capacity (v/c) ratio, delay values, and queue capacities

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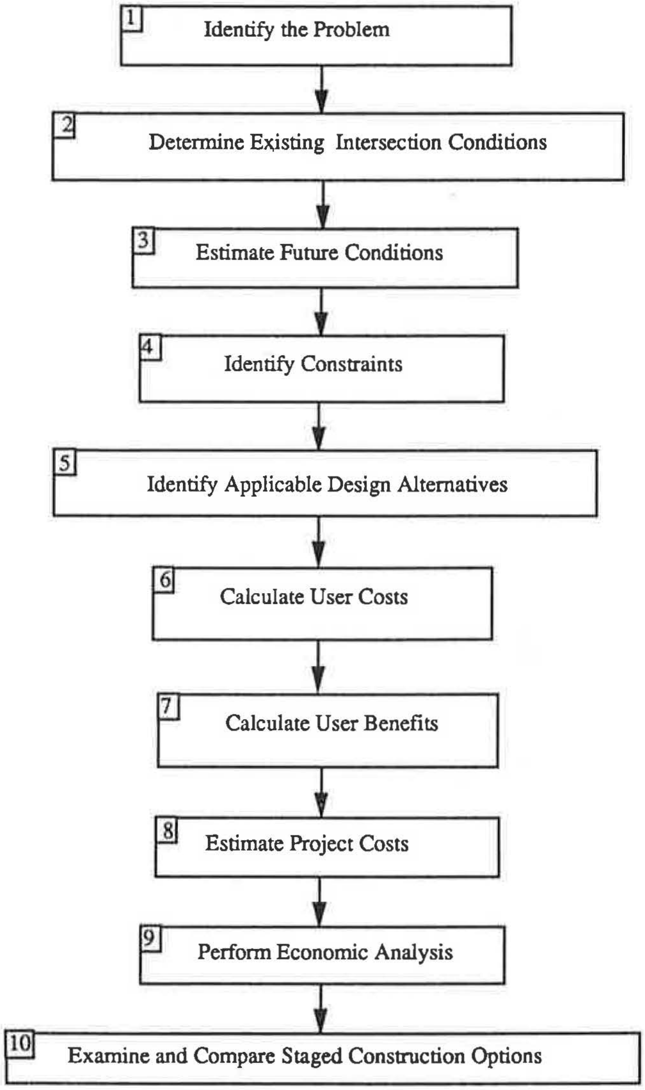

FIGURE 1 Ten-step SPP.

in each leg of the intersection. Calculation of user costs and project costs as well as user benefits from the intersection improvement enable a final economic analysis to be performed for each improvement alternative. This calculation helps determine the benefit-cost ratio of each improvement alternative.

ASPP, unlike SPP, is presently restricted to isolated urban signalized intersections with four or fewer legs, which should cover the majority of intersections experiencing deficiencies as a result of increases in traffic volume.

Assuming an intersection requiring improvement has been identified, the first step of SPP is to identify the problem causing the intersection to require improvement. The usual reasons for an intersection to be considered for improvement are operational and safety problems. Operational problems are characterized by excessive delay, insufficient capacity, or queues larger than available storage. Safety problems are characterized by a high rate of property damage, injury, or fatalities.

Although intersection safety is enhanced by the application of ASPP, sufficient safety parameters did not exist to account for all conditions to be incorporated into the ASPP model. Therefore, safety parameters were not implemented in ASPP.

Response of an intersection's operation to an improvement can easily be determined by one of a variety of traffic simulation programs (*3*). TRANSYT-7F was chosen for its ability to optimize signal timing, accurately simulate shared lanes and actuated signals, and meet the requirements of planned revisions. TRANSYT-7F is a common accepted simulation program, and its evaluation of an intersection is constantly being improved.

TRANSYT-7F output contains information needed to determine whether an intersection meets performance requirements. Two requirements are automatically checked by ASPP. These are the *v/c* ratio for each movement and the queue length for movements having a turn bay. Delay values are displayed so that the user can check excessive delay on the basis of selected criteria.

DEFINING INTERSECTION CONDITIONS

Three sets of conditions define the problem of determining when an intersection fails to meet requirements, including (a) conditions of the existing intersection, (b) conditions of growth, and (c) conditions defining adequate performance. Any change in these conditions will modify the results; therefore, for valid comparisons between different intersection improvement plans the conditions must be identical.

Determination of these conditions correspond to Steps 2 through 4 of the SPP (Figure 1). In ASPP, the user must develop a data set containing values such as number of lanes, growth rate, etc. (Figure 2), that define these three conditions. The first module of ASPP (the input module) was designed to gather values in Figure 2 from the user and create the data set (Figure 3).

Data required to define the three conditions are detailed. Extensive requirements are needed to satisfy the procedures used by the ASPP, including (a) TRANSYT-7F simulation, (b) HCM (*2*) procedures for estimating capacity, (c) constraint-checking procedures, and (d) delay cost estimation. Users must be responsible for other data such as the extent of existing right-of-way, feasible right-of-way acquisition, and consideration of social and political constraints on intersection improvement.

In order to estimate future traffic volumes, ASPP uses a compound interest formula. The growth rate for each leg of the intersection can be individually entered (Figure 2), allowing differential growth to be modeled (e.g., the different growth rate that would occur at the edge of urban areas as compared to the center of the city). Furthermore, the growth rate for two classes of heavy vehicles are modeled to account for commercial, industrial, and residential zones.

Future traffic volumes, mix of heavy vehicles, and lane capacities are estimated by ASPP using existing conditions and growth rates. Predicted values are placed into a TRANSYT-7F input deck so that the operational conditions can be estimated. These tasks are normally time-consuming, but once the existing conditions have been defined, the user's effort is reduced to entering the growth rates.

ASPP uses capacity estimation procedures of the HCM (*2*). Capacity value input to TRANSYT-7F is the capacity of the

1. Intersection name _______, City, County or DOT District name _________

(Intersection drawing appears on your PC screen (Figure 5)).

	SB			WB			NB			EB		
2. GEOMETRICS	R	T	L	R	T	L	R	T	L	R	T	L
a) no. of lanes at each approach												
b) gradient at each approach												
c) bay length at each approach												
3. TRAFFIC a) Traffic volume at each approach												
b) % of heavy trucks at each approach												
c) speed limit at each approach												
d) no. of sneakers at each approach												
e) number of pedestrians at each approach												
4. Traffic Volume growth at each approach												
5. Maximum acceptable v/c ratio (or 0.90)												

6. Signal phasing and timing (as in Table 7)

7. Maximum back of queue : <__ % of of turn bay length (or default value of 100%).

8. Vehicle Spacing _____ (or use a default value of 25 feet).

9. Check input values (Steps 1-8) for consistency and write a TRANSYT-7F data set projecting existing conditions to future conditions (year-by-year until the end of planning horizon).

FIGURE 2 Input module steps.

lanes for 1 hr of green time. Furthermore, TRANSYT-7F does not model the effect of heavy vehicles on lane capacity. Therefore, the values calculated by ASPP compensate for geometric conditions (e.g., gradient and lane width) and traffic mix, but assume 100 percent protected green. Calculated capacity values are also entered as the maximum flow rate for unprotected opposed movements during a permitted phase.

In developing ASPP, it was found that TRANSYT-7F would not simulate all the intersection geometries for any number of lanes or type of movement. Specifically, TRANSYT-7F will not simulate a turning movement with one or more exclusive lanes and a shared lane. In order to remedy this situation, it was decided to distribute the approach volume to lane groups such that the resulting *v/c* ratios for all movements would be equal. This decision is based on the assumption that lane switching would occur to cause the queues to empty at the same time. This assumption, coupled with the restriction on signal timing imposed by the presence of a shared lane, would result in equal *v/c* ratios. Formulas developed from this assumption are equivalent to HCM (*2*) formulas when no exclusive turn lanes are present. In order to test the formulas, a range of input conditions was simulated by TRANSYT-7F. Simulated *v/c* ratios found in TRANSYT-7F were within 1 to 2 percent of the *v/c* ratios predicted by the formulas.

As mentioned before, ASPP checks each simulation for compliance with constraints on the *v/c* ratio and queue length. Maximum permissible *v/c* ratio can be entered for lefts, throughs, and rights (Figure 2). Users may enter a percentage of bay length use and vehicle spacing. Defaults for these characteristics are 100 percent and 25 ft, respectively.

Constraint on maximum *v/c* ratio exists for two reasons. First, movements with high *v/c* ratios are generally not considered acceptable. Second, various delay formulas tend to differ markedly for high *v/c* (>1) ratios, but are in accord with generally acceptable *v/c* (<1) ratios. User costs are based on delay; therefore, the predicted cost from the different models will not differ greatly if the *v/c* ratios are restricted to reasonable values.

Constraint on maximum queue length is optimal. Bay length is used only for this test. If no bay length is entered for a movement, then the bay length constraint is not considered for this particular movement. TRANSYT-7F does not simulate the blockage that would occur when the queue is longer than the turn bays; therefore, the delay value for the blocked movement is underestimated.

DEVELOPMENT AND EVALUATION OF IMPROVEMENT DESIGN ALTERNATIVES

After all values from Figure 2 have been entered and stored, the user can request the input module to write a TRANSYT-7F input deck. This deck will contain sections representing volume and capacity values for the first and up to 20 successive

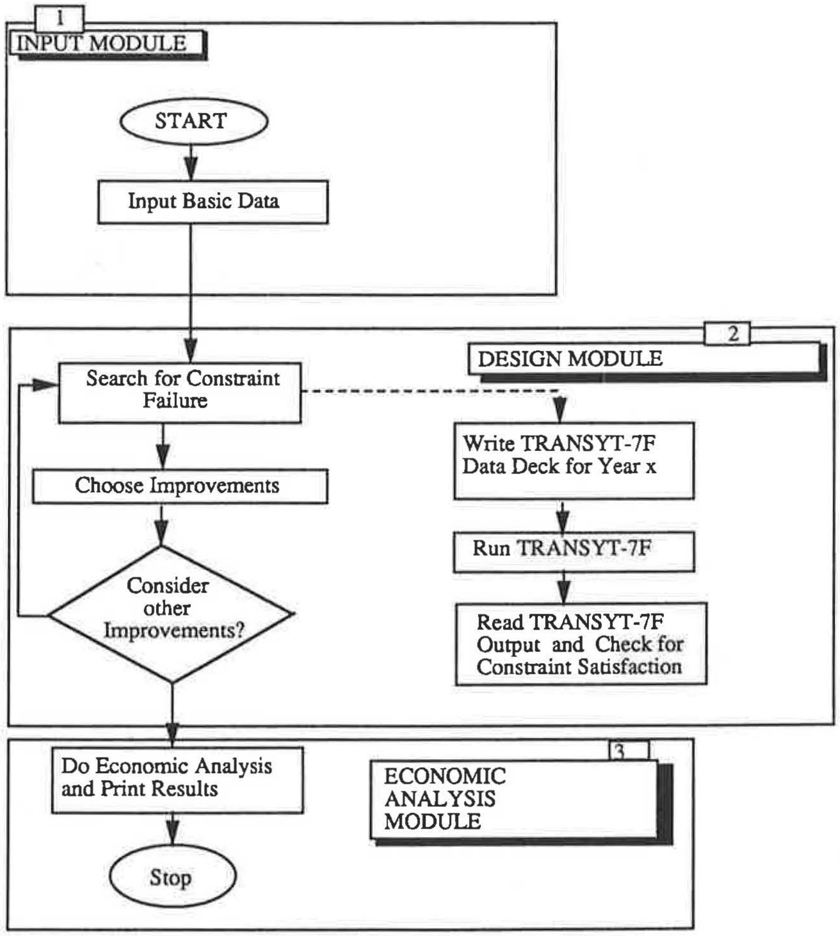

FIGURE 3 Organization of the overall ASPP program.

TABLE 1 INPUT DATA FOR CASE STUDY

	South Bound			West Bound			North Bound			East Bound		
	R	T	L	R	T	L	R	T	L	R	T	L
# of lanes	1	3	1	-1	1	1	-1	3	1	1	2	1
lane width	12	12	12	12	12	12	12	12	12	12	12	12
bay length	100	100	100	100	100	100	100	100	100	100	100	100
volumes	49	450	57	6	46	70	66	1000	73	50	650	49
%3S2 trucks	2	2	2	0	0	0	0	2	2	2	0	0
%SU trucks	0	0	0	0	0	0	0	0	0	0	0	0
% growth rate for car	3	3	3	3	3	3	3	3	3	3	3	3
% growth rate for 3S2 trucks	3	3	3	3	3	3	3	3	3	3	3	3
% growth rate for SU trucks	3	3	3	3	3	3	3	3	3	3	3	3
green ext	1	1	1	1	1	1	1	1	1	1	1	1
startup lost time	1	1	1	1	1	1	1	1	1	1	1	1
gradients		0.0			0.0			0.0			0.0	
parking	0			0			0			0		
speed limit		45			35			45			35	
pedestrian	0			0			0			0		
growth of pedestrians	3			3			3			3		
phases	3	2	0	3	2	0	0					
green time	5	45	0	5	45	0	0					
yellow	5	4	0	5	4	0	0					
all red	0	1	0	0	1	0	0					

years. While calculating future conditions, ASPP calculates an estimate of each movement's *v/c* ratio. When a *v/c* estimate is found greater than the user's chosen maximum acceptable *v/c* value, ASPP stops writing sections.

Once TRANSYT–7F has been run with the previously mentioned input deck, the user is ready to use the second module of ASPP (design module). This module is run iteratively in alternation with TRANSYT–7F (Figure 3) to develop and evaluate intersection improvement designs. An improvement design may include a set of improvements, such as the addition of a northbound left-turn bay and an eastbound exclusive right-turn lane. All improvements in a set are assumed to become effective at the beginning of the selected year for construction. Only one set of improvements can be evaluated in one alternation; however, iterative use will allow a series of improvement sets and also an alternative series of improvement sets to be evaluated. A series of improvement sets will be called an improvement alternative.

Three functions are fulfilled by the second module of the ASPP: (a) scan and store output of the prior simulation, (b) accumulate information about a selected improvement, and (c) create a TRANSYT–7F input deck to evaluate the improvement. These three functions correspond to Steps 5, 8, and 10 of the SPP process (Figure 1). Calculation of delay needed in Step 6 is performed during the simulation of an improvement design.

The first action taken by this module is to search for a TRANSYT–7F output file. If an output file is found, the design module of the ASPP will search for a punch data set. Signal timing optimization causes a punch data set to be created by TRANSYT–7F. If a punch file is found, the optimum timing is read from this file and placed in a data base of designs. After processing the punch data set, the design module of ASPP will scan the output file. Measures of effectiveness (MOE) tables from the output will be copied to a data base of simulation outputs for later use.

These data bases permit users to evaluate simulated intersections by quickly displaying a variety of information, which is then needed to develop TRANSYT–7F input decks and economic analysis. For any simulated year of an improvement alternative, the following information may be displayed: volume and capacity distribution, MOE table, and signal timing. Values indicating constraint failure will be highlighted in the MOE tables. From this information, the user should be able to determine the particular year that the intersection meets all performance requirements.

ASPP can simulate several modifications to an intersection, including (a) addition of lanes, (b) change in lane width, (c) increase of turn bay length, and (d) signal timing optimization. Any combination of these modifications may be selected for the various movements and directions. Signal timing optimization is restricted to cycle length search and split optimization. Changes in signal phasing are not presently allowed. Users should determine a combination of modifications considering physical, political, social, and budgetary limitations that might make the intersection meet requirements.

Each intersection modification must be entered along with associated construction, right-of-way, and additional maintenance costs. These costs and other relevant values about the improvement will be stored in an alternative data base for use in determining the conditions existing at a selected year. It is recommended that the user maintain sufficient documentation on each improvement set. Details of the costs should include information on right-of-way usage, underground obstructions, etc.

If cycle timing optimization was selected, ASPP writes an input deck requesting a cycle search and split optimization for the modified intersection. On obtaining the optimum timing from the punch data set, ASPP simulates the new signal timing for the remaining years. If signal timing optimization was not selected, ASPP simulates the intersection modifications for the remaining years. As with the existing condition simulation, the minimum required simulations will be executed as previously discussed.

In order to facilitate the alternation of ASPP modules and TRANSYT–7F, an assembly language program was developed that will correctly sequence these programs. To examine or modify the input deck, the user may control the sequence of programs by batch or command entry. One instance in which the modification of the input deck might be useful is for modifying an input deck, requesting a signal timing optimization to meet local regulations on minimum green time.

COSTS MODULE

When the user feels that a sufficient number of alternative improvement series have been evaluated for performance requirements, the user may select the costs module of ASPP (Figure 3). The costs module calculates the delay costs associated with a selected alternative. In addition, it will compare costs among several alternatives.

ASPP uses the same cost calculation procedure as SPP (*1*). Total intersection delay from the MOE tables for each year of the selected alternative is used to estimate the yearly delay costs. Delay costs are defined as travel time cost, running cost caused by speed change and stopping, and idling cost.

The inflation rate used to project the estimated costs to future worth and the discount rate used to return to present worth are assumed to be the same. Comparisons between alternatives are based on the sum of the present worths of each year's costs. Because the inflation and discount rates are the same, the estimate of the present worth of a cost is the estimated value of the cost regardless of the year in which it occurs. This assumption causes the economic analysis to be insensitive to variations in discount rate.

Alternatives are compared on the basis of the following costs: (a) total delay costs, (b) average yearly delay costs, (c) total intersection improvement costs, (d) average yearly intersection improvement costs, (e) total costs over the entire planning horizon [the sum of (a) and (c)], and (f) benefit/cost ratio (*B/C*) of each improvement alternative.

During the development of ASPP, a problem was noticed in programming the evaluation of *B/C* ratios. SPP defined the benefit as the difference between delay costs of the unimproved intersection to the delay costs of the improved intersection. However, because of various methods of estimating delay for high *v/c* ratios and underestimation of delay when queues become longer than bay lengths, the unimproved intersection is not simulated after these conditions occur. These conditions signal the need for a modification of the intersection;

FILE	PROGRAM	GEOMETRY	TRAFFIC	SIGNAL	GROWTH
Set Dir	Clear data	Lane #	Volumes	Phasing	Cars
Make Dir	Identify	Lane widths	%3S2 Trucks	Green Ext.	3S2 Trucks
Retrieve File	Constraints	Bay length	%Su Trucks	Lost Time	SU Trucks
Merge File	Check Data	Gradients	Parking		Peds
Save	Search		Speed Limit		
Save As			Sneakers		
Print Data			Pedestrians		
Shell to DOS			Distribution		
Exit					

PULL-DOWN MENUS FOR INPUT MODULE

FILE	REVIEW	CHANGES
Set Dir	Select	Improvements
Retrieve	Performance	Check
Identify	Distribution	Search
Shell to DOS	Timing	
Exit		

PULL-DOWN MENUS FOR DESIGN MODULE

FILE	INDIVIDUAL	COMPARATIVE
Set Dir	Select	Ranking
Retrieve	Performance	
Identify	Distribution	
Shell to DOS	Timing	
Exit	Cost Analysis	
	Plan	

PULL-DOWN MENUS FOR ECONOMIC ANALYSIS MODULE

FIGURE 4 Pull-down menus for the three ASPP modules.

therefore, simulation of the modified intersection will occur after simulation of the unimproved intersection stops. Without a value for the delay of the unimproved intersection, the benefit cannot be calculated by the SPP method. ASPP reports *B/C* ratios and considers delay costs as disbenefits.

CASE STUDY

ASPP software is better illustrated through use of a case study. The selected intersection is SW 34th Street and SW 2nd Avenue, located in the city of Gainesville, Florida. For this intersection, Table 1 presents the values for (a) existing geometric conditions; (b) predicted future traffic growth rate; (c) constraints such as *v/c*, vehicle spacing, etc.; and (d) signal timing.

Table 1 presents the steps of the input module, whereas Figure 4 shows the menus of the input module. Users may choose menu items either by cursor control or a mouse. Menu items not related to file or program control cause a screen to be displayed for entering the menu item. Input screens are provided for entering identification information and constraint specification. Maximum *v/c* values are allowed to default to 0.9, whereas bay length and average vehicle spacing are allowed to default to 100 percent and 25 ft, respectively.

All other data in Table 1, except signal timing, are entered on a screen similar to that shown in Figure 5. The relationship of screen position to data item should be intuitive. The number displayed is the number of lanes that service the lefts, throughs, and rights. The negative sign for the right turn indicates the lane is shared with another movement.

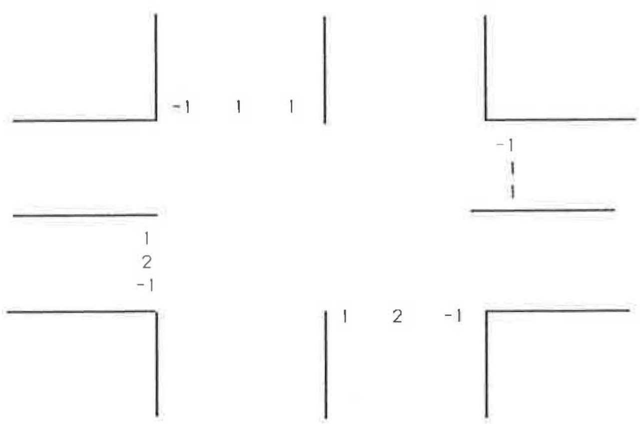

FIGURE 5 ASPP input screen for numbers of lanes.

The data for the signal timing screen are presented in Table 2. Signal phase sequences are selected by menu control. Signal timing is presented as 45 sec of permitted lefts and throughs with a 5-sec yellow clearance followed by 5 sec of protected left with 4 sec of yellow clearance and 1 sec of all red. Both north-south and east-west directions have this sequence.

After the signal timing and all the data from Table 1 were entered, input values were checked for consistency and stored. The simulate option from the menu was selected, to cause the input module to write a TRANSYT-7F input deck. TRANSYT-7F was then run to simulate existing conditions.

The next step is the start of the alternation of the second module of ASPP (design module) and TRANSYT-7F. The

TABLE 2 ASPP INPUT SCREEN FOR SIGNAL TIMING DATA

	Movement	Timing		
		Green	Yellow	Red
North/South Subsequence	P	45	4	0
	L	5	4	1
East/West Subsequence	P	45	4	0
	L	5	4	1
Walk Only Subsequence	0	0	0	0

TABLE 3 STATUS OF INTERSECTION PERFORMANCE FOR SELECTING INTERSECTION IMPROVEMENT

YEAR	Alternative 0	Alternative 1	Alternative 2	Alternative 3
0	29.3	29.3	29.3	29.3
1	29.7	29.7	29.7	29.7
2	30.0	30.0	30.0	30.0
3	30.4	30.4	30.4	30.4
4	30.8	30.8	30.8	30.8
5	31.2	31.2	31.2	31.2
6	31.7	31.7	31.7	31.7
7	32.1	32.1	32.1	32.1
8	32.8	32.8	32.8	32.8
9	33.4	33.4	33.4	33.4
10	34.3	34.3	34.3	34.3
11	35.1	35.1	35.1	35.1
12	36.1	36.1	36.1	36.1
13	37.3	37.3	37.3	37.3
14	38.8	34.5i	34.5i	34.5i
15		35.5	35.5	35.5
16		36.9	35.2i	33.7i
17			36.3	36.3
18			34.2i	34.2i
19			35.2	35.2

second module read the TRANSYT-7F output and placed the MOE tables in a data base. The select menu item was chosen, displaying a screen similar to Table 3. At this time, only the alt0 (alternative "0") column contained data. The last row containing data in this column corresponds to year 14, indicating that the existing intersection arrangement will fail to meet requirements in year 14. This year was selected for examination.

Figure 6 is the product of SPP applied to this case study, a graphical presentation of average delay versus time (in years). ASPP uses data in Table 3, which is consistent with Figure 6. The important point about Figure 6 is the relationship between construction timing of an alternative to its level of service. The effect of any improvement on level of service can be determined from the values displayed in Table 3.

In the case study, all displayed values represent level of service (LOS) D. Normally, the user would attempt to construct a series of improvements that kept the intersection at LOS C. The other aspects of ASPP were better illustrated by having the intersection fail to meet programmed constraints.

The MOE table was requested by choosing the performance menu item. Examination of the MOE table (Table 4) indicated that the northbound through-*v/c* value exceeded the constraints. Three modifications come to mind: (a) add a northbound lane, (b) retime the intersection, and (c) add an adjacent right-turn bay. Combinations of these three may also be simulated by the ASPP.

In order to determine whether one of the previously mentioned options will be effective, the user may display other information about the simulated intersection, such as volume distribution and lane capacities. The distribution menu item was chosen next to display the volume distribution and lane usage (Table 5). From this screen, it can be seen that the predicted volume of through vehicles in the shared right lane

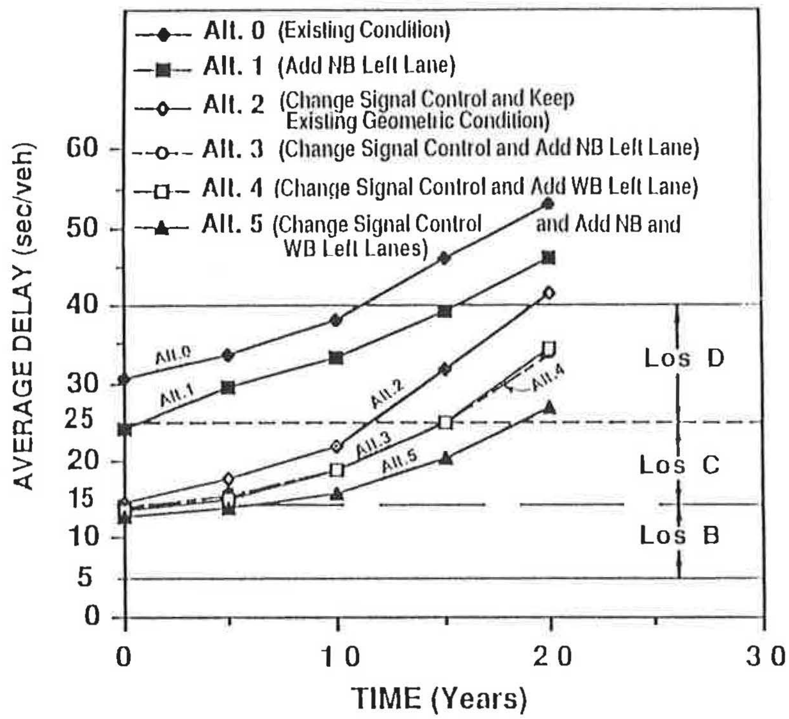

FIGURE 6 Average delay versus time graph to examine staged construction options over the design period.

is not small. Not much benefit could be expected from adding a right-turn bay.

In the MOE tables (Table 4), some high *v/c* ratios occur in the eastbound and westbound movements. Because of these high *v/c* ratios, split optimization would not be expected to solve this constraint without breaking another. The remaining choice is to add a through lane.

Before simulating the addition, the user needs to obtain information on the costs of adding the through lane. Construction costs were estimated at $115,000, right-of-way (R-W) costs at $792,000 for a through length of 2,640 ft at an average of $25/ft^2 R-W cost, and additional maintenance costs at $5,000. These costs assume a lane width of 12 ft. These cost values were then entered into the design module and the simulation process executed.

Returning to the selection screen (Table 3), one can see that adding a northbound through-lane extended the life of the intersection to year 16 (Alternative 1). MOE tables (Table 6) indicate the westbound left *v/c* value is the cause of failure. Two modifications may improve the intersection: (a) adding a lane to the westbound left-turn bay, and (b) adding an eastbound through-lane. No evidence existed to indicate that either would not work; therefore, both were evaluated. Construction, right-of-way, and additional maintenance costs for Option 1 are estimated as $40,000, $30,000, and $0, whereas these costs for Option 2 are $115,000, $396,000, and $2,500, respectively.

Exercising Option 1 (Alternative 2 in Table 3) allowed the intersection to meet the constraints until year 18. Failure occurred because of the high *v/c* value at the eastbound through-lane. At this point, adding an eastbound through-lane allowed the intersection to meet requirements for the rest of the planning horizon (20 years).

The eventual need for adding an eastbound through-lane in this alternative indicates Option 2 might be a better solution. Adding the eastbound through-lane (Alternative 3 in Table 3) reduced the westbound left *v/c* value to acceptable levels, but only until year 18. Here, adding a westbound left lane seems proper. By adding such a lane, the intersection meets performance requirements until the end of the planning horizon.

From the application of ASPP, the user has learned three significant points about the future needs of the case study. The user must add (a) a northbound through-lane, (b) an eastbound through-lane, and (c) a westbound left-turn lane.

At this point, it was decided to execute the cost module of ASPP. In Table 4, the difference in Alternatives 2 and 3 occurs in years 16 and 17, with Alternative 3 having the smaller average vehicular delay. The cost analysis (Table 7) indicates that Alternative 3 has the lower delay costs because of the addition of the eastbound through-lane in year 16.

The comparative menu item ranks the alternatives, on the basis of several costs functions (Table 8). This figure indicates Alternative 3 is the best alternative considering all comparison methods (e.g., total user costs and total government costs), except the *B/C* ratio.

As mentioned earlier, *B/C* ratio treats delay costs as disbenefits and reports this ratio as negative. Therefore, using this technique for the evaluation of intersection improvement alternatives is not proper.

Another reason exists for selecting a different technique for evaluating intersection improvement alternatives other than using the *B/C* method. Decisions as to which one of two alternatives is better depend on whether all the years within the planning horizon are observed, or only the portion in which the alternatives are different.

TABLE 4 PERFORMANCE TABLES FOR YEAR 14, ALTERNATIVE 0

NB Movements									
Exc. Left	68	2.05	1.04	0.99	32.5	88(80)	2	4	1.37
E/S Thru	90	20.26	13.15	12.70	42.1	989(91)	34	8	22.49
Sha. Thru R	87	7.94	5.46	5.29	44.7	89(89)	16	4	8.96
Shared R	87	1.86	1.28	1.24	44.7	89(89)			1.54
SB Movements									
Exc. Left	75	1.60	1.84	1.80	75.3	82(95)	3	4	1.89
E/S Thru	38	12.69	5.50	5.22	27.6	477(70)	16	12	10.29
Exc. R	14	1.38	0.55	0.52	25.1	47(63)	2	4	0.74
WB Movements									
Exc. Left	83	1.98	2.34	2.28	77.5	102(96)	3	4	2.38
Sha. Thru R	13	1.30	0.52	0.48	24.9	44(63)	2	4	0.70
Shared R	13	0.19	0.07	0.07	24.9	65(63)			0.10
EB Movements									
Exc. Left	13	1.38	0.42	0.38	18.7	41(56)	1	4	0.61
E/S Thru	81	18.32	10.55	10.03	36.7	84(86)	29	8	14.58
Exc.Right	15	1.42	0.57	0.53	25.1	48(63)			0.76

TABLE 5 VOLUME DISTRIBUTION FOR DIFFERENT MOVEMENTS FOR YEAR 14, ALTERNATIVE 0

	Exc. Left	Shared Left Left	Shared Left Thru	S/E Thru	Shared Right	Shared Right Right	Exc. Right
SB VOLUMES	86	0	0	512	169	74	0
# of Lanes	1	0	0	2	1	1	0
Capacity	1526			1606			1365
WB VOLUMES	106	0	0	0	70	10	0
# of Lanes	1	0	0	0	1	1	0
Capacity	1539			1620			1377
NB VOLUMES	110	0	0	1087	426	100	0
# of Lanes	1	0	0	2	1	1	0
Capacity	1526			1606			1365
NB VOLUMES	74	0	0	983	0	0	76
# of Lanes	1	0	0	2	0	0	1
Capacity	1539			1620			1377

TABLE 6 PERFORMANCE TABLES FOR YEAR 16, ALTERNATIVE 1

	V/C (%)	TOTAL TIME (V-MI)	TOTAL TIME (V-HR)	TOTAL DELAY (V-HR)	AVG. DELAY SEC/V)	UNIFORM STOPS No. (%)	MAX. BACK OF QUEUE No.	Cap.	FUEL . CONS. (Gal.)
<u>WB Movements</u>									
Exc. Left	92	2.09	3.42	3.36	108	108. (97)	4	4	3.22
Sha. Thru R	14	1.38	0.55	0.51	25	47. (63)	2	4	0.74
Shared R	14	0.19	0.07	0.07	25	6. (63)	0	0	0.10
<u>EB Movements</u>									
Exc. Left	14	1.47	0.46	0.41	18.9	45. (57)	1	4	0.66
E/S Thru	86	19.44	11.84	11.29	39.0	926. (89)	32	8	16.14
Exc. R	15	1.49	0.60	0.56	25.2	51. (63)	2	4	0.80

TABLE 7 COST ANALYSIS FOR ALTERNATIVES 2 AND 3

			Present Value Costs(k$) for Alternative 2.							
Year	delay	%stops	Travtime	Running	Idling	Sum	Maint	Const	RightW	Sum
0	20.90	73	1258.5	341.4	118.9	1719	0	0	0	0
1	21.08	74	1311.1	356.5	123.9	1792	0	0	0	0
2	22.07	75	1366.8	372.1	129.1	1868	0	0	0	0
3	23.07	75	1425.1	383.3	134.6	1943	0	0	0	0
4	24.07	76	1488.3	400.0	140.6	2029	0	0	0	0
5	25.08	76	1552.7	412.0	146.7	2111	0	0	0	0
6	27.00	77	1623.7	430.0	153.4	2207	0	0	0	0
7	28.20	78	1695.8	448.6	160.2	2305	0	0	0	0
8	29.60	79	1781.9	468.0	168.3	2418	0	0	0	0
9	31.10	79	1870.3	482.1	176.7	2529	0	0	0	0
10	32.90	80	1976.8	502.8	186.7	2666	0	0	0	0
11	34.60	81	2081.5	524.4	196.6	2802	0	0	0	0
12	36.70	82	2207.2	546.8	208.5	2962	0	0	0	0
13	39.10	83	2351.0	570.0	222.1	3143	0	0	0	0
14	37.20	80	2237.9	565.9	211.4	3015	5	215	792	1012
15	39.50	81	2375.0	590.2	224.4	3190	5	0	0	5
16	40.30	82	2423.8	615.4	229.0	3268	5	40	30	75
17	42.80	82	2573.5	633.8	243.1	3451	5	0	0	5
18	41.50	80	2497.7	636.9	236.0	3371	7	115	396	518
19	44.10	80	2650.5	656.0	250.4	3557	7	0	0	7
		Present Value totals				52346				1622

TABLE 7 *(continued on next page)*

TABLE 7 (*continued*)

			Present Value Costs(k$) for Alternative 3.							
Year	delay	%stops	Travtime	Running	Idling	Sum	Maint	Const	RightW	Sum
0	20.90	73	1258.5	341.4	118.9	1719	0	0	0	0
1	21.08	74	1311.1	356.5	123.9	1792	0	0	0	0
2	22.70	75	1366.8	372.1	129.1	1868	0	0	0	0
3	23.70	75	1425.1	383.3	134.6	1943	0	0	0	0
4	24.70	76	1488.3	400.0	140.6	2029	0	0	0	0
5	25.80	76	1552.7	412.0	146.7	2111	0	0	0	0
6	27.00	77	1623.7	430.0	153.4	2207	0	0	0	0
7	28.20	78	1695.8	448.6	160.2	2305	0	0	0	0
8	29.60	79	1781.9	468.0	168.3	2418	0	0	0	0
9	31.10	79	1870.3	482.1	176.7	2529	0	0	0	0
10	32.90	80	1976.8	502.8	186.7	2666	0	0	0	0
11	34.60	81	2081.5	524.4	196.6	2802	0	0	0	0
12	36.70	82	2207.2	546.8	208.5	2962	0	0	0	0
13	39.10	83	2351.0	570.0	222.1	3143	0	0	0	0
14	37.20	80	2237.9	565.9	211.4	3015	5	215	792	1012
15	39.50	81	2375.0	590.2	224.4	3190	5	0	0	5
16	38.50	79	2317.9	592.9	229.0	3130	5	115	396	518
17	41.00	79	2467.1	610.6	243.1	3311	5	0	0	5
18	41.50	80	2497.7	636.9	236.0	3371	7	40	30	77
19	44.10	80	2650.5	656.0	250.4	3557	7	0	0	7
		Present Value totals				52068				1626

TABLE 8 RANKING OF ALTERNATIVES

Alternative	**2**	**3**
Years to finish	**19**	**19**
Ranking of alternative	2	1
Total user. cost	**52346.08**	**52067.96**
Ranking of alternative	2	1
User cost/year	**2755.057**	**2740.419**
Ranking of alternative	2	1
Total Govt. cost	**1622**	**1626**
Ranking of alternative	2	1
Govt. Cost/year	**85.36842**	**85.57895**
Ranking of alternative	2	1
Total Cost	**53968.08**	**53693.96**
Ranking of alternative	2	1
B/C Ratio	**-32.27255**	**-32.02211**
Ranking of alternative	2	1

This argument can be illustrated by considering two alternatives having identical first 10 years with a benefit of 10 and a cost of 1. For the next 10 years, let both Alternatives A and B have a benefit of 2 and a cost of 1, and a benefit of 5 and a cost of 2, concurrently. Then for the entire 20 years, B/C ratios are 6 and 5 for Alternatives A and B, respectively, whereas the B/C ratios for the last 10 yr for Alternatives A and B are 2 and 2.5. On the basis of the 20-year B/C ratios, Alternative A would be chosen. However, at the end of the first 10 years, the choice of Alternatives A or B is still valid. At this point in time, Alternative B becomes the better choice.

It can be seen from Table 8 that Alternative 3 has a smaller total cost than Alternative 2. The final intersection geometry is the same for Alternatives 2 and 3; however, the year in which each modification occurred is different (Table 7). This result indicates the importance of finding the optimum year at which the modification must occur. ASPP may be used to identify the optimum improvement year.

For any alternative in Table 3, adding the northbound lane 1 year earlier will not change the present value of both construction and R-W costs (on the basis of constant dollars). However, one must recognize that the future construction and R-W costs will be different, on the basis of inflation.

Moving the construction timing forward will add additional years of maintenance costs, and the present value of public expenditure (maintenance, construction, and R-W) will increase. However, the public will benefit as a result of reduced delay costs.

As construction time is moved year by year toward the beginning of the planning period, additional benefits because of reduction in delay costs will accordingly decrease, as indicated by the decreasing vertical distance separating the lines representing Alternatives 2 and 3 in Figure 6.

In order to determine the optimum construction timing, the total cost can be minimized. This minimum will occur the first year in which the benefit exceeds the maintenance cost.

In order to determine the optimum improvement timing, ASPP is simulated as if the intersection construction would occur at year zero. Then each year of this alternative is compared to the base alternative by subtracting their delay costs. The difference is the benefit resulting from reduction in delay costs. The year in which this benefit exceeds the maintenance cost is the optimum year to finish the intersection construction.

CONCLUSION

As demonstrated, the ASPP software package is a quick and efficient implementation of the SPP, which helps planners order urban signalized intersection improvements by priority.

From the analysis, the need for future acquisition of R-W can be determined in advance. R-W can be purchased at predevelopment cost as compared to postdevelopment cost. As a result, significant cost savings should be realized.

During the effort to automate the existing SPP, several other improvements to the SPP were incorporated. For instance, the year-by-year detailed generation of improvement alternatives helps engineers to extensively examine intersection performance. In addition, the capability of the existing TRANSYT-7F was enhanced by incorporating a procedure to simulate exclusive-left- and shared-left-lane groups. Furthermore, the economic analysis module for comparing improvement alternatives has made the SPP process more accurate.

Existing ASPP is capable of handling only isolated signalized intersections. However, TRANSYT-7F is capable of simulating a coordinated network of intersections. Future research is planned to extend the capability of the existing process to include a coordinated network of intersections.

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Expert System for Aspects of the TSM Process

M. A. MORRIS AND L. J. POTGIETER

An expert system technology is applied to the transportation system management (TSM) process. Two simple expert systems were built to test the feasibility of applying expert system technology to certain aspects of TSM. Findings of the feasibility study were then used to define requirement specifications for a practical expert system to provide multisolutions for problems identified through the TSM process. Steps taken to implement a full-scale TSM expert system are then described. Experience gained during the feasibility stage of the project indicates that successful knowledge engineering is the key to successful expert system development. An extensive and detailed description of the formal knowledge engineering approach used to clarify and structure the TSM knowledge into a form usable in an expert system is provided. To test the effectiveness of the expert system, output from the system was compared with recommendations made by transportation consultants on seven large-scale intersection problems. The test indicates that the output of the system compares favorably with the recommendations made by human experts.

The systems approach to problem solving in transportation is well established and has been effective in optimizing existing transportation infrastructures and operations. However, limited funds and expertise have placed constraints on this method and it has become apparent that new tools and formalized methodologies are required to make more effective use of this approach.

An expert system is constructed for certain aspects of the transportation system management (TSM) methodology. The expert system is intended to be a practical working system to be used when necessitated by a lack of TSM expertise.

Practical transportation engineering knowledge is acquired and structured into a form usable by an expert system shell (called knowledge engineering by expert system builders).

Knowledge engineering is becoming the underlying factor in the success or failure of expert system development (*1*). Chang (*2*,*3*), Maher (*4*), and others have discussed the purely technical details of expert system construction relating to transportation.

BACKGROUND

The TSM approach has been adapted from the extensive TSM literature to suit local conditions and constraints (*5*–*7*).

Tomecki (*6*) defines the TSM process as a seven-stage process as follows:

- Stage 1. Public communications of improvement needs and potential,
- Stage 2. Problem definitions,
- Stages 3 and 4. Generation and analysis of alternative solutions,
- Stage 5. Evaluation and selection of preferred alternative solutions, and
- Stages 6 and 7. Implementation and monitoring.

Stages 1 and 2 are well defined and understood (*7*). A step-by-step procedure has been developed to obtain, through public participation, a range of problem definitions.

Stages 3 and 4 are less straightforward. Extraneous factors, such as the shortage of TSM expertise and the fragmentation of TSM techniques throughout the literature, have affected the generation of effective and wide-ranging solutions to transportation problems brought to light through Stages 1 and 2 of the TSM process. In order to overcome these problems, an expert system was proposed to address Stages 3 and 4 directly.

For each problem identified during Stages 1 and 2 of the TSM process, the expert system would be required to generate a range of solutions (Stage 3). The expert system would assist with the initial analysis (Stage 4) of the generated solutions by providing a weighted certainty factor (*8*) as to the likely effectiveness of such a solution.

These solutions are seen as proposals for further investigation and analysis and serve as a guide to multiple solutions for a given problem.

TSM EXPERT SYSTEM REQUIREMENT DEFINITION

TSM Expert System Feasibility Study

Two simple expert systems were built to investigate the feasibility of implementing an expert system for TSM. The objectives of the feasibility study were to investigate

- The method of representing transportation knowledge (the knowledge representation) that would be most suitable for the TSM project,
- The type of user interface that would be most appropriate, and
- The applicability of expert system technology to transportation planning.

Expert system shells were used for the two systems, including the KES II production system (*9*) (using a production rule or If . . . Then . . . form of knowledge representation) and

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the KES II hypothesis and test (*10*) (using a frame-like form of knowledge representation). These two shells were chosen because they provided different forms of knowledge representation, but were similar in all other respects.

Transportation information needed for both systems was obtained from the tables within the *Simplified Procedures for Evaluating Low-Cost TSM Projects* (*5*).

Construction of these feasibility study expert systems took place over a 4-month period. When completed, both expert systems were able to perform at an acceptable level and a decision was made to implement the TSM expert system.

TSM Expert System Requirement Specifications

The feasibility study gave valuable practical insight into applying expert system technology to the TSM process. The following requirements were specified for the TSM expert system on the basis of the experience gained during the feasibility study:

1. The production rule form of knowledge representation would be used for the TSM expert system. In practice, it was found that both of the methods for representing transportation knowledge discussed previously were effective in modeling transportation problems. However, the production rule method was found to be more understandable by the transportation engineers involved in the project.
2. Type and form of input to the expert system should be clearly specified. For example, information, such as traffic flow, may be given in numeric terms as vehicles per hour or as symbolic values such as low, medium, high, or saturated. Availability of such information for a given problem area or site also needs to be taken into account.
3. Type and form of the output should be clearly specified and all recommendations made by the system should be clearly understood. The feasibility study systems indicated that recommendations made by the expert systems were found to be ambiguous and were not well understood by users.
4. An intelligent front end program should be added to the expert system. The feasibility study found that lengthy question-and-answer sessions between the expert system and a user (typically 30 to 40 questions) often resulted in confusion on the part of the user. This problem was overcome by adding a program to the expert system to assist with the initial capture of information.
5. The sequence of questions asked by the expert system should, as closely as possible, mimic the question sequences and style of a typical transportation engineer or human expert. The feasibility study system contained no instructions for controlling the sequence of questions asked by the expert system. Questions tended to be presented to the user in an illogical sequence. Subjective performance of the expert system improved dramatically when structures (available in many expert system shells) were used to order the question sequence.

In addition to these requirements, the feasibility study demonstrated the importance of a clearly defined knowledge engineering methodology.

STRUCTURING AND ANALYZING THE TSM KNOWLEDGE

The following discussion focuses on the structuring and analysis of TSM knowledge and the development of production rules (If . . . Then . . . statements) for use in the expert system shell. This process, called knowledge engineering, is critical to the success of an expert system project. It is essential that a coherent approach is used to obtain and structure the knowledge within the domain to be modeled (*10,11*).

Modeling the Transportation Engineer's Approach to Problem Solving

During the feasibility study, an intense 1-day session was held to identify how transportation engineers use the TSM process to assist in the solution of transportation problems. The session also served to familiarize the expert system builder with transportation concepts.

The discussion showed that transportation engineers often use a broad two-step process when using TSM for problem solving.

Step 1. Overview Questions

Overview questions were used to obtain general information on the type and location of the problem. First, engineers required information on the location of the problem. This information was used to choose an appropriate problem category (e.g., isolated intersection, corridor, and employment center). Next, within each problem category, information was required on specific topics relevant to the problem area. For example, when the problem category was isolated intersection, the engineer would require information on geometric layout, traffic condition, problem symptom, etc. Discussions indicated that there was a specific set of topics for each problem category.

An example of the type of questions asked during the overview question step would be: "What form of traffic control is being used at the intersection?"

Step 2. Detailed Questions

Information obtained during the overview questioning was then used to guide the engineer in asking specific detailed questions relevant to the problem under consideration. An example of the type of questions asked during the detailed question step would be: "You have said that queues are forming in the left-hand lane at the intersection. Does this happen throughout the day or only at peak times?"

Structuring TSM Knowledge

In order to provide structure for the body of knowledge to be modeled in an expert system, it is necessary for the expert system builders to make a thorough study of the broad areas making up the knowledge domain. Much of the early work is devoted to this process. Fortunately, in TSM methodology much of the work on the structuring of the knowledge domain was directly available in the TSM documentation. TSM documentation (*5*) proved to be an excellent source for much of the knowledge engineering work of the project.

In order to mimic the approach followed by engineers, the knowledge area or domain was divided into seven problem categories:

- Isolated intersections,
- Street segments,
- Corridors,
- Residential communities,
- Employment centers,
- Commercial centers, and
- Regional, system-wide.

Each problem category was then treated as a separate expert system. To date, isolated intersection and street segment systems have been completed. Work is continuing on the other five expert systems.

Once work began on isolated intersections, discussions were held with transportation engineers to identify the major topics or areas of required information (ARIs) for which overview information was required. The following ARIs were identified:

- Geometric layout,
- Traffic control,
- Traffic conditions,
- Traffic problem symptoms,
- Pedestrian conditions,
- Pedestrian problem symptoms, and
- Actions.

In order to ensure that the system followed, wherever possible, the natural question sequence used by a human expert, an informal dependency graph was developed for the ARIs within each problem category.

The dependency graph idea has proved to be useful in diagrammatically representing the dependency relationships between the various ARIs within a problem category. Dependency graphs were used to describe the usual question sequence used for obtaining ARI information. Figure 1 shows the complete dependency graph for the problem category of isolated intersection. Within this dependency graph, it can be seen that intersection control is dependent on intersection traffic condition and intersection geometric layout.

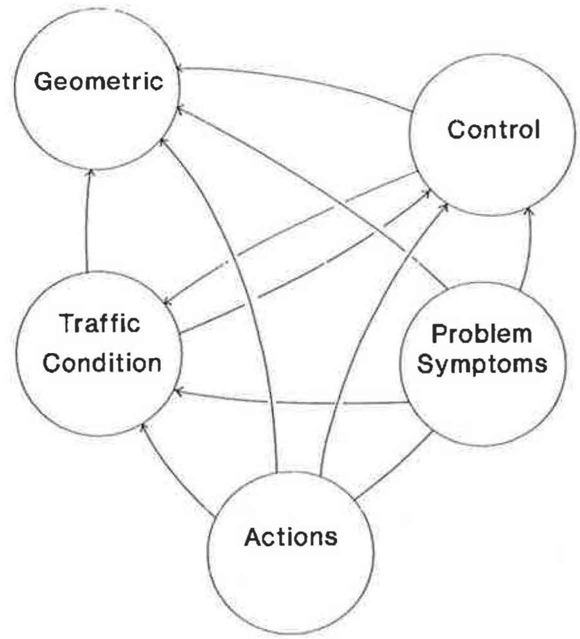

FIGURE 1 ARI dependency graph for ISOLATED INTERSECTION.

Analyzing the TSM Knowledge

After all the ARIs for a particular problem category were defined, the knowledge required for each ARI was analyzed and placed in a form acceptable to the expert system shell used for the project. All possible values for each ARI were then listed. A similar approach is outlined in Weiss and Kulikowski (*12*). Formally, the ARI values may be seen as elements of each ARI set. For intersection geometric layout, the ARI values are

- Slip lane,
- Left-turn storage bay,
- Median—open,
- Median—closed,
- High-occupancy-vehicle lanes, and
- Unorthodox layout.

ARI values for intersection control are

- No control,
- Stop signs,
- Yield signs,
- Traffic lights—fixed time,
- Traffic lights—semiactuated, and
- Traffic lights—fully activated.

Because the actions for each problem category may also be seen as an ARI, the ARI values (list of all possible actions for a problem category) were compiled from the TSM documentation (and augmented by actions relevant to local conditions). These ARI value lists formed the basis for the production rules (If . . . Then . . . statements) used in the knowledge base of the expert system.

Examples of actions taken from the ARI value list for the problem category of isolated intersection are as follows:

- Add a left-turn storage bay,
- Add a right-turn lane,
- Add a left-turn arrow phase, and
- Upgrade intersection layout.

The complete list of actions that were considered was compiled from the broad TSM literature.

The analysis to be discussed and the structuring discussed previously were then used to develop the production rules or If . . . Then . . . statements used by the expert system. Figure 2 shows an example production rule developed from the isolated intersection problem category that illustrates the concepts previously discussed. In this example, the isolated intersection ARIs, ordered per the dependency graph, were geometric, traffic control, traffic condition, traffic symptom, pedestrian condition, and pedestrian symptoms. For each of these ARIs, ARI values (i.e., no left-turn storage bay, any, saturated or high or medium, etc.) were used in developing the production rule example. The outcome action was obtained from the action list for isolated intersections.

Once the basic rule structure was defined, it was possible to systematically acquire the knowledge relating to the area being modeled. The following discussion describes the steps used to develop the detailed If . . . Then . . . information.

Eliciting the Detailed Rules

Interviews were held with experienced transportation engineers to define the rules in the form given previously. The production rules were handled one at a time. Two engineers were involved in each interview session. A third member of the expert system building team acted as a facilitator. Engineers were presented with an empty rule as shown in Figure 3. The engineers were then asked to complete the rule. The ARI value list for each ARI was used as a guideline for completing the rule.

A flexible approach was adopted. Where no value seemed appropriate to the rule being considered, new information was added to the ARI value lists. In several cases during the interview sessions, it was found that the general ARI information was inadequate for selecting a specific action. Once the ARI information was in place, the discussion group identified any specific information that they felt was specific to the rule under discussion. Specific information for the example would be whether space is available for adding a left-turn lane. A possible question generated by the expert system would be: "There are indications that a left-turn storage bay would improve the problem at the intersection. Is there room to add a left-turn lane?"

1. YES,
2. NO.

Knowledge Structuring and Analysis Summary

The underlying operation of an expert system is beyond the scope of this discussion. However, each element in a production rule is closely linked to a question generated by the expert system. For example, geometric layout would generate a question such as: "Which of the following describe the geometry of the intersection?"

1. Slip lane,
2. Left-turn storage bay,
3. Median,
4. HOV lanes, and
5. Unorthodox layout.

The expert system user would respond appropriately.

The rule structure is related to modeling of the transportation engineers' approach to problem solving as follows:

- ARIs in each rule are used to generate overview questions,
- Specialized information is used to generate detailed questions for the specific rule, and
- ARI order is related to the order obtained from the dependency graph and controls the question sequence, which is expert system shell-specific.

```
IF
 GEOMETRIC           = NO  Left turn storage bay      AND
 TRAFFIC CONTROL     = Any                            AND
 TRAFFIC CONDITION   = Saturated or High or Medium    AND
 TRAFFIC SYMPTOM     = Queues in left turn lane
                       impeding straight traffic      AND
 PED CONDITION       = Any                            AND
 PED SYMPTOMS        = Any                            AND
 SPECIFIC INFORMATION
THEN
 ACTION = Add a left turn storage bay.
```

FIGURE 2 Production rule developed from ISOLATED INTERSECTION—with ARI values.

```
IF                                                   F3
 GEOMETRIC            =                               AND
 TRAFFIC CONTROL      =                               AND
 TRAFFIC CONDITION    =                               AND
 TRAFFIC SYMPTOM      =                               AND
 PED CONDITION        =                               AND
 PED SYMPTOMS         =                               AND
 SPECIFIC INFORMATION =
THEN
 ACTION = Add a left turn storage bav.
```

FIGURE 3 Production rule developed from ISOLATED INTERSECTION—without ARI values.

OPERATION OF THE TSM EXPERT SYSTEM

The TSM expert system program has been designed to run on an IBM AT PC or compatible computer. At start up, the user is presented with the list of standard TSM problem categories and the user is required to choose an appropriate category.

The system responds with a form and diagram on the screen. The structure of both the form and the diagram depends on the problem category chosen. In the case of isolated intersections, for example, the user is presented with a stylized diagram of an intersection as shown in Figure 4.

Each solid rectangle indicates a location where a pop-up window can be activated to enter relevant information (for example, traffic flow and pedestrian flow) into the diagram. Geometric, traffic, and pedestrian flow information is entered into the diagram (through a series of pop-up menus). Information is entered separately for each approach to the intersection. A similar diagram form is used to enter problem symptom information for each approach to the intersection.

On completion of the diagram and form, the expert system proceeds with a series of specific questions (the detailed questions). Once adequate information has been obtained from a user, the system responds with a series of recommended actions, which are given in a provisional order.

The complete expert system consists of three interacting subsystems:

1. The KES II production system expert system shell,
2. The Turbo Prolog intelligent front end, and
3. Supporting C language functions.

The KES II system is embedded within the C master program. The Prolog program has been compiled separately, but is executed via a system command from the (master) program.

At startup, the user is given an option to run the expert system using data from

1. A previously stored and named case study, or
2. The previous consultation.

Alternatively, a new consultation can be initiated. The user is then asked to choose the broad problem category. The master C program then runs the Prolog program, which displays the appropriate diagram on the screen.

Because the diagram and ancillary questions such as geometric layout information have been implemented on a virtual screen, it is possible for the user to move around the screen via the edit keys. The virtual screen contains diagrams and entry fields for data required by the system. The F10 key terminates this phase of the program. A second similar diagram, for problem symptoms, is then displayed on the screen. Once again, F10 terminates the input phase of the program.

Information gathered by the Prolog program is then written to a file, which is automatically read by the KES expert system shell.

The program then exhibits typical expert system behavior. Users are asked a series of questions on the basis of the initial input from the diagrams. Conclusions are then displayed on the screen. Users can opt to repeat the consultation, terminate the consultation, or store the data from the entire consultation as a named case study.

KES II PS

The KES II PS shell, using the standard backward chaining inference strategy, was used as the basis of the TSM expert system. The system provided several features that were found to be useful.

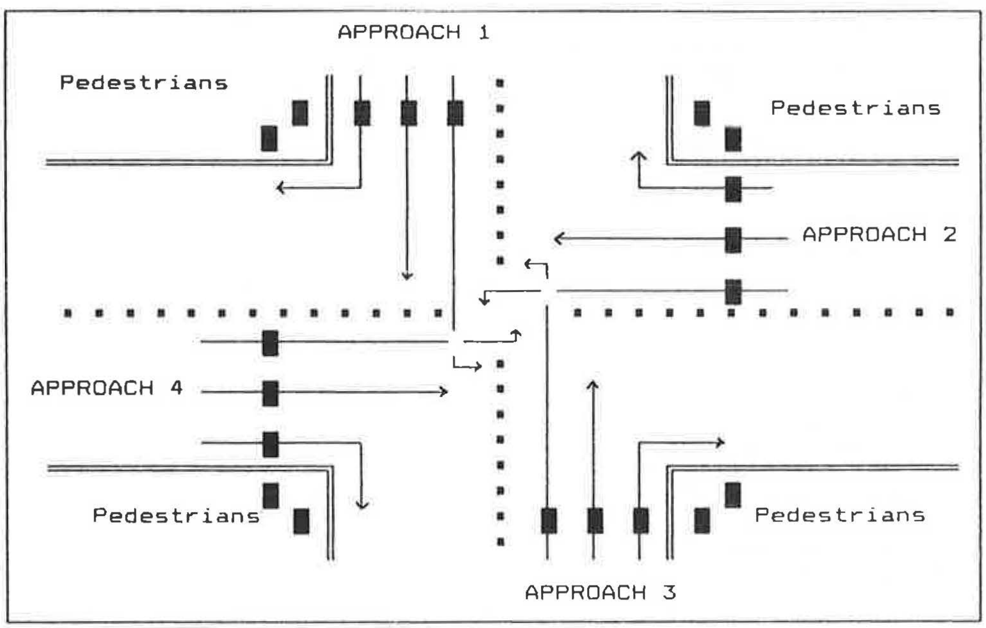

FIGURE 4 TSM expert system stylized diagram of an intersection for ISOLATED INTERSECTION.

- The KES system could be embedded in the C program and viewed as a series of functions to be called when necessary,
- Multiple knowledge bases could be loaded and unloaded from within the C program, and
- The shell has simple, yet powerful file-handling capabilities for transparently interacting with other programs within the DOS environment.

Turbo Prolog

Turbo Prolog 2.0 and the Turbo Prolog Toolbox were used to develop the virtual screens and windows used by the expert system. The Prolog language with its pattern matching ability provided an excellent basis for the development of intelligent forms and diagrams. In addition, the Prolog program was designed to prevent the user from entering contradictory information into the system.

C

Lattice C V 3.01 formed the basis for the complete expert system. It was chosen to maintain compatibility with the C interface to the KES system.

VALIDATING THE EXPERT SYSTEM

Ongoing Validation

Validation and testing of expert system performance is an integral part of expert system construction. The knowledge within the TSM expert system is based on a human expert's interpretation of a given situation or problem and therefore cannot be assumed to be 100 percent correct (*13*).

Physical construction of the TSM expert system was an iterative process—a small number of production rules was added to the system and then adjusted and tested until the system produced satisfactory results. Procedures were repeated until all the rules were added to the system.

However, it was felt that this approach did not provide exhaustive testing of the expert system.

Practical Validation

In order to ensure that the expert system provided useful practical results, the expert system's recommendations were compared with recommendations obtained from an independent study (*14*). The study consisted of an intersection investigation of 14 problem intersections in which the objectives were to

1. Select appropriate intersections for detailed analysis,
2. Recommend improvements to selected intersections that would alleviate existing problems, and
3. Document the procedures used.

Seven of the 14 intersections investigated were four-way intersections directly comparable with the knowledge already in the expert system. At present, the isolated intersection part of the system contains knowledge relevant only to four-way intersections.

Traffic counts from the study were translated into symbolic form (low, medium, high, and saturated) and fed into the expert system. General information gathered during the intersection study was used as the general (overview) information for the expert system.

In five of the seven intersections, recommendations made by the expert system program closely followed the recommendations made by the consultants. However, recommended actions such as "Check the operation of the vehicle actuation, as it is not working properly at present" were not suggested by the expert system because it did not yet contain any information relevant to this type of problem.

In each case, the TSM expert system provided a broader range of recommended actions than those provided by the consultants. This outcome is to be expected because the human expert is inclined to filter out the less than ideal solutions to a given problem. No such filtering mechanism was built into the expert system. On the other hand, the expert system's approach of giving a full range of solutions could be regarded as an advantage because of its consistency and comprehensiveness. Humans are sometimes inclined to get in a rut and offer only their personal and familiar solutions.

The major difference between the recommendations of the expert system and the consultants' study was that the expert system program's results were qualitative and required further investigation and analysis before a detailed recommendation could be implemented.

CONCLUSION

An expert system was constructed that addresses the generation and analysis of alternative solutions in the TSM process. In particular, the steps taken in structuring and analyzing the TSM transportation knowledge (the knowledge engineering) into a form acceptable to the expert system model were described.

Experience gained in the construction of the system indicates that the key to successful expert system construction is in the knowledge engineering. Without a clear understanding of how an expert goes about solving a problem, an expert system project is unlikely to succeed.

The methodology described has been effective in providing a structure for the difficult task of encapsulating human expertise within a computer program to generate multisolutions to problems within TSM.

Knowledge engineering procedures have had a useful indirect benefit on the TSM process. The formal process of gathering and structuring the TSM knowledge from disparate sources has provided a consistent approach to the classifying of knowledge and information within TSM. In addition, because of the expert system's consistent and rapid response to a given problem situation, it is likely that the completed system will provide excellent training in the TSM methodology.

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Implications of Increasing Carpool Occupancy Requirements on the Katy Freeway High-Occupancy-Vehicle Lane in Houston, Texas

DENNIS L. CHRISTIANSEN

The Texas State Department of Highways and Public Transportation and the Harris County Metropolitan Transit Authority are in the process of developing an extensive system of high-occupancy-vehicle (HOV) lanes on the freeways in Houston, Texas. Locally, these HOV lanes are referred to as transitways. Considerable attention is being given to developing appropriate techniques for operating these priority facilities. In October 1988, carpool occupancy requirements to use the lane were increased from two or more to three or more persons per vehicle between 6:45 and 8:15 a.m. in order to restore free-flow operation on the transitway. This change represented the first time in the United States that occupancy requirements to use an HOV facility had been increased. The action had its intended effect of restoring free flow to the transitway. Although in the short run total person volume for the facility declined slightly, the result was a significant increase in the value of time saved by transitway users. Increases both in bus patronage and in three-or-more-person carpool use were noted. This action was implemented with surprising ease and has worked effectively in the field. Much of the success is directly related to the design and enforcement policy used in developing and operating the Houston transitways. This approach may now be used on a routine basis as needed to effectively operate the Houston transitway facilities.

The Texas State Department of Highways and Public Transportation and the Metropolitan Transit Authority of Harris County are in the process of developing an extensive system of high-occupancy-vehicle (HOV) lanes on the freeways in Houston, Texas. Locally, these HOV lanes are referred to as transitways. Today, over 36 mi of these facilities are in operation on four separate freeways. Ultimately, nearly 96 mi of transitways will be developed at a cost approaching $700 million. These lanes are generally located in the median of the freeway, are 20 ft wide, are reversible, and are separated from the mixed-flow traffic lanes by concrete median barriers. A more complete description of this transitway system was given by Christiansen and Morris (*1*).

Because the Houston commitment to developing transitways is somewhat unique and extensive, considerable effort is being given to identifying appropriate procedures for operating the transitways. The Katy (I-10) transitway, Phase 1 of which opened in October 1984, was the first of the transitways to be completed in final form. Consequently, in many respects it has been used as a laboratory in which different operating procedures could be tested.

One of the major operational issues affecting the transitways is the decision regarding what vehicle groups will be allowed to use the transitway. In effect, a balancing act is required. On one hand, it is desirable to have a reasonably large volume of vehicles using the transitway so that it appears to be sufficiently used. On the other hand, for the transitways to be successful they need to offer a high travel speed and a reliable travel time. As a result, it is essential that volumes in the transitway be kept below capacity so that significant delay and congestion do not develop on the high-speed priority lane.

This balancing act is further complicated by two other factors. First, experience with HOV lanes in southwestern and western cities has shown that the two-or-more-person carpool volume can be substantial; the three-or-more-person carpool volume is generally quite small. However, using a three-or-more-person rather than a two-or-more-person carpool designation can reduce carpool volume by 75 percent. Second, transitway facilities have exceedingly high peaking characteristics. Generally, the hourly vehicle volume on either side of the peak hour is about half of the peak-hour volume. Thus, the need may exist to manage the peak-hour volume without adversely affecting the volumes on either side of that peak hour.

ELIGIBLE KATY TRANSITWAY USER GROUPS

Definition of which vehicle types are allowed to use the Katy transitway has changed on several occasions between its opening (in October 1984) and October 1988. When the transitway opened in October 1984, because of previous experience in Houston on the North Freeway (I-45) contraflow lane, only buses and vanpools formally authorized by the Harris County Metropolitan Transit Authority (Metro) were allowed to use the Katy transitway. Authorization involved many factors, including insurance requirements, driver training, and vehicle inspections. Drivers were issued licenses allowing them to operate in the priority lane, and vehicles using the lane displayed permits. With this approach, shortly after it opened approximately 50 vehicles used the transitway in the peak hour. Surveys (*2*) of motorists in the freeway main lanes found

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that 97 percent of those individuals felt that the transitway was being underused.

In April 1985, a decision was made to allow authorized four-or-more-person carpools to begin using the transitway to increase its use. It was found that few four-or-more-person carpools existed in the Houston traffic stream and that a carpool of that size was relatively unstable on a day-to-day basis (because of at least one person not traveling to the place of work that day). As a result, the effects of this action were minimal; only about 10 vehicles per hour (vph) were added to the peak-hour volume.

In September 1985, three-or-more-person authorized carpools were allowed onto the Katy transitway, which increased peak-hour volume to about 100 vph, but the transitway still appeared underused.

In April 1986, two-or-more-person carpools were allowed to use the transitway and all occupancy requirements were dropped. The peak-hour volume immediately increased to about 1,200 vph, and for 2 years this approach worked relatively well. The volume both of persons and vehicles using the transitway was significant and relatively high travel speeds continued to exist in the transitway.

KATY TRANSITWAY VOLUME AND CAPACITY RELATIONSHIPS

In September 1988, with the economy in the Houston area beginning to rebound, volumes using both the freeway main lanes and the transitway began to increase noticeably. Peak-hour volumes on the transitway frequently would approach or exceed 1,500 vph. Several site-specific geometric and operational constraints limit the capacity of the Katy transitway. Given these constraints, traffic analysis (*3*) showed that delays would begin to occur on the transitway as volumes exceeded about 1,200 vph, and that 1,500 vph effectively was the upper volume level that could be served with reasonably reliable travel speeds. Speeds during the peak of the peak hour were below 55 mph at these volumes. Because the eastern terminus of the transitway is temporarily located at a traffic signal, delay problems on the transitway itself occurred only during a.m. operation.

As demands began to approach and exceed 1,500 vph, the purpose of the transitway to provide travel time advantages began to be lost. Considerable delays occurred on the transitway during the a.m. peak hour, and bus passengers began complaining to the transit authority.

In response to this problem, studies (*3*) of alternatives for managing demand were undertaken. Consideration was given to (a) doing nothing, (b) requiring authorization for two-person carpools desiring to use the transitway in the peak hour, (c) metering access to the transitway, and (d) increasing carpool occupancy requirements. All of the alternatives considered had problems; there was no obvious best alternative. A policy-level decision was made to increase carpool occupancy requirements from two or more to three or more persons per vehicle for the period from 6:45 to 8:15 a.m., but the two-or-more-person policy would remain in effect during all other operating hours. The decision was implemented on 3 days' notice with relatively little marketing and became effective October 17, 1988.

This decision represented an innovative approach for operating transitway facilities. It was the first time a carpool occupancy requirement had been increased on a HOV facility, and it also was the first time that HOV requirements were varied by time of day (some HOV facilities do revert from HOV lanes to regular mixed-flow freeway lanes during off-peak periods).

IMPACTS OF THE INCREASE IN OCCUPANCY REQUIREMENTS

The increase in carpool occupancy requirements between 6:45 and 8:15 a.m. was implemented with surprisingly little difficulty. The relatively unique design (barrier-separated transitways with a limited number of access and egress locations) and regular routine enforcement associated with the transitways greatly enhanced the feasibility of this demand management approach. Data are available through March 1989 to permit evaluation of at least the short-term impacts of this action. Data relevant to the analysis are presented in Table 1.

Morning Transitway Operations

7:00 to 8:00 a.m. Transitway Travel

Between 7 and 8 a.m., the total peak-hour vehicle volume on the transitway immediately decreased by about 64 percent, from 1,400 to 510 (Table 1). Travel time delays that had been experienced on the transitway before the occupancy change were immediately eliminated (Figure 1). To that end, the change in occupancy requirements achieved its desired effect.

Since the initial decrease of about 33 percent in person-volume on the transitway between 7 and 8 a.m., demand has been increasing. For March, the person-volume increased to 3,445, 19 percent less than the volume before the change but 18 percent greater than the November–December volume.

Because the decline in vehicle-volume was greater than the decline in person-volume, average vehicle occupancy on the transitway increased from 3.1 to 4.7 persons per vehicle. The data in Table 1 also indicate that a significant volume of two-person carpools are on the transitway between 7 and 8 a.m. Some of these are clearly violators; however, most appear to have legally entered the transitway before 6:45 a.m. at its western terminus and were still in the transitway at 7:00 a.m. when counted at the eastern terminus.

6:00 to 9:30 a.m. Transitway Travel

During the a.m. peak period, person-volume immediately dropped by 17 percent; however, it has been increasing and in March was 10 percent less than what it was before changing the occupancy requirement (Figure 2).

Components of the change in person volumes Before the change in occupancy requirements, approximately 5,090 persons used the transitway in two-person carpools between 6 and 9:30 a.m. (Table 1). This figure decreased to 2,490 in the

TABLE 1 MORNING TRAVEL VOLUMES BEFORE AND AFTER CHANGE IN OCCUPANCY REQUIREMENTS, KATY FREEWAY CORRIDOR

Travel Volumes	"Representative" Pre-Occupancy Change Value[1]	Value After Occupancy Change 11/88 and 12/88 Value[2]	% Change[3]	3/89 Value	% Change[3]
Daily Transitway Person Volume	18,880	16,595	- 12%	17,831	- 6%
A.M. Peak-Period (6-9:30) Person Volume, Total	8,780	7,265	- 17%	7,945	- 10%
2 Person Carpools	5,090	2,490	- 51%	2,800	- 45%
3+ Person Carpools	935	1,835	+ 96%	1,905	+ 104%
Total, Carpool Riders	6,025	4,325	- 28%	4,705	- 22%
Patrons	2,450	2,670	+ 9%	2,885	+ 18%
Vanpool Riders	305	270	- 11%	355	+ 16%
7-8 A.M., Total Person Volume	4,320	2,915	- 33%	3,445	- 19%
Carpools	2,885	1,315	- 54%	1,750	- 39%
2 Person Carpools	2,410	230	- 90%	480	- 80%
Bus Patrons	1,310	1,500	+ 15%	1,490	+ 14%
Vanpoolers	125	100	- 20%	205	+ 64%
A.M. Peak Period Vehicle Volume, Total	2,900	1,950	- 33%	2,120	- 27%
Carpools	2,780	1,820	- 34%	1,990	- 28%
7-8 A.M., Total Vehicle Volume	1,400	510	- 64%	730	- 48%
2+ Carpool Vehicles	1,365	455	- 67%	660	- 52%
2 Person Carpools	1,205	115	- 90%	240	- 80%
3+ Carpools	160	340	+112%	420	+ 162%
Carpool Volume (6-7 and 8:15-9:30)	1,230	1,170	- 5%	1,295	+ 5%
Freeway Mainlane Volumes, 6-9:30 a.m.					
Vehicles	15,300	15,900	+ 4%	16,805	+ 10%
Total Persons	16,455	17,230	+ 5%	18,675	+ 13%
Average Vehicle Occupancy	1.075	1.084	+ 1%	1.111	+ 3%

[1]This is the value representative of the trend line that existed prior to changing the occupancy requirement. It does not reflect the values for any particular month.

[2]These are representative of the average of the November and December data.

[3]The percent change in comparison to the representative pre-occupancy change value.

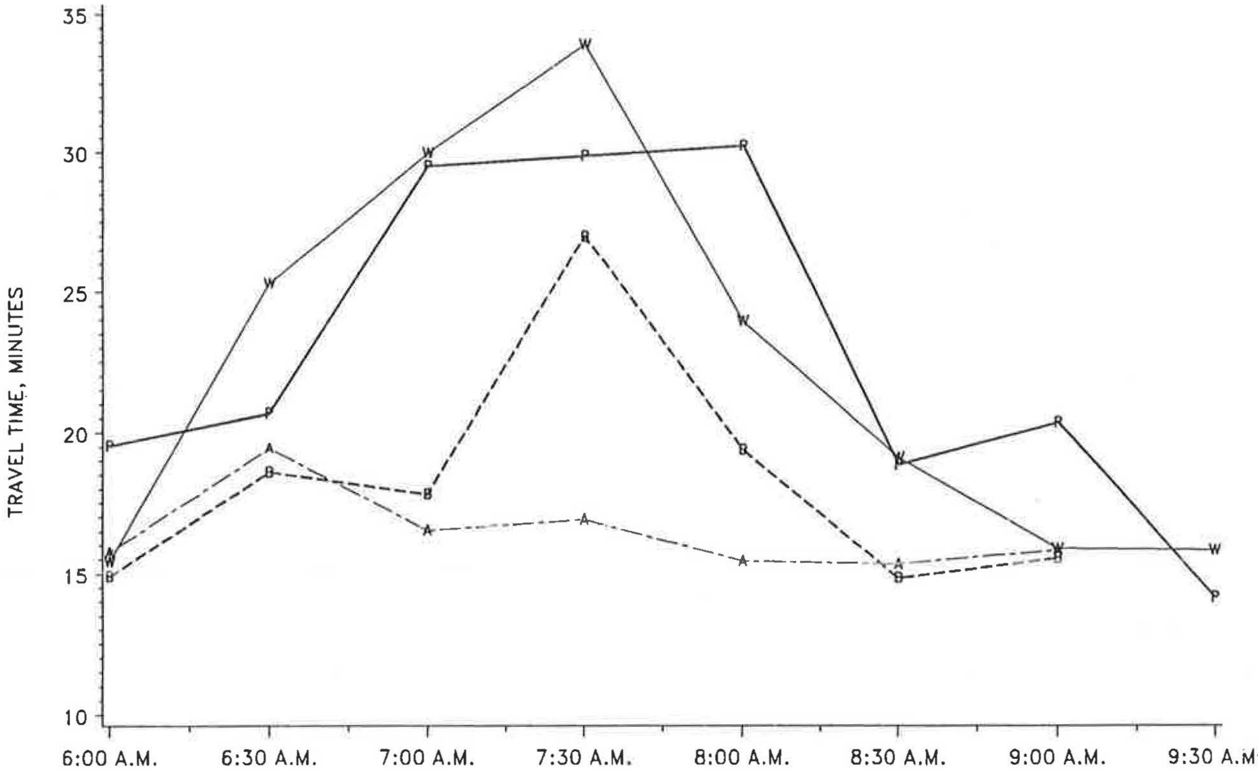
Source: Texas Transportation Institute data collection.

November–December period and was 2,800 in March. Thus, if all the individuals in those two-person carpools had ceased to use the transitway, the apparent loss in transitway ridership in the November–December period would have been 2,600 persons, and in March, 2,290 persons. Actual declines in peak-period transitway ridership were 1,515 and 835 for those periods, respectively. It is apparent that some changes have occurred in transitway travel patterns as a result of the changed occupancy requirement.

Table 2 presents the changes that have occurred in peak-period transitway ridership since the change in occupancy requirements. They indicate that a significant volume of individuals has changed to a higher-occupancy mode (either three-or-more-person carpool or bus) to be able to keep using the transitway.

Through March, a 104 percent increase in three-or-more-person carpool person-volumes had been realized, which occurred almost immediately (Figure 3). It is also significant that bus ridership in the a.m. peak period had increased by nearly 20 percent through March. Apparently, there is some modal overlap because some individuals, if necessary, will choose a higher-occupancy mode of travel.

Changes in Time of Use of the Transitway It would be expected that carpool volumes between 6:30 and 7:00 a.m. might have increased as a result of the change in occupancy requirements. Overall, carpool volumes now peak earlier than they did before the occupancy change, but the absolute volume of carpools using the transitway between 6:00 and 7:00

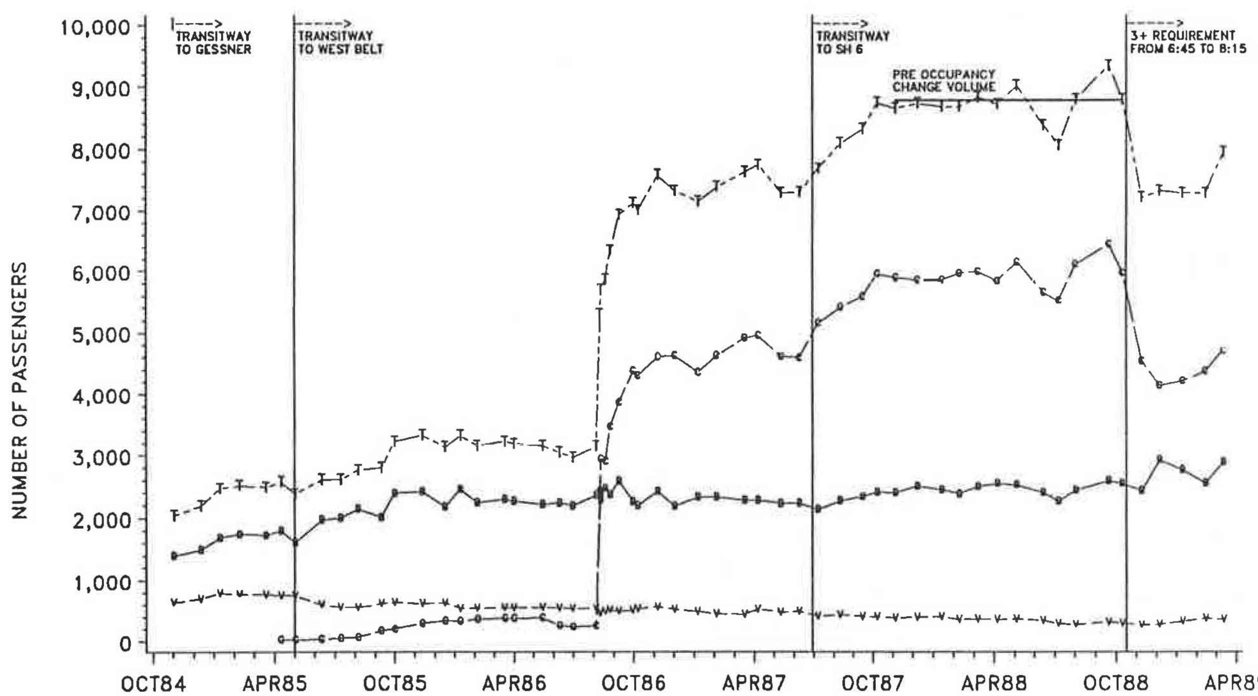


TRAVEL TIMES ARE FROM THE WESTERN TRANSITWAY TERMINUS TO THE S.P. RAILROAD
3+ CARPOOL REQUIREMENT FROM 6:45 TO 8:15 IMPLEMENTED OCTOBER 17, 1988

SOURCE : TEXAS TRANSPORTATION INSTITUTE

LEGEND : P – MAINLANE TRAVEL TIME BEFORE 3+ CHANGE (AVG. OF 3/88 & 6/88)
W – MAINLANE TRAVEL TIME AFTER 3+ CHANGE (AVG. OF 12/88 & 3/89)
B – TRANSITWAY TRAVEL TIME BEFORE 3+ REQUIREMENT
A – TRANSITWAY TRAVEL TIME AFTER 3+ REQUIREMENT

FIGURE 1 Katy Freeway main lanes and transitway, a.m. travel times.



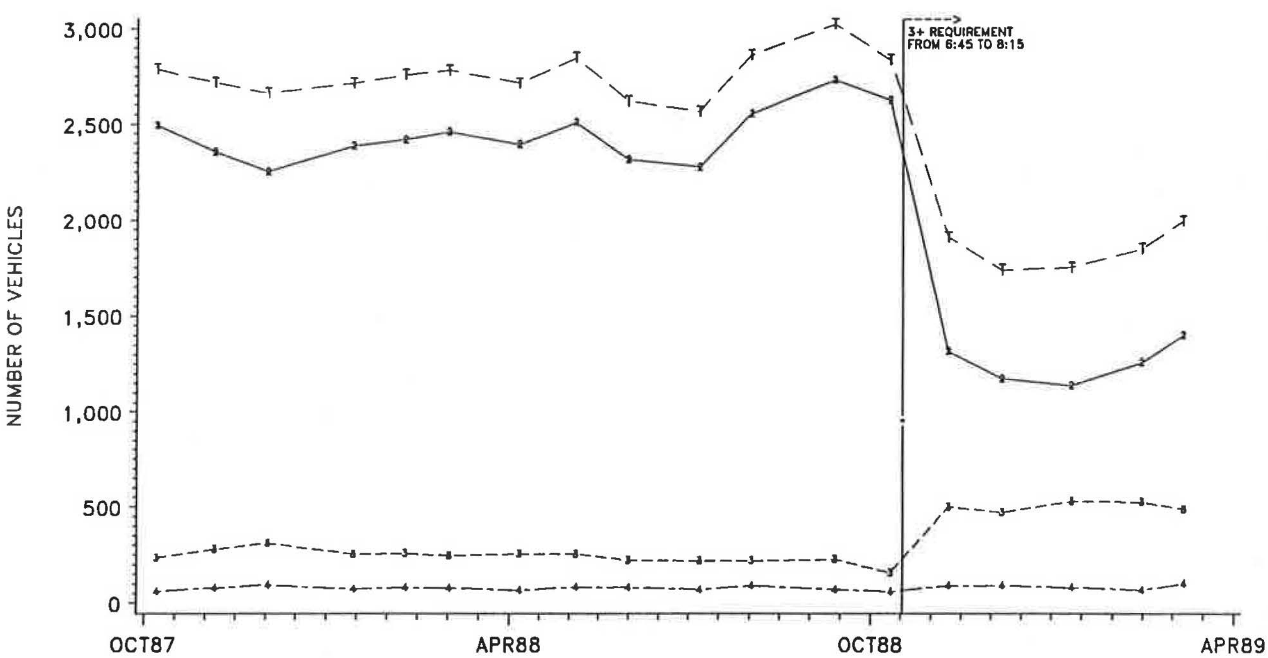
KATY TRANSITWAY PHASE 1, POST OAK TO GESSNER (4.7 MI.), OPENED OCTOBER 29, 1984
TRANSITWAY EXTENSION FROM GESSNER TO WEST BELT (1.7 MI.) OPENED MAY 2, 1985
OFF-PEAK, UNAUTHORIZED & 2+ CARPOOL OPERATION BEGAN AUGUST 11, 1986
TRANSITWAY EXTENSION FROM WEST BELT TO SH 6 (5.0 MI.) OPENED JUNE 29,1987
3+ CARPOOL REQUIREMENT FROM 6:45 TO 8:15 A.M. IMPLEMENTED OCTOBER 17, 1988
PEAK PERIOD IS 6:00 – 9:30 A.M.
DATA COLLECTED BETWEEN GESSNER AND POST OAK
SOURCE : TEXAS TRANSPORTATION INSTITUTE

LEGEND : T = TOTAL HOV PASSENGERS
B = TOTAL BUS PASSENGERS
V = TOTAL VANPOOLERS
C = TOTAL CARPOOLERS

FIGURE 2 Katy Freeway transitway, a.m. peak-period person movement.

TABLE 2 SUMMARY OF CHANGES IN a.m. PEAK-PERIOD PERSON TRAVEL ON THE KATY TRANSITWAY

Component of Change from Base Ridership	November-December Time Period	March Time Period
Base Ridership (Pre-Occupancy Change)	8,780	8,780
Change Due to Vanpooling	- 35	+ 50
Change in 2-Person Carpool Volume	-2,600	-2,290
Change in 3+ Person Carpool Volume	+ 900	+ 970
Change in Bus Patronage	+ 220	+ 435
Resulting Peak Period Ridership	7,265	7,945



KATY TRANSITWAY PHASE 1, POST OAK TO GESSNER (4.7 MI.), OPENED OCTOBER 29, 1984
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TRANSITWAY EXTENSION FROM WEST BELT TO SH 6 (5.0 MI.) OPENED JUNE 29, 1987
4+ AUTHORIZED CARPOOL OPERATION BEGAN APRIL 1, 1985
3+ AUTHORIZED CARPOOL OPERATION BEGAN SEPTEMBER 1985
OFF-PEAK, UNAUTHORIZED & 2+ CARPOOL OPERATION BEGAN AUGUST 11, 1986
3+ REQUIREMENT FROM 6:45 T 8:15 A.M. IMPLEMENTED OCTOBER 17, 1988
SOURCE : TEXAS TRANSPORTATION INSTITUTE

LEGEND : T = TOTAL CARPOOLS
2 = TOTAL 2 PERSON CARPOOLS
3 = TOTAL 3 PERSON CARPOOLS
4 = TOTAL 4 PERSON CARPOOLS

FIGURE 3 Katy Freeway transitway, a.m. peak-period carpool use.

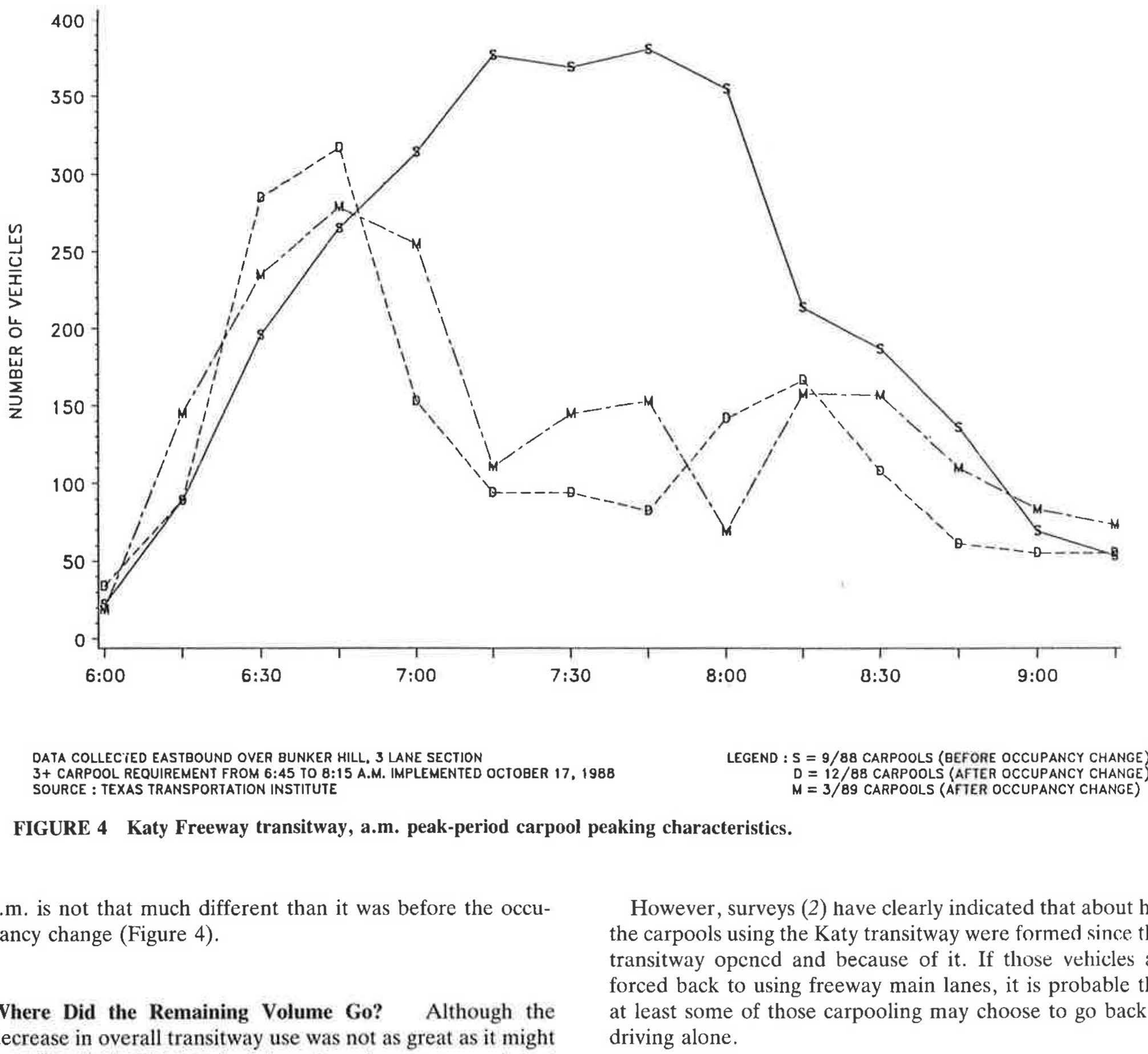

FIGURE 4 Katy Freeway transitway, a.m. peak-period carpool peaking characteristics.

a.m. is not that much different than it was before the occupancy change (Figure 4).

Where Did the Remaining Volume Go? Although the decrease in overall transitway use was not as great as it might have been had not a meaningful number of commuters switched to a higher-occupancy mode, nevertheless, fewer people used the transitway during the peak period. Compared with conditions that existed before the occupancy change, in the November–December period the person-volume was 1,515, whereas in March it was 835.

It had been speculated that some portion of this volume may have been diverted to the northwest (US-290) transitway, a new transitway partially in the same corridor as the Katy transitway and still open to two-or-more-person carpools during all operating hours. However, an analysis of trends in use on the northwest transitway suggests that no significant diversion to that transitway took place.

It seems that most of the volume no longer using the Katy transitway has diverted back to either using the Katy Freeway main lanes or using other streets in the corridor. Indeed, freeway volumes have increased (Table 1) although it is not possible to clearly identify the components of that increase. Small increases in freeway vehicle occupancy have also occurred, suggesting that additional carpools are now in the freeway main lanes.

However, surveys (*2*) have clearly indicated that about half the carpools using the Katy transitway were formed since that transitway opened and because of it. If those vehicles are forced back to using freeway main lanes, it is probable that at least some of those carpooling may choose to go back to driving alone.

Evening Transitway Operations

During the p.m. peak period (3 to 6:30 p.m.), the transitway is still open for use by two-or-more-person vehicles. As a result, it would be expected that meaningful changes in person-volume should not occur; however, a decline in vehicle volume would be expected because there are more bus riders and more three-or-more-person carpoolers caused by the actions taken in the a.m. peak period. In general, this has been the case (Table 3). By March, the increasing trend in p.m. person-volume was back in evidence and compared with preoccupancy change conditions, peak-period person volume was up 4 percent with vehicle-volume being down 4 percent.

Daily Transitway Travel Volumes

As would be expected, reducing the types of vehicles that can use the transitway during a portion of the a.m. peak would,

TABLE 3 EVENING PEAK-PERIOD (3 TO 6:30 p.m.) TRANSITWAY TRAVEL VOLUMES BEFORE AND AFTER CHANGE IN OCCUPANCY

Travel Volume	"Representative" Pre-Occupancy Change Value[1]	Value After Occupancy Change			
		11/88 and 12/88		3/89	
		Value[2]	%Change	Value	%Change[3]
Peak Period Person Volume	8,325	8,180	-2%	8,682	+4%
Peak Period Vehicle Volume	2,825	2,665	-6%	2,714	-4%

[1]This is the value of the trend line that existed prior to changing the occupancy requirement. It does not reflect the values for any particular month.

[2]These are representative of the average of November and December data.

[3]The percent change in comparison to the representative pre-occupancy change value.

Source: Texas Transportation Institute.

at least in the short run, reduce total transitway use. Compared with the conditions that existed before changing the occupancy requirement, the November–December period experienced a 12 percent decrease in daily travel. However, demand has been increasing, and in March 1989 the daily person-volume on the transitway was 6 percent less than what it was before changing the occupancy requirement (Table 1).

Value of Transitway Travel Time Saved

Although person-volumes on the transitway declined, the increase in travel time saved was substantial. This finding is partly the result of eliminating delay on the transitway and partly the result of increased congestion on freeway main lanes (Figure 1). In March 1989, travel time savings for users of the transitway were greater than they were before initiating the occupancy change requirement (Table 4). Most of the 32 percent increase in person-time saved during the a.m. peak period can be attributed to the occupancy change.

CONCLUSIONS

In order to restore high speeds and reliable travel times on the Katy transitway, occupancy requirements for carpools were increased from two or more to three or more persons between 6:45 and 8:15 a.m. in October 1988. This increase had its intended effect of immediately eliminating congestion on the transitway.

This change represented the first time carpool occupancy requirements had been increased on a HOV facility. Although considerable concern existed over whether this could be done, the change was actually accomplished with relative ease. Given the design and enforcement associated with the Houston transitways, it has been possible to enforce this restriction. The change in occupancy requirements became insignificant within several days of being implemented. Although this action directly affected over 2,000 peak-hour commuters, fewer than 36 calls were received by the operating agencies complaining about or commenting on the measures taken. Apparently, those persons using the transitway realized that the value of that facility was being greatly reduced by the high vehicle volumes.

The action resulted in many individuals choosing to use a higher-occupancy travel mode. By March 1989, peak-period bus ridership, compared with conditions before the occupancy change, had increased by 435 riders or 18 percent. Three-or-more-person carpool person-volume in the peak period increased by 970 persons, or 104 percent.

By March, daily person usage of the transitway had increased to within 6 percent of the volume that existed before the change. However, although person-volume decreased, at least in the short run, the value of time saved by users of the transitway increased substantially because of the elimination of congestion on the transitway and the increase in congestion on the freeway main lanes. The result was a 90 percent increase in the value of time saved daily by users of the transitway. During the a.m. peak period, person-hours of time saved by users of the transitway on nonincident days increased from 833 to 1,100 hr, an increase of 32 percent. Much of this increase is because of the change of occupancy requirements.

The Houston transitways are intended to move a design-year volume of 7,000 to 10,000 persons in the peak hour. This volume simply cannot be realistically attained with a two-or-more-person occupancy requirement. As a result, it was recognized that at some point in time peak-hour occupancy requirements would have to be increased. That action has now been taken successfully. This successful experiment has shown that, given the design and enforcement procedures associated with the Houston transitways, an effective operating tool can be used to help manage transitway demand to ensure that those facilities function as planned. In the future, this approach may be used on a routine basis as needed to effectively operate other Houston transitways.

TABLE 4 DAILY PERSON-HOURS OF TIME SAVED BY USERS OF THE KATY TRANSITWAY

Time Period	Hours of Time Saved		
	Representative Pre-Occupancy Change Value[1]	Value after Occupancy Change	
		Value[2]	% Change[3]
A.M. Peak Period	833	1,100	+ 32%
P.M. Peak Period	202	858	+325%
Total	1,035	1,958	+ 89%

[1]This is the average of travel time data collected in 12/87, 3/88 and 6/88. Travel time saved due to incidents is not included.

[2]This is the average of travel time data collected in 12/88 and 3/89. Travel time saved due to incidents is not included.

[3]The percent change in comparison to the 9/88 value pre-occupancy change value.

Source: Texas Transportation Institute.

ACKNOWLEDGMENT

Since 1974, the Texas State Department of Highways and Public Transportation has sponsored an on-going research effort pertaining to priority treatment for high-occupancy vehicles. In more recent years, the Harris County Metropolitan Transit Authority has also been actively involved in this research program. The oversight and funding provided by the sponsoring agencies is gratefully acknowledged.

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Publication of this paper sponsored by Committee on High-Occupancy Vehicle Systems.

Status and Effectiveness of the Houston High-Occupancy-Vehicle Lane System, 1988

DENNIS L. CHRISTIANSEN

The Houston high-occupancy-vehicle (HOV) lane system is evaluated through calendar year 1988. Locally, these HOV lanes are referred to as transitways. These facilities are being built primarily as a means to help cope with the congestion problems in the Houston area. By the end of 1988, 36.6 mi of transitways were in operation on four Houston freeways. Transitways are generally located in the median of the freeway, are 20 ft wide, are reversible, and are separated from the freeway mixed-flow lanes by concrete median barriers. Ultimately, 95.5 mi of transitways will be constructed at a cost approaching $700 million. Surveys indicate that development of these transitways has public support. The primary objective of the Houston transitways is assumed to be to increase, in a cost-effective manner, the person-movement capacity of a freeway and to do it in a manner that does not unduly affect the operation of the freeway's general-purpose mixed-flow lanes. Transitway design and operation in Houston have not unduly impacted the general-purpose freeway lanes. Data indicate that the transitways can significantly increase peak-period person movement and average vehicle occupancy. New bus riders and carpools are generated by the facilities. For a transitway with a Houston-type design to be successful and cost-effective, it may need to offer a peak-hour travel time savings of at least 6 to 8 min compared with operation in the freeway mixed-flow lanes. The transitway also needs to move over 10,000 person-trips per day.

In Houston, in the early 1970s, increases in travel demand, expressed as freeway vehicle-miles of travel (VMT), began to exceed increases in roadway supply, expressed as lane-miles of freeway. Since 1970, VMT per freeway lane-mile has increased by approximately 100 percent. As a result, congestion also increased significantly and a 1984 FHWA study (*1*) found that Houston had some of the most, if not the most, congested freeway facilities in the nation. Monitoring of overall urban congestion in major Texas cities has clearly indicated that mobility levels in Houston have become undesirable (*2*). However, at the same time, congestion in Houston has been moderating in recent years. Nevertheless, the congestion problem in Houston is serious and continues to require attention.

In response to this congestion problem, a variety of actions are being taken. One involves the implementation on the urban freeways of a system of priority lanes for high-occupancy vehicles. Locally, these high-occupancy-vehicle (HOV) lanes are commonly referred to as transitways and are being jointly developed by the Texas State Department of Highways and Public Transportation and the Metropolitan Transit Authority of Harris County (Metro).

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As part of an ongoing research effort, a comprehensive evaluation of these transitway facilities is being performed. Evaluations are being conducted using two approaches. First, before and after trend line data being collected for each freeway on which a transitway is being developed provide a means for identifying changes that occur in those corridors. Second, similar data are being collected in corridors that do not have transitways. These control corridors help to isolate the specific impacts of the transitways.

Data relative to transitway and freeway operations and effectiveness in Houston are presented and evaluated through December 1988. Data are presented for all four operating transitways.

OVERVIEW OF THE HOUSTON TRANSITWAY SYSTEM

A commitment has been made to develop approximately 96 mi of freeway transitway in the Houston area (Figure 1). As of December 1988, four separate transitway facilities had been opened with a total of 36.6 mi of transitway in operation. Daily operation and enforcement of these facilities are the responsibility of the Metropolitan Transit Authority (Metro). Selected characteristics of the operating transitways are presented in Table 1.

Although some sections of two-direction transitway are being developed, the typical Houston transitway is located in the freeway median, is approximately 20 ft wide, is reversible, and is separated from the general-purpose freeway main lanes by concrete median barriers. In some locations, transitway implementation was accomplished by narrowing freeway main lanes and inside shoulder width.

Access to the median transitways is provided in a variety of manners. At some locations, slip ramps are used to provide access and egress to and from the inside freeway lane. Openings in the barriers allow direct access to the transitway. Although slip ramps are relatively inexpensive, they have a variety of operational disadvantages. As a consequence, most access to these median transitways is being provided by grade-separated interchanges of various designs. With these designs, the transitway becomes elevated in the freeway median and grade-separated ramps provide connections to surface streets, park-and-ride lots, bus transfer centers, etc. These grade-separated interchanges are typically constructed at a cost in the range of $2 to $5 million each.

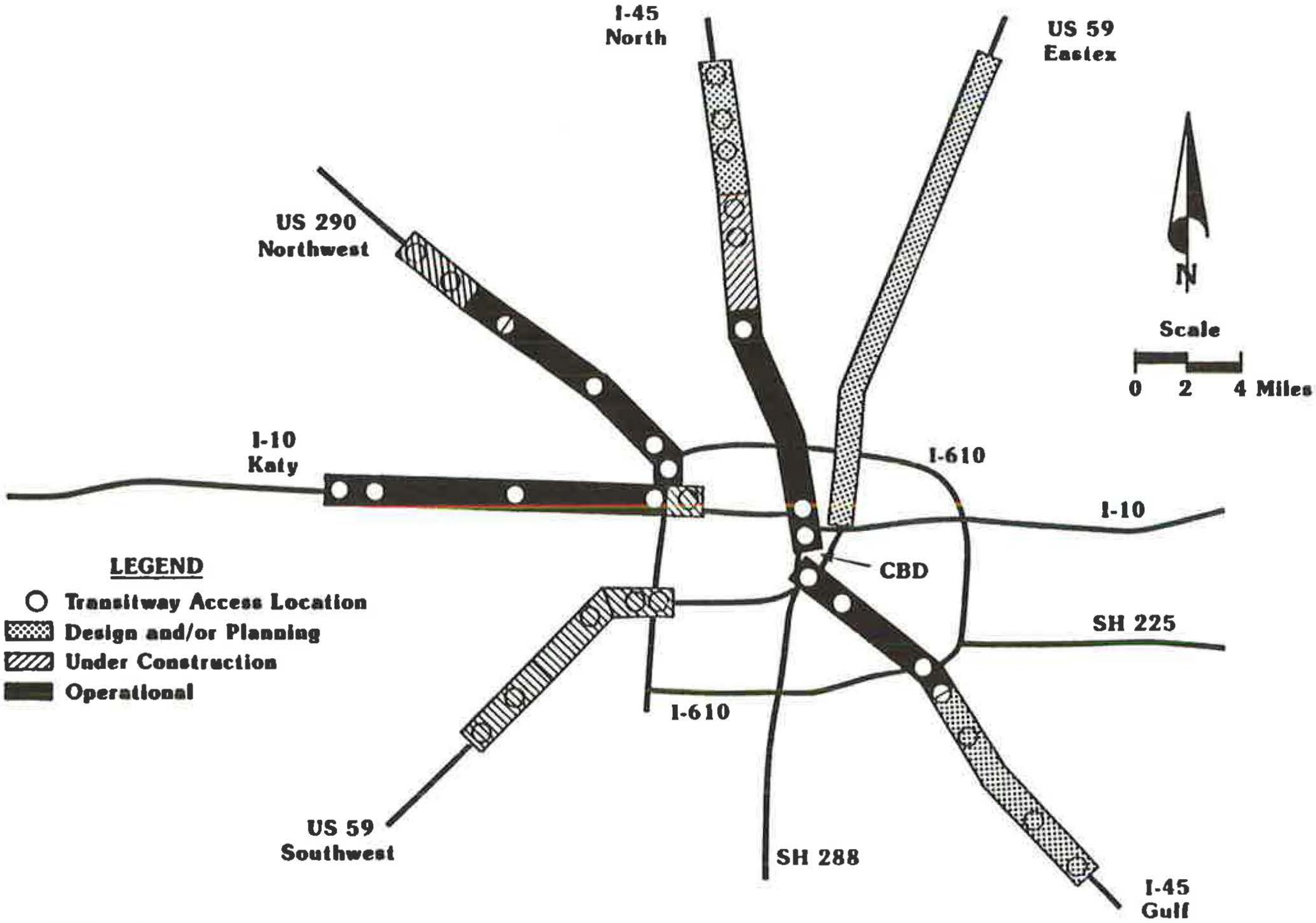

FIGURE 1 Status of Houston transitway development, March 1989.

The estimated capital cost of the entire 96 mi system is approximately \$689 million, or about \$7.2 million/mi. The 36.6 mi of facility in operation have been built for a construction cost of approximately \$132 million, or \$3.6 million/mi. For the five committed transitways, approximately 80 percent of the cost is being funded using transit dollars, with the remaining cost being funded with highway monies. In addition, the highway right-of-way in the median is being made available for these transitway projects.

Daily operation and enforcement of the transitways are a Metro responsibility, which is costing approximately \$250,000/year per transitway.

Public Attitudes Regarding the Development of the Houston Transitway System

Because the transitway system being developed in Houston is somewhat unique and will involve an expenditure of approximately \$700 million, public attitudes pertaining to transitway development have been an area of continued interest. Over the years, motorists using the general-purpose freeway main lanes have been surveyed to identify their attitudes concerning these priority lane projects. Surveys have been performed both on freeways that have transitways (Katy and North) and on a freeway (Eastex) that does not currently have a transitway. A primary issue addressed in these surveys was whether the transitways were perceived by the public to be good transportation improvements.

Acceptance of the transitway as effective improvements appears to have grown over time. When asked in 1986 and 1988 if the transitways were good transportation improvements, responses from both the Katy and North corridors were generally 63 percent yes, 21 percent no, and 16 percent not sure. In a corridor (Eastex) that does not currently have a transitway, the responses were 58 percent yes, 15 percent no, and 27 percent not sure. It should be emphasized that these responses are those of the motorists using the highly congested mixed-flow freeway lanes. Although these individuals may perceive that they are receiving relatively few direct benefits from transitway development, nevertheless, in their opinion the transitways are good transportation improvements.

Transitway Use and Travel Time Savings

Total daily person-trips served by the Houston transitway system in December 1988 exceeded 40,000, a 23.5 percent increase over 1987 (Table 2). As would be expected, the transitway lanes move a relatively high percentage of peak-hour person-movement in a relatively small percentage of total vehicles (Figure 2). The single transitway lane on both the North and Katy Freeways accommodates between 35 and 45 percent of the total peak-hour, peak-direction person-volume.

TABLE 1 STATUS OF OPERATING TRANSITWAYS, DECEMBER 1988

Transitway	Date First Phase Opened	Miles in Operation	Vehicles Allowed to Use Transitway	Hours of Weekday[1] Operation
Katy (I-10)	October 1984	11.5	3+ vehicles from 6:45 to 8:15 a.m. 2+ during other operating hours	4 a.m. to 1 p.m. inbound 2 p.m. to 10 p.m. outbound
North (I-45)	November 1984[2]	9.1	Authorized buses and vanpools[3]	5:45 to 8:45 a.m. inbound 3:30 to 7:00 p.m. outbound
Northwest (US 290)	August 1988	9.5	2+ vehicles	4 a.m. to 1 p.m. inbound 2 p.m. to 10 p.m. outbound
Gulf (I-45)	May 1988	6.5	2+ vehicles	4 a.m. to 1 p.m. inbound 2 p.m. to 10 p.m. outbound
TOTAL		36.6		

[1]The transitways are presently closed on weekends.

[2]A contraflow lane was implemented on the North Freeway in August 1979. It was replaced with a barrier-separated reversible lane in November 1984.

[3]Due to construction in the corridor, only buses and vans authorized by Metro are presently allowed to use the transitway.

However, the ridership increase between 1987 and 1988 presented in Table 2 occurred because two new transitways opened during 1988. Daily use of both transitways that were operational in 1987 declined in 1988 when compared to 1987. Daily ridership per mile of transitway declined from 1,583 in 1987 to 1,101 in 1988, a decrease of 30.4 percent.

An examination of transitway operations suggests that at least three factors are helpful in explaining ridership levels on an HOV facility.

Length of Transitway Operation

Even successful HOV projects have experienced rapidly increasing ridership during the first several years of operation. Ridership data (*3*) from the North and Katy transitways in Houston, the San Bernardino Busway in Los Angeles, and the Shirley Highway in the Washington, D.C., area show that, over the first 3 years of operation, all experienced ridership increases more than 200 percent. Apparently, mode choice changes continue to occur over a period of several years. Both the North and Katy transitways have experienced this growth period. However, at the end of 1988 both the Northwest and Gulf transitways had been operational for less than 8 months.

Vehicle Groups Allowed to Use Transitways

As would be expected, allowing carpools to use a transitway or reducing carpool occupancy requirements will result in an increase in transitway person-volume (as long as the vehicular capacity of the transitway lane is not exceeded), which explains the trend in use of the North transitway. Vanpooling in general has been declining in Houston, which is reflected in the ridership trends of the North transitway. The opening of this transitway to carpools (which may occur in 1989) should increase North transitway use. A somewhat similar experience has been occurring on the Katy transitway. Before instituting the three-or-more-person (HOV-3) carpool requirement from 6:45 to 8:15 a.m. in October 1988, usage of that transitway had been increasing throughout 1988 and exceeded 19,000 daily trips in September 1988. The change in occupancy requirements, which was necessary to address a vehicular capacity problem on the transitway, caused an immediate 17 percent drop in a.m. peak-period transitway person-volumes. Since October, that usage has been increasing as daily volumes in March 1989 increased to 17,600, a 5 percent increase over the December level presented in Table 2. A more detailed discussion of the implications of the carpool occupancy increase on the Katy transitway has been given by Christiansen and Morris (*4*).

Essential Travel Time Savings

Provision of travel time savings is perhaps the most important single factor influencing transitway use. Simply, unless severe freeway congestion exists and the transitway offers meaningful time savings, usage of transitways will not be high. It has been postulated for several years that a priority HOV lane

TABLE 2 SUMMARY OF SELECTED HOUSTON TRANSITWAY OPERATIONAL DATA

Data	Katy[1]			North[2]			Northwest[3]	Gulf[3]	Total, 4 Transitways		
	12/87	12/88	% Change	12/87	12/88	% Change	12/88	12/88	12/87	12/88	% Change
Miles of transitway	11.5	11.5	0.0%	9.1	9.1	0.0%	9.5	6.5	20.6	36.6	+77.7%
Transitway Person Volume											
Daily	17897	16772	- 6.3%	14722	12946	- 12.1%	5283	5291	32619	40292	+ 23.5%
A.M. Peak Hour	4580	3881	-15.3%	3732	3732	- 5.0%	1821	1787	8508	11221	+ 31.9%
A.M. Peak Period	8703	7319	-15.9%	7238	6640	- 8.3%	3235	2754	15941	19948	+ 25.1%
P.M. Peak Hour	3812	3750	- 1.6%	3765	2725	- 27.6%	985	780	7577	8240	+ 8.8%
P.M. Peak Period	8129	8429	+ 3.7%	7484	6306	- 15.7%	1960	2469	15613	19164	+ 22.7%
Transitway Vehicle Volume											
Daily	5733	5079	-11.4%	697	531	- 23.8%	1844	1424	6430	8878	+ 38.1%
A.M. Peak Hour	1469	938	-36.1%	189	151	- 20.1%	668	490	1658	2247	+ 35.5%
A.M. Peak Period	2788	1862	-33.2%	329	265	- 19.5%	1164	719	3117	4010	+ 28.6%
P.M. Peak Hour	1180	1122	- 4.9%	157	125	- 20.4%	304	372	1337	1923	+ 43.8%
P.M. Peak Period	2517	2723	+ 8.2%	368	266	- 27.7%	636	632	2885	4257	+ 47.6%
Avg. Vehicle Occupancy, A.M. Peak Hour	3.12	4.14	+32.7%	20.8	24.7	+ 18.8%	2.73	3.65	5.13	4.99	- 2.7%
Transitway Travel Time Savings, Avg. Peak Hour (min.)[4]	8.5	13.8	+62.3%	7.9	6.2	- 21.5%	4.3	5.3	16.4	29.6	+ 80.0%
Annual Value of Travel Time Saved ($ millions)[5]	$2.8	$8.6	+207.1%	$6.8	$4.0	- 41.2%	$0.8	$1.4	$9.6[6]	$14.8[6]	+ 54.2%

Notes: Peak hour is defined as the hour in which person movement is the highest. As a result, it is not always the same hour. The peak period is a 3.5 hour time period for all transitways except the North, where it is 3 hours in the a.m. and 3.5 hours in the p.m.

[1]In October 1988, occupancy requirements to use the Katy Transitway between 6:45 and 8:15 a.m. were increased from 2+ to 3+. In 1987, the transitway operated from 5:45 a.m. to 8:00 p.m.; in 1988, it operated from 4 a.m. to 10 p.m.

[2]The North Transitway, due to ongoing construction in the corridor, is used only by authorized buses and vanpools and operates for fewer hours per day than do the other transitways.

[3]Neither the Gulf nor the Northwest Transitways were operational in 1987.

[4]Travel time data can vary significantly due to normal variations in traffic flow. Time shown is average of a.m. and p.m. peak hours on a non-incident day.

[5]Based on travel time savings per day factored to account for travel time savings resulting from incidents and a value of time of $9/hour. The value shown is the upper end of the estimated range of travel time savings.

Source: Texas Transportation Institute.

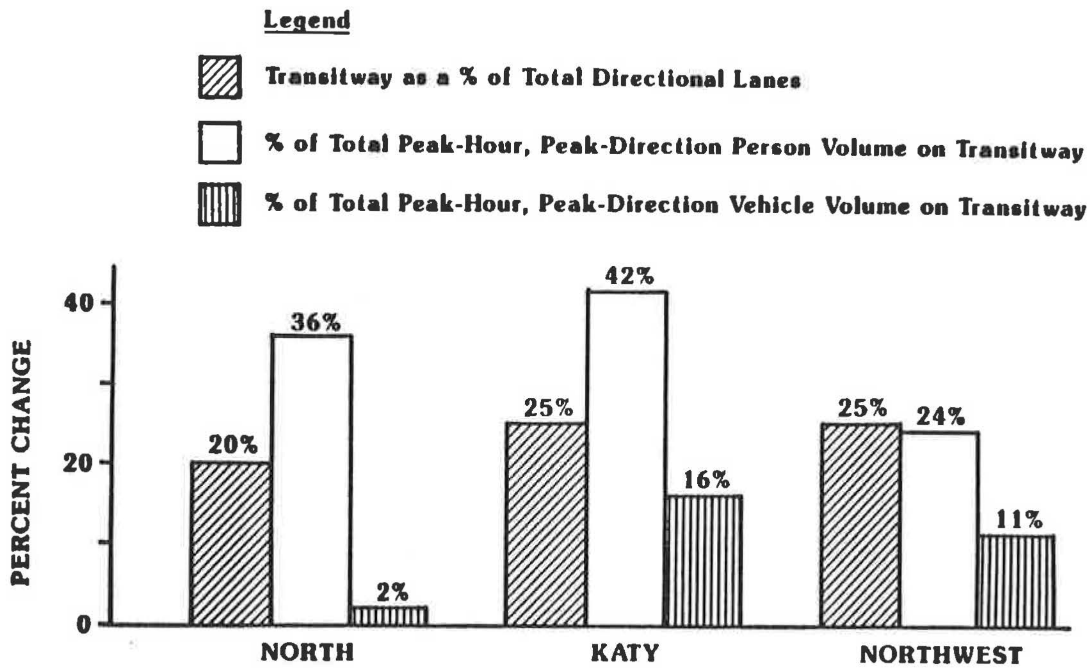

FIGURE 2 Transitway volumes as a percent of total (freeway plus transitway) a.m. peak-hour, peak-direction volumes.

must provide at least 1 min of travel time savings per mile of lane to be successful (*5*). Houston data (Figure 3), collected over several years, suggest that unless the transitway offers a travel time savings in excess of 7 to 8 min during the peak hour, use of the transitway will be marginal. This conclusion currently affects several of the Houston freeway transitways. Completion of the North Freeway main lane widening between I-610 and North Shepherd, combined with the opening of the Hardy toll road in that same corridor, has at least temporarily reduced transitway travel time savings offered by the North transitway. In 1979, when the North Freeway contraflow lane first opened, 15-min travel time savings to contraflow users were typical, but in 1988 the corresponding time savings were about 6 min. The section of the Gulf transitway currently in operation is located in a freeway segment that has recently been significantly expanded and the transitway currently offers peak-hour travel time savings of about 5 min. This marginal level of travel time savings will continue at least until the second phase of the transitway is completed. Although 9.5 mi of the Northwest transitway are operational, the geometrics and operations at the temporary terminus of this priority lane at West Little York cause severe congestion for transitway users. In fact, in the afternoon travel time savings generated on the transitway are more than negated by the congestion experienced at the terminus of the transitway. Completion of this transitway, scheduled for 1989, should eliminate this problem and result in an increase in transitway use. Until that occurs, marginal peak-hour travel time savings of about 4 to 5 min will continue to exist.

Transitway Travel Time Savings

Although transitway volumes have not been showing significant increases, the value of travel time saved by users of the transitways has increased because of the experience on the Katy Freeway. Changing the occupancy requirement to HOV-3 from 6:45 to 8:15 a.m. eliminated the delay that had been occurring on the transitway. At the same time, general freeway congestion was intensifying. Although person-volumes on the transitway declined somewhat, at least in the short run, delay incurred on the transitway declined by a much greater amount, resulting in an increase in travel time saved. The annual value of time saved by all users of the Houston transitway system in 1988 was approximately $14.8 million (Table 2). Nearly 60 percent of those savings were realized on the Katy Freeway transitway.

MEASURING THE EFFECTIVENESS OF THE TRANSITWAY SYSTEM

Before establishing criteria by which to measure the effectiveness of the transitways, the primary objectives for those transitway projects must be identified. Numerous potential objectives exist, some qualitative in nature and some that can be quantified. A 1985 survey (*6*) of HOV lane projects determined that increasing roadway capacity and reducing vehicle-miles of travel were the primary reasons for implementing HOV lanes nationwide. In Houston, the primary reason for transitway development was to increase effective roadway capacity. In the face of increasing congestion and projected freeway average daily traffic volumes in the range of 300,000 vehicles or more, travel demand simply could not be served either physically or economically just by building more additional mixed-flow freeway lanes. The transitways, with a design year volume of 7,000 to 10,000 persons/hr, could nearly double the person-movement capacity of a roadway and provide a conceptual means of serving projected travel demands. Thus, the primary objective of the Houston transitways is assumed

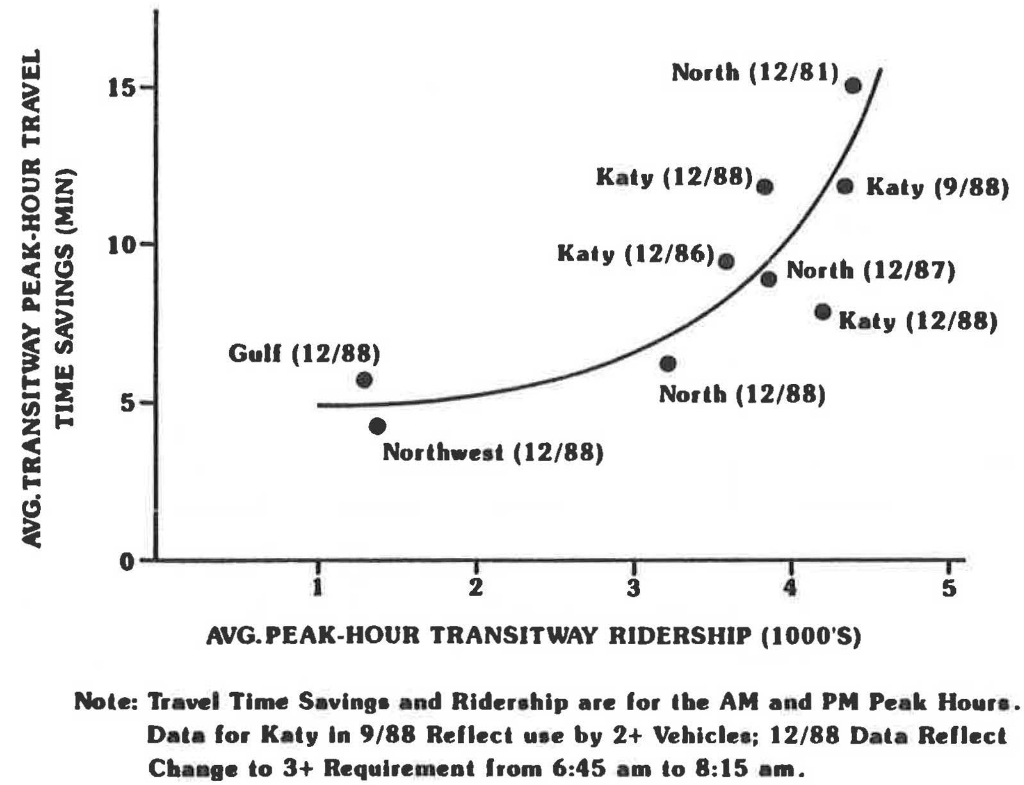

FIGURE 3 Relationship between peak-hour transitway ridership and peak-hour transitway travel time savings.

to be to increase, in a cost-effective manner, the person-movement capacity of a freeway and to do it in a manner that does not unduly impact the operation of the freeway's general-purpose, mixed-flow lanes.

A variety of positive benefits can be realized from the development of a successful transitway. Given the assumed primary objective of the transitways being developed in Houston, several potential measures of effectiveness can be quantified and used to help evaluate the performance of the transitway system.

Transitway Projects Should be Cost-Effective

Unless the transitway project is cost-effective, the project will not be able to successfully compete for the limited funds available. Many of the potential benefits associated with a transitway, such as air quality, energy, and regional economic effects, are difficult to quantify. However, one that can be quantified is the value of the time saved by those persons using the transitway. If the project has a benefit-cost ratio greater than 1 only on the basis of this single benefit, the project is cost-effective. This approach would suggest that the average annual value of time saved by users of the transitway over the life of the project should be at least 10 percent of the total transitway construction cost.

Percentage Increases in Peak-Hour, Peak-Direction Person-Volumes Resulting from Transitways Should at Least be Greater Than the Percentage Increase in Directional Lanes Added to the Roadway

In effect, this goal will be accomplished by increasing the average vehicle occupancy (persons per vehicle) on a roadway. Much of the increase in the average vehicle occupancy should be the result of creating new carpoolers and new bus transit riders. Previous research (*5*) has suggested that average occupancy should increase by about 10 percent for a project to be successful, with the percent of the total person-movement occurring in the HOV lanes used as the measure of success. Experience in Houston would suggest this threshold might be a conservative measure of success because a 10 to 15 percent increase in average peak-hour vehicle occupancy for the entire roadway might be a more appropriate indicator of whether a transitway is effective.

Transitways Should Not Unduly Affect the Operation of the Freeway Mixed-Flow Lanes but Should Increase the Per-Lane Efficiency of the Roadway

Transitways should not severely degrade safety or operations of the freeway main lanes. Also, the transitway should significantly increase (say, by more than 25 percent) the peak-hour, peak-direction efficiency per lane of the roadway facility. As defined in this discussion, peak-hour, per-lane efficiency is defined as the peak-hour person-volume, times average speed, divided by number of lanes.

Criterion 1: Transitway Projects Should Be Cost-Effective

Clearly, transitway development is not desirable unless cost-effective. Many of the potential benefits associated with a transitway facility, although possibly significant, are difficult to quantify without making numerous assumptions. Included in this benefit list are factors such as air quality, energy consumption, impact on regional economic development, impacts of improved bus schedule reliability, etc. Nevertheless, all of these can be potentially significant benefits.

However, one benefit that can be quantified relatively easily is the value of the time saved by users of the transitway facility. If the project is cost-effective solely on the basis of that criterion, it would be even more cost-effective if all the other potential benefits were considered. Also, if the transitway operational values associated with a cost-effective project can be identified, other measures of effectiveness are easier to establish.

Depending on the assumptions made concerning the discount rate and project life, different conclusions could be drawn concerning the level of travel time savings required to make the transitway project cost-effective solely on the basis of that criterion. However, as a rule of thumb, if the average annual value of the transitway user travel time savings is at least 10 percent of the construction cost of the project, the transitway project will be cost-effective (assuming a constant stream of benefits, a 20-year project life, and a 4 percent discount rate). In Houston, the conclusion is also based on the fact that the present value of the operating and enforcement costs is small compared with the capital cost.

Because congestion can generally be expected to increase in the future, the average annual value of time saved over the project life should be greater than the amount saved in the early years of the project. However, if the project appears cost-effective on the basis of today's level of use, the transitway should prove to be even more cost-effective as use increases. On the basis of the information presented in Table 2, the current annual value of time saved by users of the transitways as a percent of the capital cost of the transitway as currently operating is Katy, 27 percent; North, 14 percent; Gulf, 5 percent; and Northwest, 2 percent. The value of time being saved on the North and Katy transitways is significant when compared with the other two operating transitways, both of which have been in operation for less than 1 year.

Although the data and the analysis could be better, the procedure developed can be used as a means of estimating ridership levels needed for a transitway with a Houston-type design and associated cost to at least appear to be cost-effective (Figure 4). These facilities would need to serve more than 10,000 person-trips daily, which would roughly translate to serving in excess of 2,500 persons in the peak hour. Although the data supporting these conclusions are not definitive, this general finding is in agreement with previous research (7) pertaining to the cost-effectiveness of barrier-separated transitways, which used simulation models as a means of identifying transitway cost-effectiveness.

Criterion 2: Transitways Should Significantly Increase Roadway Person-Movement and Average Vehicle Occupancy

A primary reason for implementing transitways is to increase the person-movement capacity of the roadway during peak operating periods. Because transitways do increase the number of directional lanes, in order to be cost-effective the transitway should at least increase peak-hour person-movement by an amount greater than the increase in lanes added to the roadway caused by the transitway. If the transitway does not do this, an additional mixed-flow, general-purpose lane could be a more effective improvement. For two (Katy and North) of the three Houston transitways for which data are available, this type of increase clearly has occurred. The Katy transitway increased the number of directional lanes by 33 percent, and the a.m. peak-hour person-movement by 80 percent. The corresponding values for the North transitway are 25 and 65 percent, respectively. On the more recently opened Northwest transitway, directional lanes were increased by 33 percent and to date a.m. peak-hour person movement has increased by only 22 percent.

For the transitway to generate a disproportionately large increase in person-movement, the transitway must also increase

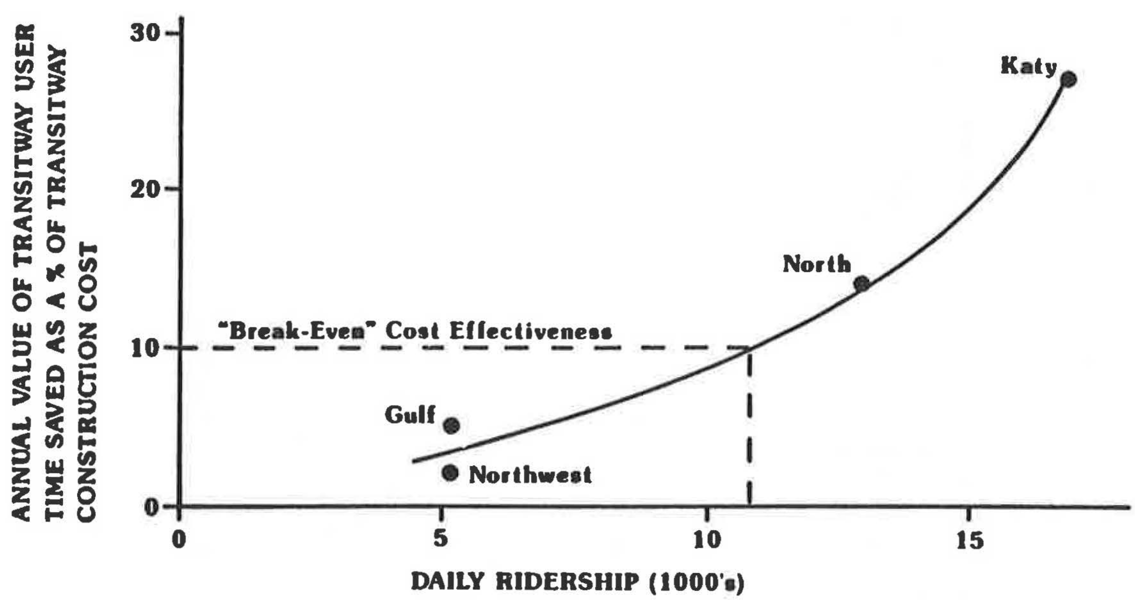

FIGURE 4 Estimated transitway ridership required for transitway to be cost-effective.

the average vehicle occupancy (persons per vehicle) characteristic of the roadway. Transitways are intended to offer a travel alternative that a significant percentage of commuters will find attractive and, therefore, will choose to either carpool or ride a bus. The result of these rideshare decisions will be that the occupancy (combined freeway and transitway volumes) for the overall roadway facility will increase.

In comparison to pretransitway conditions, a.m. peak-hour average vehicle occupancy on the North Freeway has increased by 25 percent, from 1.28 to 1.60 persons per vehicle. On the Katy Freeway, a 23 percent increase from 1.26 to 1.55 persons per vehicle has been realized. Those occupancies are unusually high for Texas freeways. To date, occupancy on the Northwest Freeway has increased by 10 percent, from 1.14 to 1.26 persons per vehicle. The fact that these increases in occupancy can be at least partially attributed to the presence of the transitway is supported by the fact that occupancy has actually declined by 9 percent on the control freeway that does not yet have a transitway (Southwest Freeway).

Carpool Component

The increase in average vehicle occupancy on a roadway should be the result of new rideshare patrons. If all the transitway accomplishes is to divert existing carpools from parallel routes to the transitway, the effectiveness of the transitway would need to be questioned.

Because carpools naturally have a fairly high turnover rate, difficulties arise from how to precisely determine how many of the carpools using a transitway are new carpools formed because of the transitway. One indicator is the previous mode of travel for the carpoolers (Figure 5). These data indicate that between 27 and 45 percent of the current carpoolers on the transitways were previously in drive-alone vehicles. The sum of drive-alone plus new trips, which is in the range of 35 to 56 percent, could be representative of new carpools.

However, because of the relatively high turnover rate of carpools, particularly for transitways that have operated for several years such as the Katy, at least some of those with a previous mode of drive-alone would have formed carpools regardless of whether a transitway were in existence. In order to try to identify this portion of the carpool component, carpoolers using the transitways were asked if they would be carpooling if there were no transitway (Table 3). On the mature Katy transitway, approximately 40 to 45 percent of the existing carpoolers previously drove alone and formed a carpool as a result of the transitway. The corresponding value for the less mature transitways appears to be in the range of 20 to 34 percent. Apparently, the transitways have been a factor in creating new carpools because the percentage of carpoolers whose previous mode was drive alone is representative of new carpools formed as a result of the transitway.

In comparing pre-transitway conditions to current conditions, the type of increase in carpooling that has been observed in freeway corridors with transitways has not occurred in the control corridor not having a transitway. Although the a.m. peak-hour volume of HOV-2 carpools (freeway plus transitway) has increased by 85 percent on the Katy and 128 percent on the Northwest Freeway, on the control freeway (Southwest) not having a transitway, the corresponding carpool volume has increased by 26 percent over the comparable time period.

Preliminary data also suggest that carpools formed in corridors with transitways may last longer than carpools in corridors without transitways. Surveys in 1986 of carpoolers using

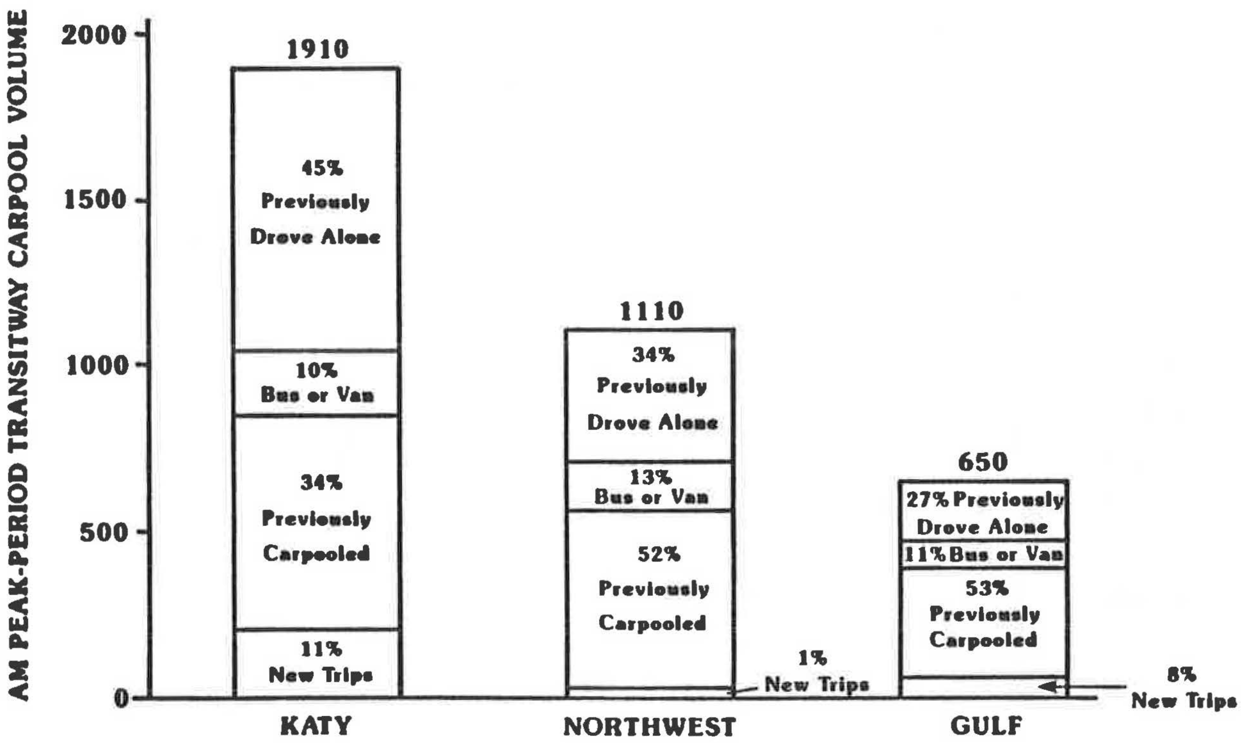

FIGURE 5 Previous mode of travel for transitway carpoolers.

TABLE 3 ESTIMATED IMPACT OF TRANSITWAYS IN FORMING NEW CARPOOLS

Transitway	Apparent % New Carpools Based on Previous Mode[1]	Would You Carpool if No Transitway			Est. % Carpools Due to Transitway[3]
		Yes	No	Not Sure	
Katy	56%	53%	35%	11%	40%-45%
Northwest	35%	70%	21%	9%	25%-34%
Gulf	35%	75%	14%	11%	20%-27%

[1]From Figure 5, the sum of "Drove Alone" plus "new trips".

[2]Transitway carpooler response to the question "If the transitway had not opened, would you be carpooling today?"

[3]It is assumed that the sum of the "no" responses plus one-half of the "not sure" responses equals the lower end of the percentage of total transitway carpools that were previously "drive alone" that formed a carpool as a result of the transitway. The upper end is the "drove alone" component from Figure 5.

the Katy transitway indicated that the average carpool on that facility had been in existence 30 months. Surveys in late 1988 on two transitways that had just opened indicated that the average carpool in those corridors had been in existence for about 20 months.

Bus Transit Component

As was the case with carpooling, available data suggest that the transitways have resulted in the creation of a large volume of new bus riders. For example, compared with pretransitway conditions, peak-hour, peak-direction bus ridership on the Katy Freeway has increased by 373 percent. Previous mode data for the North and Katy transitways suggest that fewer than 25 percent of bus riders on the transitway rode a bus before being transitway bus riders (Figure 6).

The fact that transitways generate new bus riders is further illustrated by the response to the question "If the transitway had not opened, would you be riding a bus now?" These data, presented in Table 4, suggest that approximately 50 percent of the bus riders in the Katy and North corridors are riding buses because of the existence of the transitway.

However, not all of these new riders can be attributed solely to the development of a transitway. The increased frequency of bus service being provided would, by itself, have more than doubled pretransitway bus ridership (assuming a service elasticity of 0.50). About half of the current bus riders on the Katy transitway are estimated to be new riders generated as a result of implementing the transitway. On the recently opened Northwest transitway, about all of the 23 percent increase in peak-period bus ridership can be attributed to the increase in frequency of bus service provided.

However, although a.m. peak-period bus ridership on the Katy transitway has increased by 224 percent and on the Northwest transitway has increased by 16 percent, in the control freeway corridor, not having a transitway, no change in bus ridership has occurred over the comparable time period. The same experience has occurred in observing the number of vehicles parking at bus park-and-ride lots in the corridor. Compared with pretransitway conditions, a 196 percent increase in parked cars has taken place in the Katy corridor and a 35 percent increase in the Northwest corridor. In the control corridor not having a transitway, a 1 percent increase has been observed over the comparable period of time.

Criterion 3: Transitways Should Increase the Overall Efficiency of the Roadway

Transitways can be cost-effective and can increase the person-movement capacity of a roadway. However, the transitway should not unduly affect the operation of the freeway main lanes. Transitways should also increase the overall efficiency of the roadway in which the transitway is a part. If these criteria are not realized, other potential transitway benefits such as air quality and energy impacts will not be maximized.

Impact on Freeway Main Lane Operations

Transitways, in order to be successful, must offer a significant travel time savings. As such, transitways are congestion-dependent improvements. Severe congestion must exist on the freeway for the transitway to be able to be successful by offering a significant travel time savings.

Available data suggest that the implementation of transitways with a design similar to that being used in Houston does not greatly affect the operation of the freeway main lanes, either positively or negatively. Transitways have not

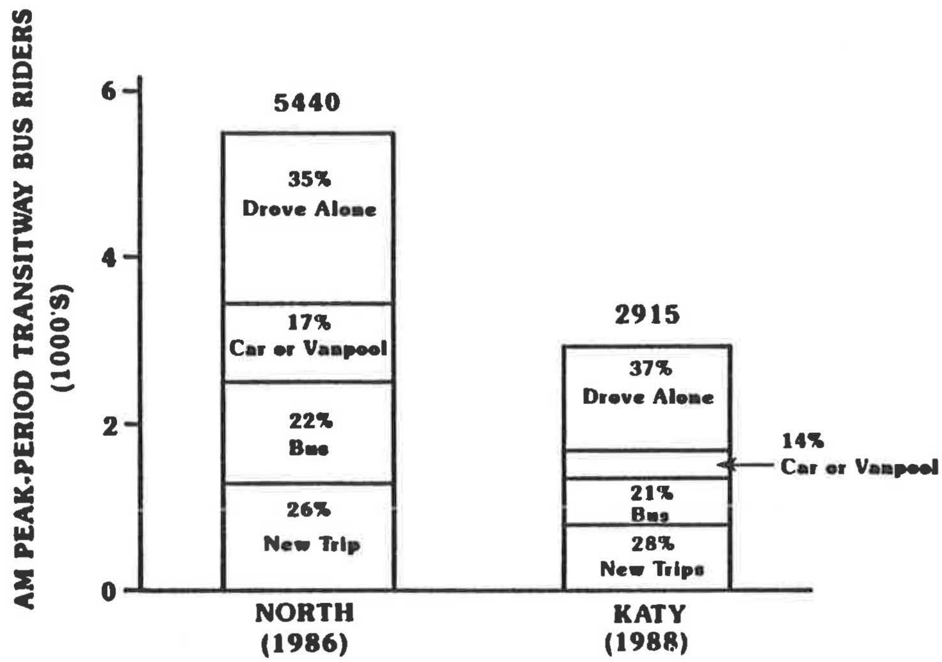

FIGURE 6 Previous mode of travel for transitway bus riders.

TABLE 4 ESTIMATED IMPACT OF TRANSITWAYS IN CREATING NEW BUS RIDERS

Transitway	Would You Be Riding a Bus if no Transitway[1]			Est. % New Bus Riders Due to Transitway[2]
	Yes	No	Not Sure	
North (1986)	23%	41%	36%	59%
Katy (1988)	36%	32%	32%	48%

[1]Transit rider response to the question "If the transitway had not opened, would you be riding a bus now?"

[2]It is assumed that the sum of the "no" response plus one-half of the "not sure" responses equals the percentage of existing transitway bus riders who would be riding the bus (i.e., new riders) if there were no transitway.

greatly altered demand for the freeway main lanes because during peak periods, in comparison to pretransitway conditions, the vehicular volume per freeway main lane is essentially unchanged or has increased slightly. Although speeds on some freeways have actually increased since transitway implementation, this increase is largely attributable to factors other than the transitway. In addition, compared with pretransitway conditions, accident rates for the freeways with transitways have generally declined slightly. For example, for the control freeway (Southwest Freeway) without a transitway, accident rates have remained essentially unchanged for the comparable time periods.

Impact on Overall Roadway Efficiency

Transitways are intended to move substantial volumes of commuters at relatively fast speeds. As such, successful transitways should improve the overall efficiency of a freeway. For purposes of this discussion, peak-hour efficiency of the freeway is expressed as the product of the peak-hour person-volume times the speed at which that volume is moved. Peak-hour efficiency is expressed on a per-lane basis. In all cases for which data are available, implementation of the transitway increased the overall efficiency of the facility (Table 5). These increases in efficiency have been larger than those experienced on a freeway that does not have a transitway.

This criterion has weaknesses in that it does not directly address what would have happened to overall roadway efficiency had the new lane been used as another mixed-flow lane rather than a transitway. However, the North Freeway where, in addition to the transitway, an additional mixed-flow lane has been added provides some measure of this impact. About half of the overall increase in roadway efficiency has occurred in the main lanes (Table 5). Virtually all of the increase in the Northwest Freeway is caused by improvements in main

TABLE 5 ESTIMATED CHANGE IN PEAK-HOUR PER-LANE EFFICIENCY, BEFORE AND AFTER TRANSITWAY IMPLEMENTATION

Freeway	Pre-Transitway Freeway Efficiency	Current Freeway Efficiency	Current Combined Freeway and Transitway Efficiency	Percent Change
North	41	65	89	+117%
Katy	38	44	77	+103%
Northwest	62	88	90	+ 45%

[1]Peak-hour per lane efficiency defined on the person volume per lane times the average speed divided by 1000. Thus, it is a measure both of the volume moved and the speed at which that volume is moved.

lane operation, in line with the 35 percent increase experienced on the control freeway. For a transitway to be effective, the transitway alone should increase the efficiency of the roadway by at least about 20 percent. In both the North and Katy corridors, a meaningful increase in per-lane efficiency has occurred that can be attributed to the transitway. This result has not occurred to date in the Northwest corridor.

CONCLUSION

A 95.5-mi system of freeway transitways is being developed in Houston with 36.6 mi operating in four different freeway corridors today. Development of the system appears to have public support.

The principal objective of the Houston transitways was assumed to be to cost-effectively increase the person-movement capacity of the freeways and to do this in a manner that does not unduly affect the operation of the freeway main lanes. With this assumed objective, several performance measures have been developed.

In assessing the performance of the transitway in meeting its objectives, the following quantitative values can be used as guides.

Objective: Transitways Should Be Cost-Effective

Potential performance measure—

- Conservatively, the project will have a benefit-cost ratio >1 if the average annual value of the time saved by users of the transitway over the life of the project exceeds 10 percent of the initial construction cost of the transitway.

Objective: Increase Roadway Person-Movement

Potential performance measures—

- Daily transitway ridership should be in excess of 10,000;
- The transitway should increase peak-hour, peak-direction person-movement by an amount greater than the increase in directional lanes added to the roadway due to transitway implementation; and
- The transitway should increase the a.m. peak-hour, peak-direction average vehicle occupancy (persons-per-vehicle) for the roadway by at least 10 to 15 percent.
  —More than 25 percent of the carpools using the transitway should be new carpools created because of the transitway, and
  —More than 25 percent of the bus riders using the transitway should be new bus riders created because of the transitway.

Objective: Don't Unduly Impact Freeway Main Lane Operations

Potential performance measures—

- A statistically significant increase in either freeway congestion or freeway accident rate should not result solely from transitway implementation.
- Absolute value of the total roadway per-lane efficiency should increase by at least a factor of 20 because of implementation of the transitway (total roadway efficiency should be at least 20 times greater than freeway main lane efficiency). Efficiency is the product of person-volume times speed.

Performance measures suggest that the Katy transitway is clearly fulfilling its intended objective. Although the North transitway also appears to be effective, allowing carpools onto this facility will increase its attractiveness and should, on the basis of current carpool demand estimates, make it comparable to the Katy transitway in terms of performance. As presently operated, neither the Gulf nor the Northwest transitway can be considered to be effective. However, only the first phase of these projects is presently operating and future extensions will significantly increase potential transitway travel time savings and, thus, enhance the attractiveness of the facilities. Also, these facilities have not operated for a long period of time (less than a year) and some growth in transitway use can be expected to take place over time.

Continued monitoring of all the committed transitways will take place as part of ongoing research projects.

ACKNOWLEDGMENTS

Since 1974, the Texas State Department of Highways and Public Transportation has sponsored an ongoing research effort pertaining to priority treatment for high-occupancy vehicles. In more recent years, the Harris County Metropolitan Transit Authority has also been actively involved in this research program. The oversight and funding provided by the sponsoring agencies are gratefully acknowledged.

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Role of High-Occupancy-Vehicle Lanes in Highway Construction Management

ALLAN E. PINT, CHARLEEN A. ZIMMER, AND FRANCIS E. LOETTERLE

The Minnesota Department of Transportation (Mn/DOT) is constructing I-394 along the portion of US-12 that extends from downtown Minneapolis to the suburb of Wayzata. When completed, I-394 will have high-occupancy-vehicle (HOV) lanes. Mn/DOT built a temporary HOV lane along US-12 before constructing I-394 to introduce the HOV lane concept to commuters and to improve capacity during construction. Mn/DOT and the FHWA have been conducting an evaluation of this temporary HOV lane. Phase I evaluated operation in an arterial highway environment before construction. Phase II evaluated operation and use of the HOV lane during highway construction. Five key issues were addressed in the Phase II evaluation: (a) what can be learned about the design and operation of HOV lanes, (b) who uses HOV lanes and what factors cause people to choose carpooling or the bus over driving alone, (c) how has construction affected use of the HOV lane, (d) what was the role of the HOV lane in construction traffic management, and (e) how has the HOV lane affected highway construction? Key findings are summarized regarding these questions and advantageous circumstances under which the use of HOV lanes during construction are identified.

In 1985, the Minnesota Department of Transportation (Mn/DOT) began construction of I-394 in Minneapolis, Minnesota. As part of this project, Mn/DOT constructed a temporary high-occupancy-vehicle (HOV) lane on US-12 to link the western suburbs of the Twin Cities to downtown Minneapolis (see Figure 1). The interim HOV lane was built for the purposes of introducing the HOV lane concept to commuters before construction of permanent HOV lanes and providing added capacity during construction.

Mn/DOT and FHWA have funded an ongoing evaluation of the I-394 HOV lane to track its progress before, during, and after construction. Phase I reported on the first year of operation in a before-construction condition. The current phase, Phase II, evaluates the operation of the interim HOV lane during construction but before major segments of the highway have been completed. The final phase of the evaluation will focus on the operation of the HOV lane after completion of major segments of I-394.

Five primary questions are being asked in the current phase of the evaluation:

1. What can be learned about the design and operation of HOV lanes on arterial highways and during construction?
2. Who uses the HOV lane and what factors caused people to choose carpooling or the bus over driving alone?
3. How has construction affected use of the HOV lane?
4. What was the role of the HOV lane in traffic management during construction?
5. How has the HOV lane affected the highway construction project?

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FUTURE I-394 TRANSPORTATION SYSTEM

When completed, I-394 will have two mixed traffic lanes in each direction and two lanes for high-occupancy vehicles (3 mi of separated reversible lanes and 8 mi of concurrent flow diamond lanes). I-394 is being built along the alignment of existing US-12, from downtown Minneapolis to the third-ring suburban municipality of Wayzata, 11 mi to the west. US-12 has a 3-mi freeway section on the east end (two lanes in each direction plus auxiliary lanes) and an 8-mi signalized suburban arterial section (two lanes in each direction) on the western end.

Mn/DOT and FHWA are working together to provide more than concrete and bridges on I-394. Programs and facilities are being provided to integrate regular route transit and carpooling into the highway facility and to encourage increased use of these forms of transit. The intent of the I-394 transportation system, as this combination of facilities and programs has come to be known, is to maximize the number of people carried by encouraging carpooling and bus ridership. Key design features are shown in Figure 2 and include two bus transfer stations, seven park-and-ride lots, ramp metering with HOV bypass lanes, three parking garages in downtown Minneapolis with preferential carpool parking, skyway connections between the garages and downtown Minneapolis, and a sophisticated traffic management and surveillance system. These facilities will be supported by expanded timed-transfer bus service, carpool matching services, aggressive HOV enforcement, and an extensive public information program.

The estimated total cost of construction is $420 million. Construction began in 1985 and is scheduled for completion in 1993. Interstate completion funds provided 90 percent of the funding for the project with state funds used for the matching 10 percent.

INTERIM HOV LANE

The I-394 interim HOV lane combines concurrent flow diamond lanes and a single reversible lane. Diamond lanes are lanes that are marked with a diamond symbol and reserved for HOVs, but are not physically separated from the regular traffic lanes. The single reversible lane is physically separated

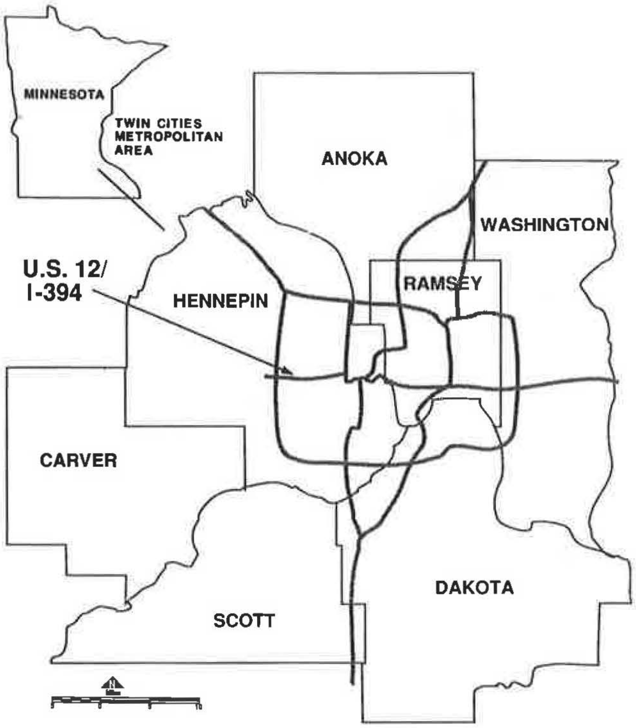

FIGURE 1 Regional location of I-394.

from the regular traffic lanes by a jersey barrier and is located in the median of US-12. During construction, the initial design of the interim HOV lane has been and will continue to be modified as different portions of the highway undergo reconstruction.

WHO USES THE HOV LANE?

Data have been collected on the I-394 HOV lane since it opened in November 1985. Volume, occupancy, bus ridership, and travel time data were collected in April 1989. Questionnaires were also distributed to people using the regular lanes, using the HOV lane, and riding on buses that used the HOV lane. Survey response rates were 37 percent for regular lane drivers, 29 percent for carpoolers in the HOV lane, and 50 percent for bus riders. The following information was derived from this survey.

Demographics

The typical carpooler on the I-394 interim HOV lane is 31 to 45 years old, lives in a 2- to 4-person household, owns two automobiles, and has a household income of over $50,000. These characteristics are also typical of the regular lane driver. However, the typical bus rider is younger, has fewer family

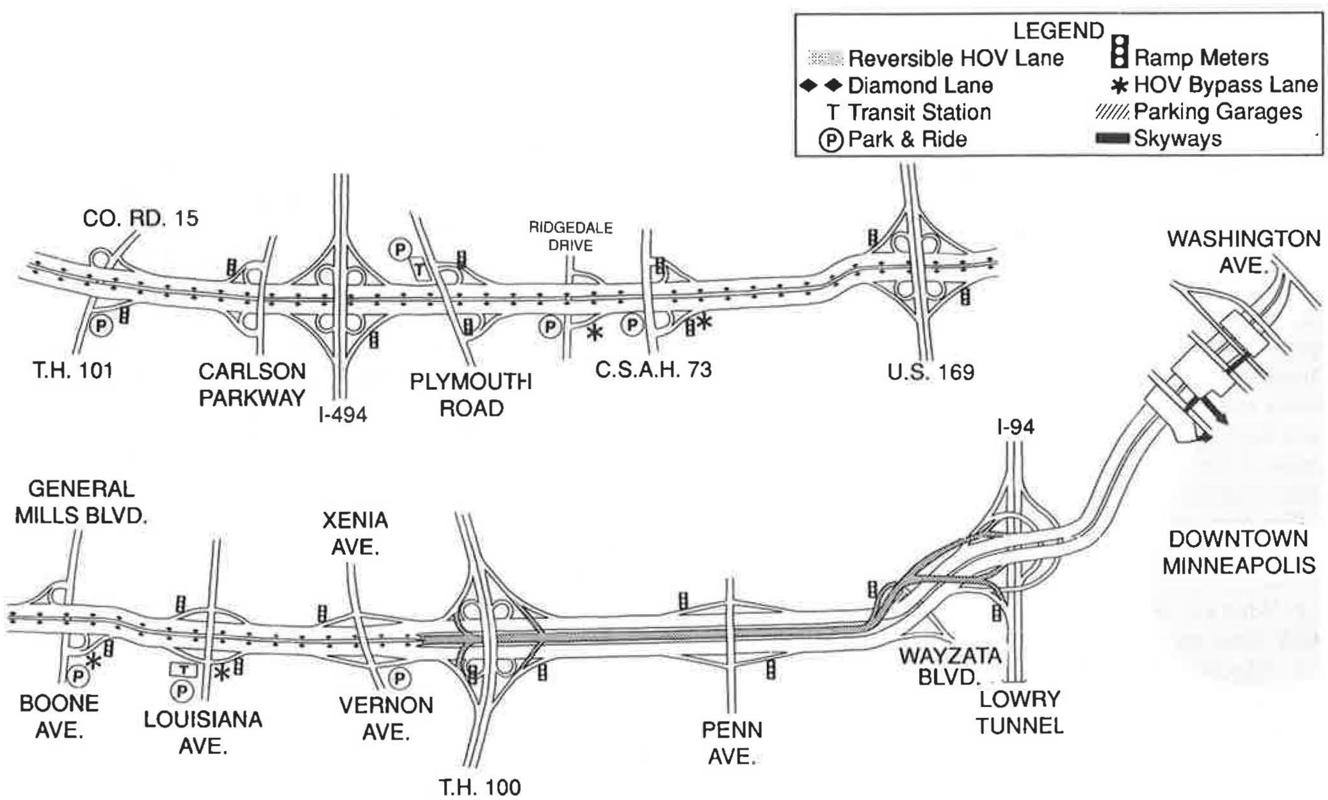

FIGURE 2 I-394 transportation system.

members, owns fewer automobiles, and has a lower household income.

Trip Purpose

The majority of respondents (86 percent of regular lane drivers, 83 percent of HOV lane respondents, and 98 percent of bus riders) were traveling to or from work, which is consistent with findings before construction activity.

Trip Frequency

Approximately 62 percent of carpoolers use the HOV lane 5 days per week. Eleven percent use the HOV lane four days per week, 11 percent three days, 8 percent two days, and 9 percent one day a week. Carpoolers in the morning tend to use the HOV lane more regularly than carpoolers in the afternoon. These patterns are similar to those found before construction activity.

Trip Destination

The survey results show a strong correlation between mode and downtown destination. Ninety-one percent of bus riders, 71 percent of carpoolers in the HOV lane, and 29 percent of regular lane drivers who use US-12 eastbound in the morning were going to downtown Minneapolis.

DESIGN AND OPERATION OF HOV LANES ON ARTERIAL HIGHWAYS AND DURING CONSTRUCTION

The I-394 temporary HOV lane is unusual in two respects. First, this lane uses a reversible HOV lane on a signalized arterial highway. Second, both concurrent-flow diamond lanes and reversible HOV lanes are used during construction. One of the purposes of the I-394 case study was to evaluate the design and operational characteristics of the HOV lane. Observations were made regarding

- Occupancy requirements,
- Hours of operation,
- Use during special events,
- Considerations in combining diamond lanes and reversible lanes,
- Left exits and entrances,
- Intersection operations,
- Problems with lane gates, and
- Snow removal.

Occupancy Requirements

One of the principal research concerns of the I-394 case study was the evaluation of a carpool occupancy requirement of only two people. This occupancy requirement was established because there were so few carpools with three or more people when the HOV lane was first opened. Surveys in 1986 and 1989 indicate that 80 percent of carpools in the HOV lane have two people, 14 percent have three, and 6 percent have four or more people. These proportions have remained consistent before and during construction. In the April 1989 survey, 76 percent of carpoolers said they would not have started carpooling in the HOV lane if the occupancy requirement had been three or more.

Hours of Operation

The HOV lane is open to traffic eastbound from 6 to 10 a.m. in the morning and westbound from 2 to 7 p.m. in the afternoon with hours sometimes shortened because of construction activities. Mn/DOT research indicates that it may be cost-effective to operate the lane for longer hours in the afternoon but not in the morning. The HOV lane is opened and closed manually by Mn/DOT personnel. An active reminder system and a back-up plan have been necessary to ensure that the lane opens on time consistently.

Use During Special Events

Since late 1986, the HOV lane has been opened during special events, particularly for Minnesota Vikings and Minnesota Twins games. An April 1989 survey of regular lane drivers on US-12 indicated that 74 percent of people driving alone in the mixed traffic lanes have used the HOV lane at least during a special event.

Combination of Reversible HOV Lanes and Diamond Lanes

Use of both a single reversible HOV lane and two concurrent-flow diamond lanes has provided much needed flexibility during construction. However, care must be taken to provide adequate signing and transition areas to limit driver confusion and maintain safety. An early concern was that short diamond lane segments might result in people weaving in and out of the mixed traffic lanes to jump queues. Although it occurs, lane switching is much less common than initially anticipated.

Left Entrances and Exits

Entrances to and from the reversible HOV lane are from the left lane. The I-394 experience indicates that such a design can work safely if an adequate merging section can be provided. However, locations exist where HOVs experience delays in entering or exiting the HOV lane because of congestion in the regular lanes.

Intersection Operations

The HOV lane runs through several signalized intersections. Because of operational and safety concerns, no turns are permitted to or from the HOV lane at these intersections. Illegal turning at the intersections was a problem initially but additional signing, good enforcement, and time have significantly reduced this problem. Signal timing, which is critical for ensuring maximum time savings for HOVs, is set to facilitate the progression of drivers in the HOV lane.

Gates

Entrance gates have been an ongoing maintenance problem because of weight, wind damage, and poor night visibility. The majority of accidents related to the HOV lane involved vehicles hitting the gates at night.

Snow Removal

No special procedures have been needed to provide adequate snow removal. Maintenance activities are managed as part of Mn/DOT's standard highway maintenance program.

WHAT INCENTIVES AFFECTED MODE CHOICE?

In the April 1989 surveys, people using HOV lanes were asked what the greatest benefits of the HOV lane were. Time savings were the most important benefit to carpoolers, whereas cost savings were the most important benefit to bus riders.

People using the regular traffic lanes were asked what incentives would encourage them to use the HOV lane. Incentives that appealed most to people driving alone were time savings (18 percent), help finding a partner (10 percent), operating cost savings (5 percent), and parking cost savings (4 percent). Forty-eight percent of regular lane drivers indicated they could be encouraged to use the HOV lane, whereas 52 percent indicated that nothing would encourage them to use the HOV lane.

Time Savings

Carpoolers note time savings as the most significant benefit of the HOV lane and perceive an average time savings of 10 min per trip. This finding is consistent with results of surveys prior to construction on US-12. Measured time savings in the morning peak hour are still between 8 and 10 min but less than 5 min during other hours. Both perceived and measured time savings are less in the afternoon than in the morning. Carpoolers also often note that the HOV lane is more reliable than the regular traffic lanes. Most bus riders perceive no time savings.

Parking Cost Savings

In the April 1989 surveys, 19 percent of carpoolers and 63 percent of bus riders say they save money on parking. This percentage may have changed significantly since the first I-394 parking garage was opened in August 1989. Carpoolers from I-394 who park in these garages pay $10 per month compared to the regular rate of $80 per month.

Operating Cost Savings

Thirty-six percent of carpoolers and 56 percent of bus riders say they save money on fuel. Twenty-one percent of carpoolers and 35 percent of bus riders say they save money on vehicle operating costs.

Work Hours

Fifty-three percent of carpoolers and 63 percent of bus riders reported fixed working schedules compared with only 34 percent of regular lane drivers. Regular lane drivers most frequently cite job-related reasons such as job schedule or need car for work as reasons they do not use the HOV lane. However, 19 percent said they had no one to carpool with and 6 percent said they had an irregular carpool partner.

HOW HAS CONSTRUCTION AFFECTED USE OF THE HOV LANE?

One of the primary purposes of the I-394 HOV lane was to provide additional traffic capacity during construction, which was easily accomplished initially because an additional lane was added to an already congested facility. However, highway construction typically causes considerable traffic diversion, which affects the travel times on the interim HOV lane as well as regular traffic lanes. The I-394 Case Study—Phase II is comparing the use of the HOV lane before and during construction to determine the impacts of construction on HOV use.

Changes in HOV Lane Volumes

The highest volumes in the HOV lane were reached 1 year after the HOV lane opened, just a few months before mainline construction started on US-12 (see Table 1). Overall, volumes in the HOV lane have decreased since construction began but

TABLE 1 HISTORICAL TRAFFIC VOLUMES ON US-12 WEST OF TURNER'S CROSSROAD

	AM Peak Hour		AM Peak Period	
	HOV	Regular	HOV	Regular
May 1984	--	1,890	--	4,940
November 1985	410	1,750	740	4,660
May 1986	495	1,610	860	4,570
November 1986	560	1,650	960	4,840
May 1987	480	1,900	790	4,950
November 1987	490	1,840	790	4,700
May 1988	470	1,990	770	5,150
November 1988	480	1,650	780	5,010
May 1989	420	1,940	670	4,830
November 1989	470	1,940	780	5,060

are still higher than they were when the HOV lane first opened. Volumes in the regular lanes also decreased when construction began but have increased to levels that equal or exceed pre-HOV lane volumes. Typically, volumes both in the HOV lane and in the regular traffic lanes drop in April when construction starts but then gradually rebound to previous volumes by November when construction ends. In November 1989, 1,550 people in 470 vehicles were using the HOV lane during the a.m. peak hour, compared with approximately 1,300 people in 950 vehicles in each regular traffic lane.

Changes in Carpool Volumes

Since 1984, before construction began, eastbound a.m. peak-hour traffic has increased by 9 percent. Carpooling in both the HOV lane and the regular traffic lanes has increased by 117 percent during the same time period.

Changes in Automobile Occupancy

Automobile occupancy during the a.m. peak hour increased from 1.17 to 1.25 persons when the HOV lane opened and continued to increase to 1.29 during the first year of operation (see Figure 3). Since construction began, automobile occupancy during the a.m. peak hour has declined slightly to 1.28 persons. Automobile occupancy on similar highways in the Twin Cities metropolitan region has been declining and was about 1.12 persons per vehicle in the peak hours in 1989.

Changes in Bus Ridership

Bus ridership has remained fairly stable since 1984. Shortly after the HOV lane opened in November 1985, an express route was added to the existing local bus service on US-12. Total weekday ridership of these two routes in October 1989 was about 2,400. Over the 5-year period, this ridership level has fluctuated between 2,300 and 2,900. This variation seems to be more directly related to the changing seasons than to any long-term trend, although ridership on the local route, which does not use the HOV lane, has been affected by the elimination of some roadside transit stops because of construction.

Approximately half of the bus riders on US-12 drive alone to a park-and-ride lot to ride the bus. Another 10 percent are dropped off at a bus stop. Thirty-nine percent walked or bicycled and 4 percent rode another bus. Over half of the bus respondents (56 percent) have ridden the bus more than 2 years, whereas another 15 percent have ridden the bus for at least 1 year. Eighty-seven percent plan to continue riding the bus after I-394 is completed. Twenty-five percent of bus riders receive some assistance from their employer to pay transit fares.

Changes in Prior Mode

Twenty-eight percent of carpoolers in the HOV lane drove alone on US-12 before they started using the HOV lane. Another 11 percent drove alone on other routes. One year after opening of the HOV lane, 26 percent said they drove alone on US-12 previously and 12 percent said they drove alone on other routes. The largest change in previous mode was a dramatic decrease in the number of people who previously carpooled on other routes (see Figure 4). The most significant impact of construction on HOV lane use was the rediversion of carpoolers from other routes back to their previous routes. This diversion may have been a direct result of the elimination of access between the HOV lane and T.H. 169 because of construction bypasses.

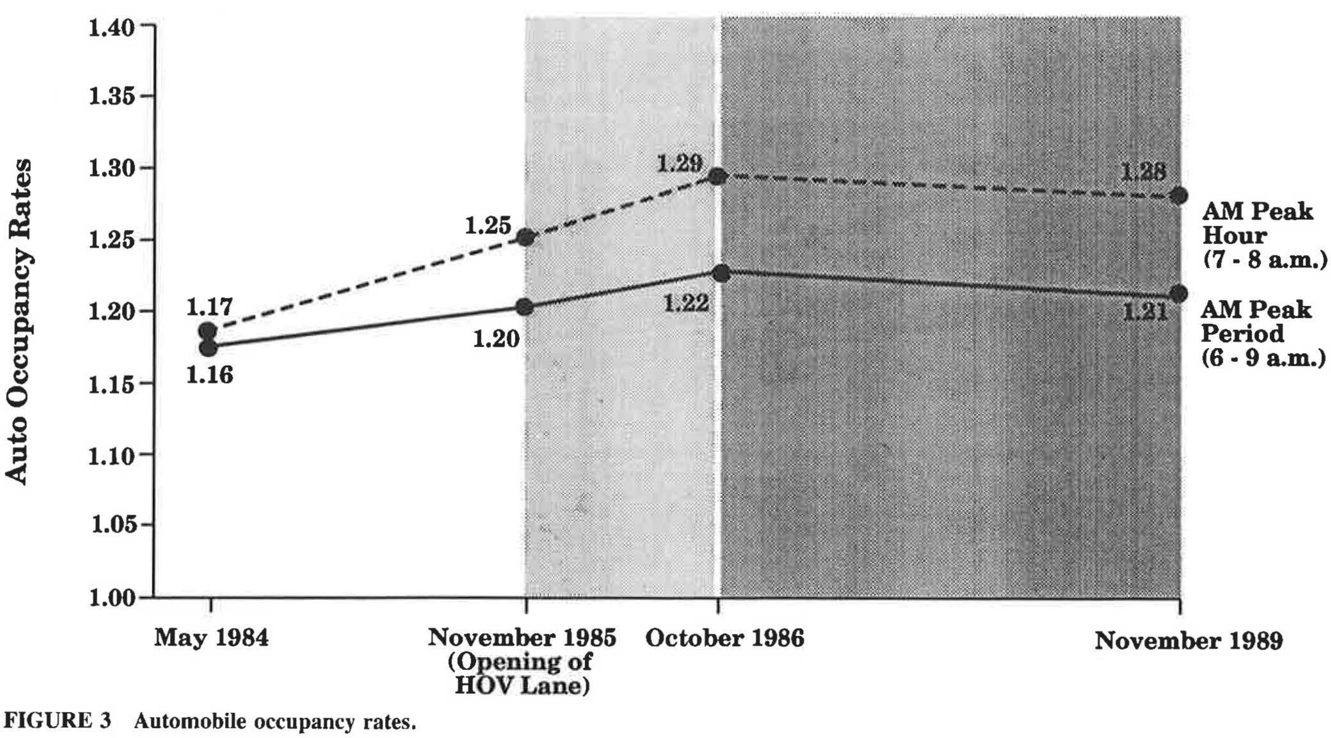

FIGURE 3 Automobile occupancy rates.

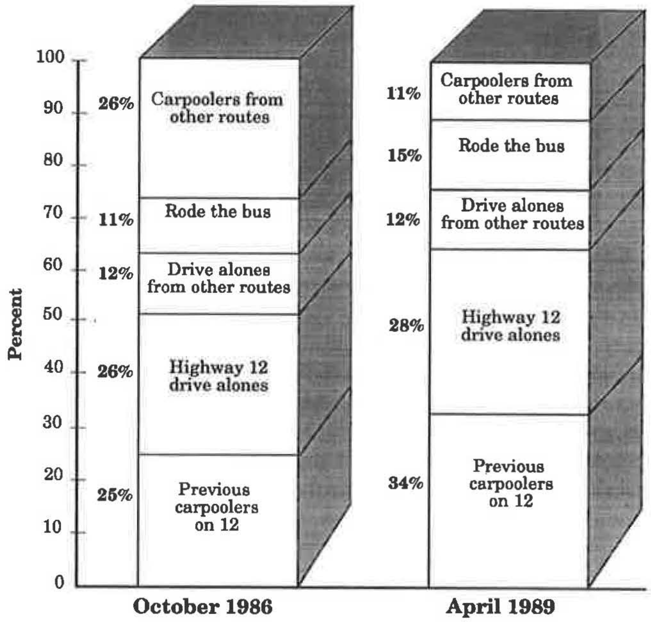

FIGURE 4 Previous travel mode of a.m. peak-hour I-394 HOV lane users.

Traffic Diversion

Twelve percent of regular lane drivers said they avoided US-12 during construction in 1988 and 8 percent said they would avoid the route during the 1989 construction season. By comparison, 20 percent of carpoolers said they avoided US-12 during construction in 1988 and 17 percent said they would do the same during the 1989 construction season. Of the bus riders, 87 percent rode the bus during the 1988 construction season, whereas 95 percent of bus riders planned to continue riding the bus during the 1989 construction season.

Satisfaction

Fifty-two percent of HOV lane carpoolers are very satisfied with the facility and 44 percent are somewhat satisfied. Fifty-two percent of bus riders are very satisfied with bus service and 45 percent are somewhat satisfied.

ROLE OF HOV LANE IN TRAFFIC MANAGEMENT DURING CONSTRUCTION

The interim HOV lane is an integral part of an overall strategy for effective management of traffic during construction. Most important, the HOV lane provided added traffic capacity during the construction period along a corridor with high traffic congestion. Key strategies for managing traffic during the I-394 construction project included

- Maintenance of people-carrying capacity,
- Encouragement of carpooling,
- Maintenance of access,
- Signing, and
- Public information.

Maintenance of People-Carrying Capacity

Two regular lanes are kept open in each direction at least during peak hours, which had to be accomplished without taking more right-of-way than what is required for the completed roadway. Contractors cannot restrict lanes between 6 and 9 a.m. or between 3 and 6 p.m. Contractors may be fined if this condition is not met. Signal timing is coordinated where possible and maximum green times are allocated to US-12 traffic.

Encouragement of Carpooling

Because two regular lanes have been kept open, the HOV lane is a bonus lane for commuters. The hours of operation for the HOV lane have been maintained during the construction periods.

Since 1985, Mn/DOT has also maintained surface parking lots near downtown Minneapolis for free carpool parking. The lots have changed location and size as a result of construction activities, but have been actively used on a regular basis. In August 1989, Mn/DOT opened the 5th Street garage with 1,600 spaces with priority access and reduced-rate contracts for I-394 carpoolers. This facility is the first of three parking garages with carpool preference being built as part of the I-394 transportation system. In the first week of December 1989, there were 456 I-394 HOV parkers with monthly contracts in the 5th Street garage out of a total of 1,245 parkers per day.

Existing park-and-ride lots and major bus stops have been maintained along the corridor. However, the size and location of park-and-ride lots have varied and many local bus stops have been eliminated on US-12. Mn/DOT has worked closely with the Metropolitan Transit Commission (MTC) to coordinate bus service and maintain service to existing transit passengers to the greatest extent possible.

Maintenance of Access

An important goal of traffic management on the I-394 project has been to maintain reasonable access to all businesses and residential areas. Mn/DOT has worked with local municipalities to develop a business area signing program for directional signing from US-12. The signs are made and installed by the city with businesses bearing the cost. Mn/DOT has also allowed special signing for individual businesses in unique situations where access is significantly changed. This program has been effective in satisfying business concerns. Many illegal signs are also present on the project but are allowed to remain if they do not block construction signage or cause a safety problem.

Special attention is paid to access to Ridgedale Regional Shopping Center, especially during the busy holiday season. The Ridgedale Drive interchange was reconstructed in 1989, removing a key access route to the Ridgedale Regional Shopping Center. During ramp construction, the HOV lane between Ridgedale Drive and Plymouth Road operated only eastbound from 6 to 9 a.m. weekdays. Part of the HOV lane served as a westbound ramp to Ridgedale at all other hours.

This change was permitted temporarily because travel time savings for westbound HOVs were nonexistent because of construction bypass design in the westbound direction.

Changeable Message Signs

Changeable message signs are used extensively to communicate traffic changes to the motorist. Through the I-394 contract, Mn/DOT has acquired four of these signs, which have been effective in communicating traffic changes to motorists.

Public Information

Extensive and early public information has been essential in minimizing the complaints received in the field. Public meetings are held in early spring and late fall at each major intersection to provide information to businesses and residents about seasonal construction activities. These meetings have been coordinated through the local chamber of commerce. Day-to-day changes are communicated with a hand-delivered construction bulletin. A semiannual newsletter is mailed to all households and businesses in the corridor. Mn/DOT also works closely with the media to provide information on construction bypasses and delays through press releases, interviews, and announcements by the corridor manager. Information on carpooling and bus services, including information on the HOV lane, is provided at all meetings and in all printed materials.

Newspaper articles, highway signs, and the newsletter are the most frequently cited sources of information on I-394 for bus riders, carpoolers, and regular lane drivers. Other sources of information include billboards, newspaper advertisements, brochures, and the local transit provider. Recognition of the newsletter as a source of information has increased steadily since its first publication. Bus riders, carpoolers, and regular lane drivers all want more timely information on construction activities. Carpoolers also expressed a strong interest in more information on carpool parking.

HOV LANE EFFECTS ON CONSTRUCTION

In general, the highway segments that include the interim HOV lane are the most difficult to construct. Difficulties arise because, while a six-lane freeway and its associated frontage roads are being constructed, five or six lanes of traffic are being maintained within the right-of-way of the final freeway and frontage road system. In many cases, maintaining this level of access requires the construction of temporary roadways that add to the cost of construction and the time required to complete a segment. For example, the construction project near the US-169 interchange and the General Mills Boulevard interchange included 1.5 mi of roadway and two interchanges at a cost of $45 million. Twenty percent of this cost was attributed to activities and temporary construction related to maintaining traffic.

The greatest design challenge of the interim HOV lane has been to incorporate the lane into the construction bypass and staging plans for the project. Both the design and the location of the HOV lane have been required to change periodically during construction in order to meet this challenge. Figure 5 shows the overall construction staging of the project and the associated changes in the HOV lane. The primary objectives of these plans have been to ensure that the HOV lane remains open continuously, is safe to use, provides head-of-the-line preference to HOVs at congestion points, and can be tied into permanent HOV sections as they are completed.

Recognizing the impacts of the HOV lane on construction costs and schedules, Mn/DOT has aggressively used a number of contract management tools to ensure timely completion of project stages. These tools include the coordination of construction staging and related transit projects, contract fast-tracking, contract incentive and disincentive clauses, field modifications, and management by a corridor manager.

Construction Staging

The I-394 project is divided into eight major construction segments. Each of these projects was advanced through the design process separately and let to construction when ready. Initially, construction staging was designed to be compatible between adjoining segments; however, staging generally changes during construction. As a result, the plans are modified in the field after consultation with the designers and traffic engineers.

Coordination With Construction of Transit Facilities

Mn/DOT is also constructing two transit stations and five park-and-ride lots as part of the I-394 project. These projects are staged to open at the same time the interchange serving the facility is opened. Although separate site amenity contracts including buildings, signing, and landscaping will be awarded, construction will be coordinated with the site development work.

Contract Fast-Tracking

Fast-tracking is used extensively to keep contractors on a tight time schedule. Mn/DOT engineers determine completion dates of the various stages. Contractors are expected to meet these completion dates even if it requires additional personnel and equipment or extended working hours including overtime and weekend work. Contractors are also required to stage work so that bridges and walls can be built during the winter months. Bar charts and other progress schedule requirements are included in all contracts to monitor contractor performance and schedule.

Incentive and Disincentive Clauses

All contracts include liquidated damages if work is completed late. Damages are generally $5,000 per day for work beyond the completion dates. All Mn/DOT contracts also include a value engineering clause. Although large incentive and disincentive clauses have only been used in isolated cases, the

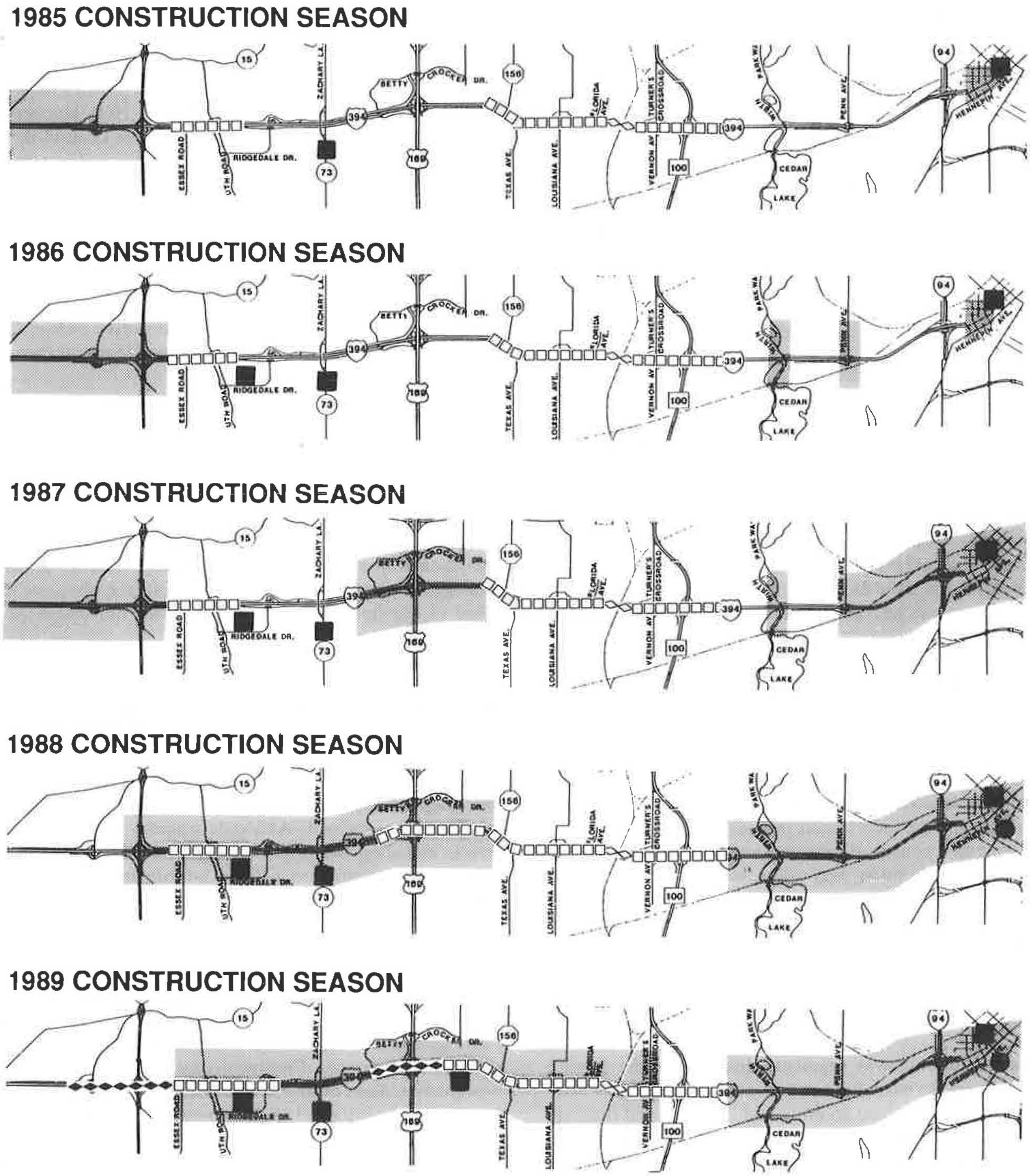

FIGURE 5 I-394 construction staging schedule. (continued on next page)

last major construction contract on the I-394 project will include a bonus of up to $1,000,000 for early completion. The bonus and penalty clause is $5,000 per day for either early completion or late completion. Early completion of this segment, which includes an interchange between T.H. 100 and I-394, is desirable because the segment has the highest volume interchange along the project.

Field Modifications

Because the I-394 project was fast-tracked through design as well as construction, field modifications and changes are often required. In order to facilitate these changes expeditiously, a member of the design team is assigned to the construction administration team, which has worked well to resolve prob-

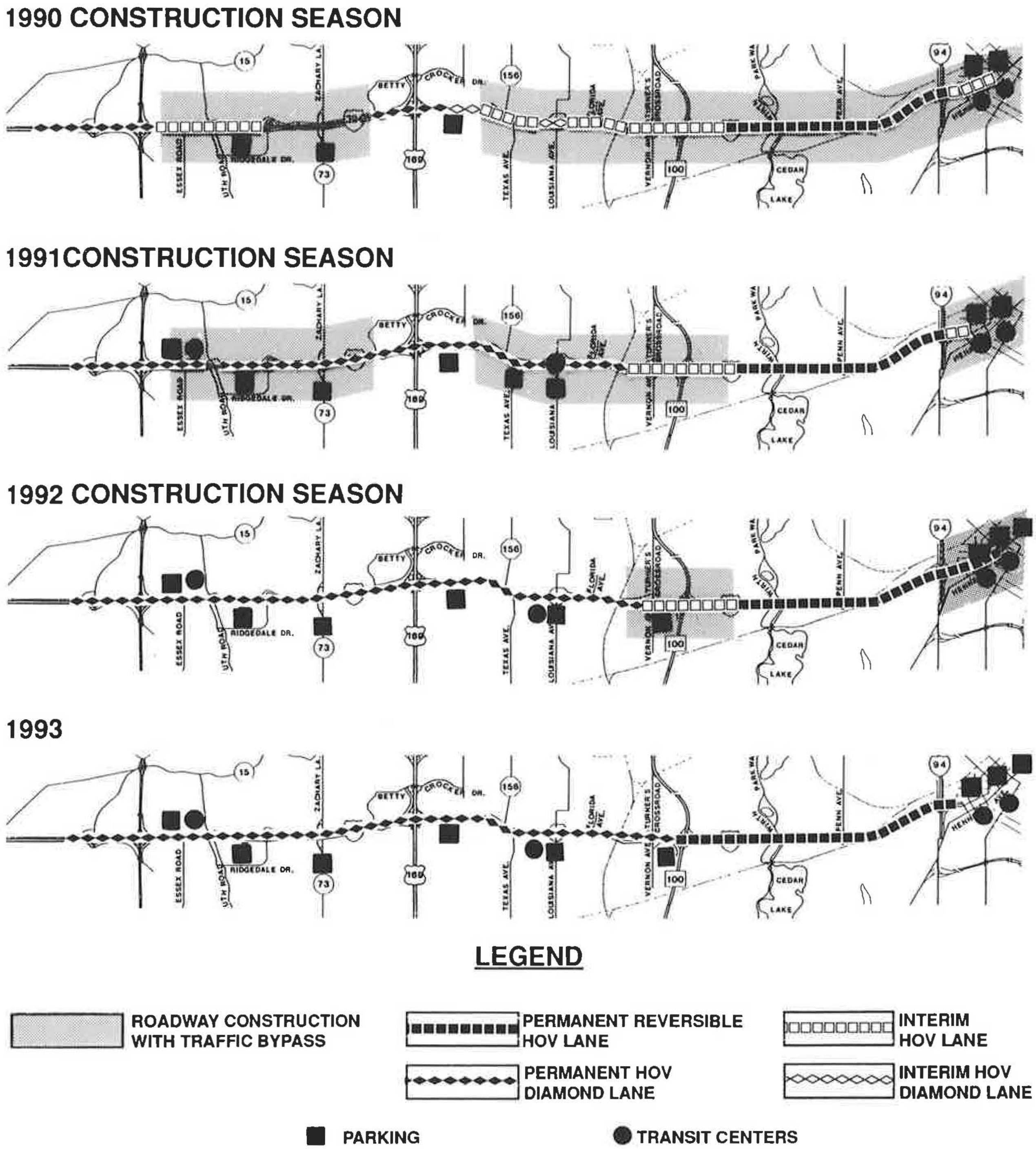

FIGURE 5 (continued from previous page)

lems as they occur, and to make minor redesigns in the field. Mn/DOT generally attempts to settle all contractor claims in the field.

In several situations, the bypass plans have had to be modified in the field to either narrow general traffic lanes or shorten the HOV lane to safely accommodate the HOV lane and its entrances or exits. Generally, the decision to shorten the HOV lane has been based on sight distance for sate merges and on the length of anticipated queues.

Corridor Manager

Use of a corridor manager for the project as well as one construction engineer responsible for all major construction contracts has been an effective management tool. The corridor manager is the chief spokesperson for and coordinates all aspects of the I-394 project. The corridor manager is also the manager and chief advocate of the HOV lane and all other transit elements of the project.

CONCLUSION

Although construction is not yet complete on I-394 and the permanent HOV lanes are not yet open, the project has provided valuable information on the design, operation, and use of HOV lanes on an arterial highway and during a major highway construction project. Key conclusions from the research to date are as follows.

Design and Operation of HOV Lanes

- A reversible HOV lane in combination with concurrent flow diamond lanes on a signalized arterial highway can operate successfully and safely, even during construction;
- The carpool occupancy requirement of two persons was a significant factor in mode change; and
- Opening the HOV lane during special events exposed a high percentage of commuters on US-12 to the benefits and use of the HOV lane.

Incentives for Carpooling

- Time savings is the most important incentive for a mode shift to carpooling,
- Cost savings is the most important incentive for bus riders, and
- Job-related issues are the most frequent deterrent to carpooling.

Impacts of Construction on HOV Lane Use

- Construction causes traffic diversion of HOV lane users as well as people using the regular traffic lane, although the majority of users diverted by construction were carpoolers who had been initially attracted from other routes when the HOV lane was first opened and before construction was initiated;
- HOV lane use decreased when construction first began, but traffic volumes tend to return to similar levels in the fall when construction ends;
- When diversion to the HOV lane from other routes has been discounted, growth in the number of carpoolers previously driving alone on US-12 has been observed, even during construction;
- Even though measured time savings have declined during construction, carpoolers perceive that they are saving as much time using the HOV lane as before construction; and
- HOV lane users feel that the HOV lane is safe even during construction.

Impact of the HOV Lane on Construction Activities

- The HOV lane reduces overall congestion in the corridor, making traffic flow smoother through the construction zones, which makes it easier for the contractor to move workers and equipment on the job.
- Provision of a HOV lane during construction complicates construction staging and traffic switches. Construction costs are increased and construction time may be extended.
- The following three conditions would improve the benefits of using a HOV lane as a construction management strategy:
  —A permanent HOV lane is part of the construction project. The I-394 HOV lane has been effective as an advance version of the permanent HOV lane, allowing Mn/DOT the opportunity to market and promote the advantages of carpooling before the permanent facility is completed.
  —No nearby parallel routes are available for traffic diversion. Travel time savings of the HOV lane and many HOV lane users were lost during construction because there were parallel routes available nearby for traffic diversion. This diversion resulted in improved traffic flow and travel times in the regular lanes. Impacts of a HOV lane during construction would be significantly greater on a roadway with no parallel routes.
  —Regular lane traffic capacity cannot be maintained on the highway under construction. Two lanes of traffic in each direction have been maintained on the I-394 project, which is equivalent to the capacity available before construction. As a result, good traffic flow has been maintained during construction but has reduced the impact of the HOV lane. HOV lanes would have a greater impact on facilities on which regular lane traffic capacity is reduced by construction.

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Agency Practice for Monitoring Violations of High-Occupancy-Vehicle Facilities

G. Scott Rutherford, Ruth K. Kinchen, and Leslie N. Jacobson

Various states monitor their high-occupancy-vehicle (HOV) facilities for violations of passenger occupancy requirements. Few states have long-term programs to monitor violations and little published literature is available. Most current monitoring activities involve human observers; however, new photographic techniques may soon offer improvement.

As urban congestion increases, the need and justification for high-occupancy-vehicle (HOV) facilities increase. Concurrently, the temptation for motorists to violate the occupancy restrictions also increases with increased congestion levels in the general-purpose lanes (*1*). If HOV facilities are to play an increasingly important role in urban mobility, transportation and law enforcement agencies will need to work together to find effective means to maintain violations at a reasonable level or face possible public and political demands for the elimination of HOV facilities.

The methods agencies use to monitor violations on HOV facilities are reviewed. Although the primary objective of this review was to locate and examine agencies that surveyed compliance rates over long periods of time, short-term studies of HOV compliance rates were also reviewed. Although little published information was available on this subject, both published and unpublished literature, as well as telephone conversations with knowledgeable professionals, provided the information presented. This information is not meant to be a complete list of freeway HOV facilities in the United States, or the monitoring methods used in all areas, but rather a sample of the monitoring methods used on some HOV facilities. Given the nature of the information, little detailed data or analysis regarding costs, design effects, and reliability of methods can be presented.

MONITORING ACTIVITIES

Short-Term Monitoring

Short-term monitoring of violation rates on HOV facilities is fairly common and is often used to determine the effectiveness of recently constructed HOV lanes. Because the justification often given to policy makers for the construction of HOV lanes is to significantly increase the people-carrying capacity of the transportation network, transportation agencies usually monitor the new facilities just after their construction to determine their impact on traffic flow.

Violation rates are commonly examined because high violation rates may indicate a need for better enforcement or marketing of the HOV lanes, or a need to make engineering design changes (*2*). Although HOV facilities with high violation rates may improve overall traffic conditions, the HOV lanes may still be unacceptable, because the presence of many violators in the HOV lanes produces poor public perception and may erode public respect for HOV facilities in general.

Another reason for monitoring violation rates on new HOV facilities is that high violation rates may indicate a potential safety problem (*3*). Construction of HOV lanes and their operation should not have a negative impact on the accident rate (*4*). HOV lanes are most likely to affect and be affected by accidents in areas in which the HOV facilities are not physically separated from the mixed-flow lanes. This lack of separation allows vehicles to weave in and out of the HOV lane, creating a potentially dangerous situation, particularly when traffic in the HOV lane is flowing much faster than that in the mixed-flow lanes. High violation rates in such an area indicate a need to study the lanes more closely to determine whether a substantial amount of weaving is occurring.

Another type of short-term monitoring program examines the effects that selected changes in the HOV facility have on the violation rate. Examples of such changes include

- Changes in occupancy requirement (*5,6*),
- New signs or markings (*7*),
- Changes in hours of operation (*8*), or
- An increased level of enforcement (*2*).

Of these changes, an increased level of enforcement is usually reviewed in conjunction with violation rates because enforcement probably has more impact on violation rates than the other modifications. Many agencies examine the effects of changes on HOV lanes as part of the study conducted immediately after the facility has opened.

Long-Term Monitoring

The literature contains few references to ongoing long-term HOV violation monitoring programs. This condition probably reflects the fact that such programs are relatively expensive and may have no immediate impacts on traffic congestion or that in some locations violations are not a major issue. Monitoring program expense is not easily justified when compared with construction activities or other, more visible projects. A

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TABLE 1 LONG-TERM MONITORING METHODS USED BY SOME STATES

Name of State	Location of Facilities	Parameters Measured	Monitoring Frequency	Monitoring Methods and Equipment Used
Virginia	I-66, I-95, I-395 Washington, D.C.	Occupancy; Violation rate	Quarterly	One person per lane records up to six occupants using traffic counter boards.
California	I-10, SR-91, SR-55; LA & Orange Counties	Occupancy	Bimonthly	Two people monitor at each location. One person records occupancy, one classifies vehicles.
Texas	Houston; others proposed	Occupancy	Monthly	One person per lane uses a tape recorder to record data.
Oregon	The Banfield Freeway; Portland	Occupancy, Violation rate	Monthly from 1975 to 1982	One or two people collected data on all lanes of traffic using traffic counter boards.
New Jersey	Entrance to George Washington Bridge; New York City	Occupancy	March 1983 to April 1984; November 1986 to October 1987	Two persons collected data for three lanes of traffic using traffic counter boards.

survey of states identified only three that have continuing HOV violation rate monitoring programs—Virginia, California, and Texas. Two other states, Oregon and New Jersey, have had long-term programs in the past. Table 1 presents these states' monitoring programs.

HOV VIOLATION MONITORING METHODS

Most states that currently have or have had HOV lanes have monitored those lanes for at least a short period just after the lanes were constructed. However, not all states have examined violation rates as part of their initial study, and of those that have, not all of them have included methodology information in their reports. For this reason, letters were sent to states that operate HOV lanes asking for information regarding HOV monitoring methodology. Thus, the methodology information that follows came from sources other than published literature on the subject, including unpublished literature, written responses to a letter, and telephone conversations with the respective operators.

Virginia

The Virginia Department of Transportation annually uses human observers to collect data on HOV violation rates and usage on Interstates 66, 95, and 395. Table 2 presents violation rates for these three facilities and other states. On I-95, the HOV lanes, which have a violation rate of 34 percent, are concurrent-flow, nonseparated diamond lanes, whereas the Shirley Highway (I-395), which is a continuation of I-95, contains two fully separated and reversible HOV lanes and has a violation rate of 2 percent. On I-66, the two lanes in the peak direction are reserved for carpools, buses, and Dulles airport traffic during peak periods. The minimum occupancy

TABLE 2 REPORTED VIOLATION RATES ON SOME FACILITIES

State	Location	Type	Violation Rate*
Virginia	I-95 I-395 I-66	Concurrent, non-separated Fully separated, reversible HOV and airport in peak	34% 2% 20-30%
California	I-10 SR-91 SR-55		
Texas		Exclusive transitway	1%
Oregon	Banfield Freeway	Concurrent, non-separated	(3+) 20% (2+) 10%
New Jersey	George Washington Bridge	Concurrent, non-separated	30%
Colorado	South Santa Fe Highway	Concurrent, non-separated	9-31%
Massachusetts	I-93	Concurrent, asphalt curb Entrance monitored by state police	1%

$$^{*}\text{Violation rate} = \frac{\text{\# Violating Vehicles in HOV Lane}}{\text{Total \# of Vehicles in HOV Lane}}$$

Note: These numbers are very difficult to compare due to many factors and are listed for illustration only.

required for use of the HOV lanes is three persons on all HOV facilities in the state. However, airport traffic has no occupancy requirement. Therefore, monitoring I-66 is a challenge and the reported 20 to 30 percent violation rate is difficult to verify.

On the Shirley Highway, one person observes each lane and records up to six occupants per vehicle on traffic counter boards. In addition, vehicles are classified as cars, public buses, and private buses. Occupancy data on the buses are furnished by the bus companies. On I-66, no trucks are allowed any time and on I-95 and I-395 trucks are allowed with three-or-more-person occupancy.

Other HOV lanes in Virginia are monitored in a similar manner. However, I-95 is more difficult to monitor because the shoulders are quite narrow, making observation of the lanes difficult. High violation rates on this facility, presented in Table 2, are caused by the lack of physical separation between the general lanes and the HOV lane and the difficulty in enforcing the lane. High rates on I-66 are probably caused by Dulles airport traffic.

California

The California Department of Transportation (Caltrans) operates a number of HOV facilities throughout the state. However, only the HOV lanes in the Los Angeles and San Diego areas are monitored on a regular basis. Caltrans monitors occupancy on both mixed-flow and HOV lanes bimonthly on a number of freeways in Los Angeles and Orange counties. The three freeways monitored that include HOV lanes are Interstate 10 (the El Monte Busway) and State Routes 55 and 91. A portion of the El Monte Busway is physically separated from mixed-flow traffic, but the rest of the HOV lane is separated from general traffic by a 13-ft buffer zone. Occupancy requirements also vary among the lanes—minimum occupancy is three on the busway and two on the other lanes.

In order to obtain HOV occupancy rates, data are collected in ½-hr segments by a team of two counters for each location. One person is responsible for counting the number of persons in each vehicle, classifying each as having one through five occupants or six or more occupants. The second person records data on vehicle type, classifying vehicles as vanpools, motorcycles, buses, or trucks. Automobiles are not classified. The information collected from both people is then combined to determine the number and type of vehicles and the number of persons using the HOV facility. Violation rates can be extracted from these data.

Data are collected only on clear-weather weekdays with no unusual traffic conditions. Counts are not made on Mondays, Fridays, or any other days that may exhibit unusual traffic conditions (e.g., the day before a holiday). In general, counts are conducted from elevated positions, to the right of the vehicle passenger side. Examples of such positions are overpasses, pedestrian overcrossings, and the tops of cut areas.

Texas

The Texas Transportation Institute (TTI) collects a wide range of data on the use of the HOV facilities in Houston. HOV facilities in the Houston metropolitan area are reversible, barrier-separated lanes located in the freeway median. Data collected monthly on these lanes include person and vehicle volumes and vehicle occupancy. Additional data on travel times and speeds are collected quarterly (*6*).

The Metropolitan Transit Authority of Harris County (Metro) police (in the Houston area) enforce all transitways. Most violators are cited, with the possible exception of violators who sneak by when an officer is issuing a citation to another person. Therefore, the number of violators using the HOV lane is close to the number of citations issued. The violation rate is currently estimated to be 1 percent (D. L. Christiansen, informal communication).

Human observers collect occupancy data over 3½-hr peak periods. One person observes each lane and records the occupancy of each vehicle by speaking into a tape recorder. Only vanpools and buses are classified because their occupancy can be determined later from data provided by other sources (e.g., from Metro, which operates the buses).

Currently, occupancy rates are determined from the data collected in the field, which is then loaded into a computer. The state has recently ordered new computerized equipment that is capable of recognizing hundreds of words. Information recorded by this machine can be loaded directly into a computer, greatly shortening the time necessary to process those data (D. L. Christiansen, informal communication).

Oregon

Oregon does not currently operate any mainline HOV lanes, although the state does have 14 HOV bypass lanes on metered on-ramps (*9*). The Banfield Freeway, near Portland, did contain a HOV lane in each direction, but these lanes were discontinued in 1982 when construction began for Portland's light rail system, which operates in the same corridor. However, when these HOV lanes were in operation, the Oregon Department of Transportation conducted an extensive monitoring program on the lanes to determine their effectiveness. Violation rates were also determined as part of this study.

Occupancy counts were usually conducted by two people, each of whom used a four-column traffic counter board on three consecutive days (Tuesday through Thursday) on the second full week of the month. One person collected data on the two general-purpose lanes, whereas the second determined occupancy in the HOV lane only. The counters recorded each vehicle as having one, two, or three-or-more occupants, but vehicles were not classified by type. However, when sufficient personnel were not available, one person collected all the data using two four-column counter boards.

When two people were available, data were collected for 10 min in the peak direction and then, following a 2-min break, were collected in the nonpeak direction for 5 min. This cycle was repeated throughout the 3-hr peak period. If only one person was collecting data, only one direction of traffic would be counted per day.

The average numbers of one-, two-, and three-or-more–occupant vehicles were found by taking the average of each over the 3 days. These figures were then used to determine both occupancy and violation rates. Violation rates varied over the course of the lane's operation largely because the minimum occupancy necessary to use the lane was changed. Before February 1979, when the minimum occupancy was lowered from three-or-more to two-or-more persons, the violation rate was approximately 20 percent. However, after the carpool definition changed, the violation rate dropped to about 10 percent.

New Jersey

The Port Authority of New York and New Jersey operates a HOV lane at each Hudson River Crossing between New Jersey and New York (*5*). In addition, the New Jersey Department of Transportation operates several HOV lanes. However, the only HOV lane in the state that has been recently monitored is located on the approach to the George Washington Bridge into New York City. In 1983, the New Jersey Department of Transportation conducted an extensive monitoring program of the bridge when it decided to expand the bus-only lane into a longer bus-carpool lane. Violation rates were examined as part of this study.

Data were manually recorded with a five-button traffic counter. The first three buttons were used to record the number of cars containing one, two, and three-or-more persons. The fourth button recorded the number of trucks, and misses were recorded with the fifth button. If the observer saw a vehicle but could not determine the number of occupants, the sighting was counted as a miss.

If the number of misses was low in comparison with the total traffic volume, this number was included in the total traffic figure but was not used for computing automobile occupancy. However, if this number was large, project personnel then extrapolated how many one-, two-, and three-or-more–person occupied vehicles this figure contained by examining the nonmiss data. Project personnel assumed that the miss data would exhibit the same one, two, and three-or-more split that the nonmiss data had displayed. Excessive misses were considered reason to discount the affected interval's data.

Monitoring locations were three toll plazas, two of which had three outbound lanes. The third plaza had only two outbound lanes. At the larger toll plazas, two people monitored the three lanes. One person observed the outside lane and the other observed the two inside lanes. Data were collected over 5-min intervals, with one person observing the outside lane and the other observing the two closer lanes. Every 5 min, the two observers switched positions, but the counters were set back to zero only after 20 min had passed. At that point, the two observers took a 10-min break, after which counting resumed for another 20 min. The same procedure was used to monitor lanes at the smaller toll plaza, except that a single person monitored the two lanes.

Although the HOV lane was operational only from 7 to 9 a.m., data were collected from 6:30 to 9:30 a.m. Counts were done once per month during the midweek (Tuesday through Thursday), usually during the third week of the month. Data were generally not collected during inclement weather and darkness was not a deterrent because the count locations were relatively well-lit toll plazas.

Results from the monitoring program showed that violation rates varied during the hours in which the lane was in operation. Violation rates averaged about 40 percent during the first and last 15 min of the lane's operation, whereas these rates averaged only 30 percent during the core 1½ hr of operation.

Washington

The Washington State Department of Transportation (WSDOT) does not monitor violation rates on the state's HOV facilities on a continuing basis. WSDOT periodically collects data on HOV violation rates and vehicle occupancy in the HOV lanes, but these studies are not conducted regularly. Little effort has gone into either collecting information on HOV violation rates at a given location over a long period of time or compiling the information that has been collected so that long- or short-term trends in the violation rate can be observed.

The Seattle area has a number of HOV facilities consisting of both concurrent flow HOV lanes in the inside or outside lane and HOV on-ramps, some of which bypass metered general-purpose on-ramps. HOV lanes on the outside shoulder of westbound SR-520 were the first to be opened in the Seattle area (1973), followed by the opening of HOV lanes on the inside lanes of I-5 in 1983 and the outside lanes of I-405 in 1984. The newly constructed (June 1989) I-90 bridge across Lake Washington will also have two reversible HOV lanes when the entire bridge system is completed in 1992. In the interim, there is a single, westbound HOV lane. HOV lanes also exist on SR-522, north of Seattle, and on Aurora Avenue and SR-509 in Seattle.

Several studies have been conducted since the inception of these lanes to evaluate their overall performance. However, these studies have generally focused their attention on the HOV facilities on I-5 (*10,11*) with their intent not to establish violation rates, but to examine a range of parameters, such as HOV volumes, vehicle occupancy in the HOV lanes, and the accident rate on roads that contain HOV lanes. One recent study examined the violation rate on the I-405 HOV lanes associated with an enforcement emphasis program (*12*), and a previous study investigated violation rates on the I-5 HOV lanes after the HERO program was implemented (*11*). However, both studies were short-term monitoring projects and little effort has been spent in monitoring HOV violation rates over the long term.

Generally, human observers with traffic counter boards collected HOV violation data for the studies that examined violation rates. However, observers also used small, portable computers to collect vehicle occupancy data (*13*). The use of these computers offered several advantages. First, the computers were able to record the time of each observation, an ability that largely eliminated the need to supervise data collectors. Second, the data collected could easily be transferred to a microcomputer for further analysis. Third, the computer program for collecting occupancy data allowed the data collector to correct bad observations.

Colorado

Colorado has operated a HOV lane on the South Santa Fe Highway in Denver since October 1986 (*14*). The Colorado

Department of Highways recorded the lane's occupancy and violation rate for 1 year after the lane's inception but no longer monitors the facility regularly. Violation rates during the year the lane was monitored varied from 9 to 31 percent. However, no clear link between congestion in the mixed-flow lanes and the HOV violation rate existed. In fact, the variation may have been caused by the low usage of the lane. Small changes in the actual number of violators could have caused relatively large changes in the violation rate. The low usage of the lane was probably a result of the lack of congestion in the adjacent mixed-flow lanes. This supposition is borne out by the fact that as many as 50 percent of the vehicles eligible to use the lane drove in the mixed-flow lanes instead.

Florida

Although Florida has no ongoing compliance monitoring program, several years ago the University of Florida conducted a study of HOV lane usage on Interstate 95 in Miami (*15*). Violation rates were determined by field observations made from a vehicle driving in the direction opposite to the movement in the HOV concurrent flow lane, which is on the far left-hand lane (*15*). This study could not obtain more specific details on data collection because of changes in staff.

Hawaii

Hawaii incorporated HOV lanes on several roads in the Honolulu metropolitan area—the Kalanianaole Highway and the Moanalua Freeway. Although the HOV lanes on both roads were evaluated several years after they were constructed (*16,17*), the lanes have not been regularly monitored since that time because of safety concerns. The shoulders on the sections of roadway with HOV lanes are extremely narrow (sometimes less than 2 ft wide), thereby making observation of vehicles from the side of the road hazardous. Attempts to monitor the lanes from overpasses have not been successful either (G. Hirokawa, informal communication).

Massachusetts

The only HOV lane in Massachusetts is on southbound I-93 in Boston. This lane, which is only about 4,000 ft long, is separated from the regular traffic lanes by a bituminous concrete curb located in the adjacent lane. The facility is enforced daily by the state police, who position themselves about 1,000 ft beyond the entrance to the lane. About 10 percent of the violators are actually cited; the remainder are directed by the police to reenter the general lanes through an enforcement chute designed for that purpose (C. Sterling, informal communication). As a result of constant police surveillance, violation rates are low, around 1 percent (*18*). Thus, a formal violation monitoring program is probably not necessary.

Minnesota

Currently, Minnesota is operating a single, reversible HOV lane in the median of Highway 12 in Minneapolis (*19*). This facility is only temporary because Highway 12 is being rebuilt into I-394. When complete, the new freeway will include two HOV lanes.

The state department of transportation does not regularly monitor compliance on the temporary facility. However, when citizens complain about HOV violations, the state patrol enforces the lane but finds few people actually violating the lane. Many of the apparent violators have children or dozers aboard who are not easily visible. Actual violation rates are not available.

PHOTOGRAPHIC MONITORING METHODS

Although much interest has been expressed in the use of photographic or video equipment to monitor HOV violations, the review of the literature revealed that no state has used photography for this purpose. Therefore, research was conducted to determine whether photographic equipment that has been used for other traffic monitoring purposes might be applicable to monitoring HOV violations. In addition, research was conducted to identify those factors that influence the selection of photographic equipment.

Three primary considerations that affect photographic equipment selection include the cost, ability of the equipment to take usable pictures in low light conditions, and size of the equipment. The cost of obtaining a camera capable of accurately determining vehicle occupancy is high. However, because the camera can probably be used for other tasks besides monitoring vehicle occupancy, its cost could be spread among several different tasks. Furthermore, although the camera should be able to take usable photographs in low light conditions, such as the early morning or evening hours (7:30 a.m. or 5:00 p.m.), the equipment does not need to determine occupancy at night or under weather conditions in which human observers would be unable to determine occupancy. However, it is extremely important that the equipment be unobtrusive to passing motorists. Highly visible equipment may cause traffic disruptions or at least may cause HOV lane violators to alter their behavior as a result of the camera's presence.

Given these considerations, several types of photographic equipment have promise for determining the occupancy of moving vehicles—still photography, closed-circuit television, and video cameras.

Still Photography

The prototype photographic system developed by the Naval Surface Weapons Center in 1977 is one example of a monitoring system that uses still photography (C. Sterling, informal communication). The system consisted of a 16-mm camera and a flash unit (200 watt-sec), both mounted on a single tripod, and an optical vehicle sensor to ensure that the camera and flash unit operated in unison. The Shirley Highway (I-95) HOV facility in Virginia was selected as a test site for the prototype to ensure that the system could accurately determine the occupancy of vehicles with people in the back seat.

Initial tests of the system demonstrated that, of all the film, filter, and developer combinations tested, black and white infrared film with an infrared filter on the flash achieved the best results. These first tests also revealed that the best vantage point from which to see inside passing vehicles was in front of passing vehicles at an angle of about 45 degrees measured from the axis of the vehicle.

Following the initial development tests, operational tests were conducted to determine how well the system performed under actual, continuous field conditions. The equipment was set up under a ramp buttress to minimize the visibility of the 4-ft-tall system.

Several important results were revealed by these tests:

- Presence of the camera had no adverse effects on passing traffic,
- Equipment was capable of recording the entire peak period without failure, and
- Number of occupants could usually be determined.

However, a major problem was also revealed by the operational tests. Infrared light from the flash unit was absorbed by some car windows without penetrating and illuminating the insides of these vehicles. As a result, human forms in the pictures of these vehicles could be discerned only slightly, if at all.

Several other camera systems using still photography have been developed since the Naval Surface Weapons Center conducted its research. However, these new systems have generally been developed for purposes other than for monitoring HOV violation rates. Zellweger Uster Ltd., a Swiss firm, has designed several high-speed camera systems (R. P. Umdemstock, informal communication). One was designed to photograph the license numbers of vehicles that illegally cross an intersection during a red light. Another system photographs the license plate or the interior of vehicles violating the posted speed limit. Each system is attached to a flash unit and can therefore take pictures at night. Although neither system was designed to monitor vehicle occupancy or HOV violation rates, development might be possible of a camera system specifically for monitoring vehicle occupancy from equipment similar to Zellweger's. However, the cost of using a system to monitor HOV violation rates might well be prohibitive. Zellweger Uster estimated the price of a photo-radar unit on a tripod with a protective steel box to be $72,000. This price does not include installation.

Closed-Circuit Television

The simplest photography monitoring method may be closed-circuit television (CCTV) cameras. WSDOT has approximately 40 of these permanently mounted on roads in the Puget Sound area. However, slightly less than half of these are located adjacent to HOV lanes. In addition, several difficulties may be associated with the use of these cameras for monitoring HOV violation rates. First, no CCTV cameras are located adjacent to the SR-405 or SR-520 HOV lanes; only the I-5 HOV lanes can be viewed through CCTV. Second, the lenses currently installed on the cameras are unable to determine the occupancy of even the front seats of passing vehicles, largely because the cameras were installed to allow WSDOT personnel to scan roads for accidents and other incidents and not to see inside individual vehicles.

These problems do not necessarily preclude CCTV cameras from further consideration as a potential means of monitoring HOV compliance. WSDOT plans to install additional cameras along both SR-405 and SR-520 adjacent to the current HOV lanes. Therefore, WSDOT should determine whether the existing cameras could be fitted with new lenses that could both view the road and focus into the interiors of individual vehicles.

A Texas firm named Traffic Monitoring Technologies is currently experimenting with the use of CCTV in combination with still photography (M. Fustus, informal communication). A TV camera and computer are set up in a vehicle near the HOV facility. A person inside the vehicle monitors the HOV lane and takes pictures of suspicious vehicles. Owners of vehicles who are found violating the HOV facility are then issued citations through the mail. However, because the primary purpose of the system is to enforce, rather than to monitor, HOV facilities, the equipment is not currently set up to photograph every vehicle in the HOV lane.

Video Cameras

Use of video cameras to monitor HOV violation rates should also be considered. Advantages of video cameras over more sophisticated photographic equipment are that the equipment is far less expensive, easier to acquire, and more lightweight and mobile. The video camera Caltrans currently uses to observe traffic is capable of determining the front-seat occupancy of vehicles.

A video system might also be specially designed to photograph the entire interior of moving vehicles. Infrared light might be used to illuminate vehicles' interiors in the morning or evening hours. Further research on newly developed video equipment will be undertaken as part of this project to determine whether such a system could be developed.

CONCLUSION

A review of the literature on HOV lanes and conversations with knowledgeable personnel across the country revealed that long-term monitoring of HOV facilities is rare. In fact, of the states contacted, only three—Virginia, California, and Texas—currently monitor their HOV facilities on a regular basis. Many of the other states surveyed had monitored HOV violation rates at least once in the past.

None of the states reviewed had used any method other than human observers to collect HOV violation data. However, photographic equipment has been used for other traffic monitoring purposes, such as detecting and photographing vehicles that violate the posted speed limits. Three types of photographic equipment were identified that might be used to monitor HOV violation rates, including closed-circuit TV, still photography, and video equipment.

This survey left several important questions to be answered through additional research:

- What is an acceptable level of violations for various facilities?

• What is the relationship between enforcement and violations?

• How do various physical designs affect violations and enforcement?

• Why are agencies not monitoring HOV facilities on a routine basis?

• Should a monitoring manual be developed that details methods and costs?

Many current transportation plans assume that HOV facilities will provide substantial added mobility to urban areas. Given this assumption, monitoring must be considered essential for operational reasons, enforcement, facility justification, and program evaluation. Without solid information on effectiveness, HOV programs will not be able to compete with other facility needs.

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Evaluation Tools of Urban Interchange Design and Operation

A. ESSAM RADWAN AND ROGER L. HATTON

Urban interchanges are a means of facilitating traffic movements between arterial streets and freeway ramps. The single point diamond interchange (SPDI) and the conventional diamond interchange are two specific interchange designs. Essentially, both designs can be treated as signalized intersections. Deviation from the standard signalized intersection operation can be attributed to factors such as longer clearance interval, larger turning radii, different phasing schemes, and different signal offsets between adjacent intersections. Available computer software was reviewed to determine its ability to simulate the operation of the urban diamond interchanges operation. Data collected at two sites in the Phoenix metropolitan area were used. Five programs were chosen: PASSER II–87, PASSER III–88, TRANSYT-7F, TRAF-NETSIM, and TEXAS. An assessment of each program was conducted to determine its ability to simulate both the SPDI and the conventional diamond interchange. It was concluded that the PASSER III–88 and the TEXAS models simulated the SPDI fairly well. All models except the TEXAS model were able to simulate the conventional diamond design.

The Phoenix metropolitan area has been witnessing a substantial growth in population and urban travel. Over the next two decades, close to 230 mi of freeways will be added to the existing network. With this growth in facility design and construction, different designs of urban interchanges need to be considered.

Other metropolitan areas are heavily involved in projects to rehabilitate and reconstruct major sections of the urban Interstate system. Interchanges on such facilities are critical elements of corridor operational efficiency. Whether a new freeway is being added to the network or rehabilitation is being conducted on existing facilities, developing a set of warrants for different types of urban interchanges at any given site is crucial. Such warrants would include factors such as traffic demand, land availability and cost, traffic signal phasing, frontage road spacings, interchange capacity, construction costs, and road user costs.

To be able to evaluate large numbers of design alternatives, computer software that can simulate the operation of urban interchanges must be selected. Two criteria essential to this selection process are ability and credibility, i.e., the ability to simulate a given diamond interchange operation and the credibility of the produced output to reasonably represent the real world.

The main objective of this research effort was to assess available computer software in terms of its ability to simulate two types of urban diamond interchanges: the single point diamond interchange (SPDI) and the conventional diamond interchange.

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BACKGROUND

The conventional diamond, shown in Figure 1, is the most used interchange on urban freeways. Its design is most suitable in suburban and urban locations where traffic volumes are low to moderate and where right-of-way is restrictive. The conventional diamond is characterized by dual intersections, three-phase signal control with overlap movements, and tight turning radii.

The dual intersection design combined with the three-phase signal scheme has been proven to delay the interior movements that account for approximately one-third of the total delay with the conventional diamond (*1*).

The SPDI (also known as the urban interchange) was introduced more than two decades ago. It has recently become more attractive because of its compact design, which requires less right-of-way. The SPDI, shown in Figure 2, operates as a single three-phase intersection. The through movement from the off-ramp can be prohibited, resulting in less delay to off-ramp traffic. One advantage of the SPDI over the conventional diamond is the large turning radius provided for the left-turn movement. The turning radius of the SPDI is approximately 300 ft; for the conventional diamond, it is only 50 to 60 ft (*2*). The larger turning radius allows vehicles to accelerate while turning, resulting in less delay and higher capacity.

A critical element that significantly affects the SPDI's capacity is the signal clearance duration. The distance between the two stop bars on the arterial typically ranges between 150 and 250 ft. Considering an approach speed of 35 mph, the minimum required clearance interval ranges between 7 and 9 sec (*3*). This is usually accomplished by using a yellow and an all-red clearance interval. As the number of left-turn lanes and freeway lanes increases, the minimum clearance interval increases and the efficiency gained by this type of interchange may be quickly lost.

A recent study (*4*) attempted to develop a methodology to determine when grade separations are appropriate. This methodology includes an economic analysis based on a benefit-cost study for ranking three-level diamond interchanges. Delay estimates were derived using the TRANSYT-7F model.

A comparative assessment of the SPDI and the compressed diamond was recently published in the *ITE Journal* (*3*), which stated: "The analyses presented make it evident that applications are limited for the single-point diamond. Generally

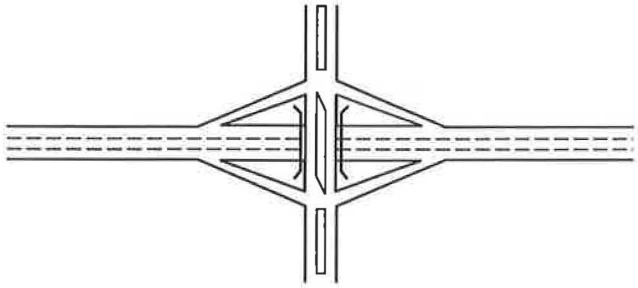

FIGURE 1 Conventional diamond interchange.

speaking, the compressed diamond is less costly, has similar right-of-way requirements, and is more efficient."

The conclusions reached in this study were based on computer simulation runs using the TRANSYT-7F model, which has been highly thought of as an optimization model for large grid networks. The model's ability to simulate isolated diamond interchanges in a realistic fashion is questionable because the platoon dispersion model it uses is mostly suitable for optimizing signal settings of a grid street network and is not appropriate for representing the different geometries of urban interchanges.

The following section addresses the available computer software for traffic signal system analysis and the software's potential applications. Five selected models were used to simulate the two types of interchanges: (a) TRANSYT-7F, (b) PASSER II–87, (c) PASSER III–88, (d) NETSIM, and (e) TEXAS. A detailed analysis was conducted to document the assumptions, advantages, and drawbacks of each model.

TRAFFIC SIGNAL SYSTEM ANALYSIS COMPUTER SOFTWARE

Computer software for traffic signal system analysis can be generally classified into two major categories: macroscopic models and microscopic models. Macroscopic computer models use mathematical expressions to analyze traffic flow over urban streets and to determine a system's measures of effectiveness (such as delay, queue, and fuel consumption). Two types of activities can be carried out using these models, namely, optimization and simulation. The optimization option permits the software user to search for the signal timing plan that results in the lowest vehicular delay. Simulation is applied to predetermined signal settings to assess system performance.

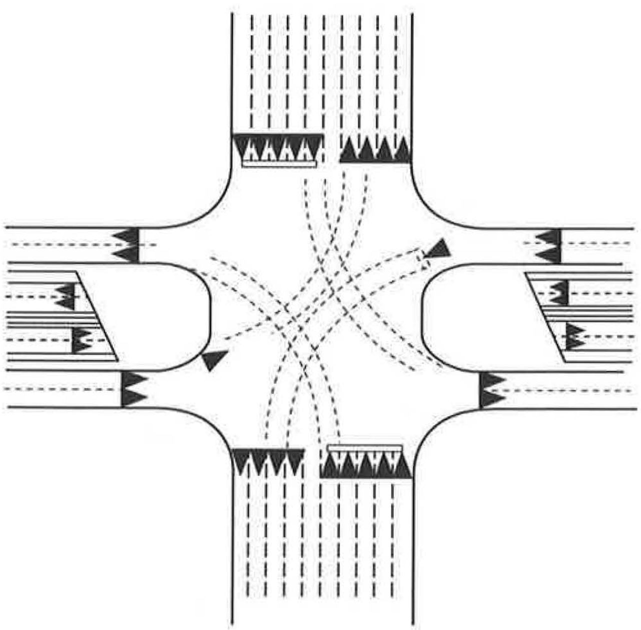

FIGURE 2 Single point diamond interchange.

Examples of macroscopic computer models are SOAP, TRANSYT-7F, PASSER II–87, SIGOP, and MAXBAND. The SOAP model is used for isolated intersections only, whereas the remaining models are tailored for arterials and grid networks. Although the other four macroscopic models are suitable for multi-intersection operation, they can be used to simulate a single- or dual-intersection setting. All five models are deterministic.

PASSER III–88 is another macroscopic computer model. It was developed exclusively to evaluate conventional diamond interchanges and can be applied to an isolated interchange and a frontage road progressive system.

Microscopic computer models simulate individual vehicle movements through the street system and update their status in small time increments. These models can be used to investigate a wide mix of traffic control and traffic management strategies, including pretimed or actuated signal control, sign control, special-use or general-use traffic lanes, and standard or channelized geometrics. Microscopic simulation models are designed to consider different statistical distributions for driver types, vehicle types, gap acceptance, vehicular speeds, and other factors. The ability to simulate vehicle movements in each lane and select different design and control alternatives makes them more attractive than macroscopic models. However, microscopic models require more input data than macroscopic models.

There are three microscopic simulation programs available on the market: NETSIM, TEXAS, and EVIPAS. Although NETSIM and TEXAS are simulation models only, EVIPAS can be used both for simulation and optimization. Both EVIPAS and TEXAS are solely designed for isolated intersections, whereas NETSIM can be used both for isolated and grid networks. The EVIPAS model was excluded as a potential program because it is only available for mainframe computers and requires long execution time.

SIMULATION OF THE SPDI

To evaluate the five selected computer programs, an SPDI was selected in Tempe, Arizona (*5*). This SPDI is the first of its type constructed in that state. A recent count at this site indicated that the p.m. peak-hour volume is 5,011 veh/hr and the 24-hr volume is 65,264 vehicles.

The condition diagram of the interchange is shown in Figure 3. The eastbound approach—University Drive—has four lanes, including a through-right, a through, and two left-turn lanes. The westbound approach has five lanes, including a right-turn, two through, and two left-turn lanes. A raised median separates eastbound and westbound traffic on both approaches. The dual left-turn paths on both approaches are based on a 279-ft turning radius with pavement markings provided through

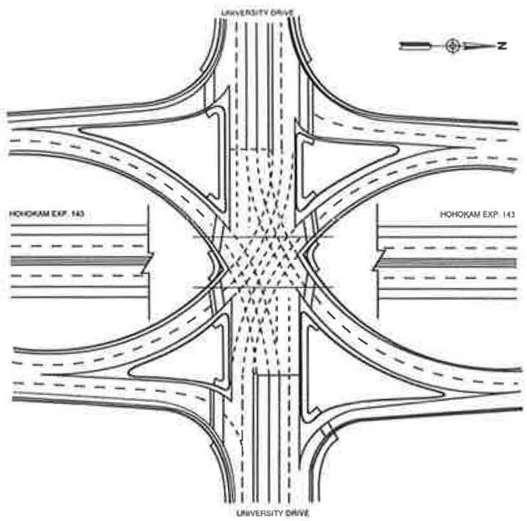

FIGURE 3 SPDI condition diagram.

the intersection. Right-turn channelization is provided on both approaches.

Both the northbound and southbound off-ramps have four lanes, including two right-turn and two left-turn lanes. The dual left-turn paths on both approaches are based on a 270-ft turning radius, and the separation between on- and off-ramp approaches is approximately 215 ft. Channelization of the off-ramp terminals prohibits northbound and southbound movements across the interchange.

The traffic signal phasing is a three-phase operation. The signal phase times are presented in Table 1, and the phase movements are shown in Figure 4. An overlap phase is initiated when either the eastbound or the westbound left-turn movement dissipates before the other. Induction loops are located on all approach lanes for the actuation and extension of each phase.

Traffic data related to vehicular volumes and delay were collected during the evening peak and are summarized in Table 2. All queues cleared the intersection during the respective green phase for each movement, except the eastbound through movement (*5*).

The stopped-time delay measurements were based on 15-sec observations and were conducted for all four left-turn movements. Stopped-time delay is defined as the time during which the traffic is actually standing still (*6*). The travel time delay measurements were made by timing vehicles from the moment they crossed a point located at a certain distance upstream of the stop bar to the moment they crossed the stop bar. This travel distance was set at 800 ft upstream of the stop bar of the left-turn movements for the northbound and southbound directions and 1,000 ft upstream of the stop bar of the through movement for the eastbound and westbound directions (*6*).

TABLE 1 SIGNAL PHASING TIMES (*5*)

	EB-LT	WB	NB-LT	WB-LT	EB	SB-LT
Minimum Green	8	10	5	8	10	5
Maximum Green	30	35	30	30	35	30
Yellow	3.0	4.3	3.0	3.0	4.3	3.0
Red clearance	2.0	6.2	7.8	2.0	6.2	7.8
Vehicle Extension	0.5	1.2**	0.5	0.5	0.2	0.5
Average Phase @ P.M. Peak	16	35*	16	17.5*	34.5	17

* Includes Overlap Phase

**Higher than normal to handle truck traffic

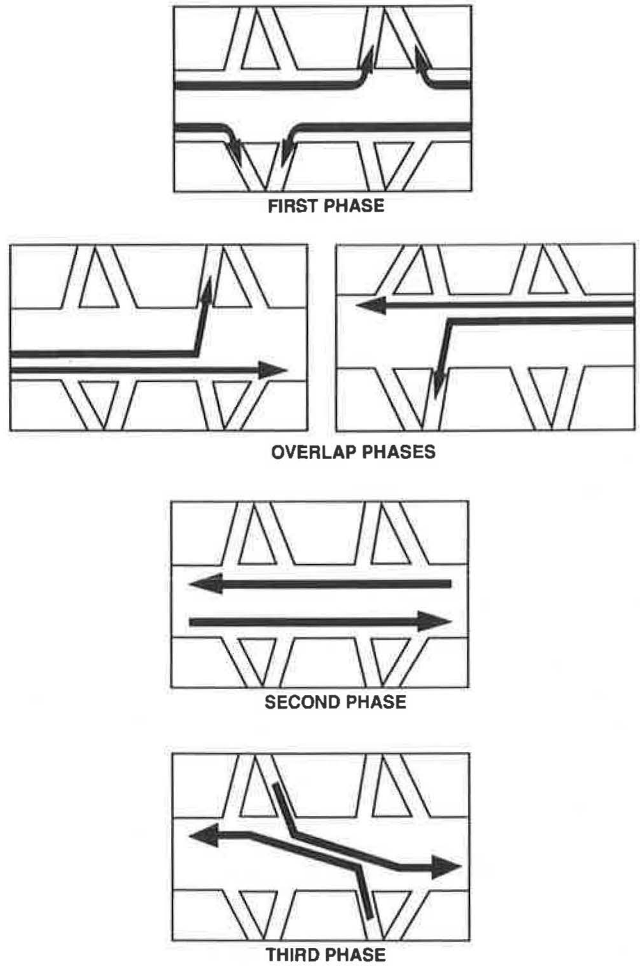

FIGURE 4 Phasing diagram for the SPDI.

PASSER II–87 Software

PASSER II–87 is the most recent release of the Progression Analysis and Signal System Evaluation Routine developed by the Texas Transportation Institute for the Texas State Department of Highways and Public Transportation (SDHPT) (*7*). The basic purpose of the model is to assist the traffic engineer in determining optimal traffic signal timings for progression along an arterial considering various multiphase sequences. It is a macroscopic, deterministic, optimization model. The delay estimate in this model is based on a modified Webster's delay formula to take into account the differences in arrival rates between green and red intervals.

To simulate the SPDI in PASSER II–87, it was assumed that the interchange was composed of two close intersections with zero signal offset. It was also assumed that the saturation flow rate was 1,800 veh/hr of green per lane regardless of the movement type (left-turn or through). The results of the simulation runs are documented in a later section.

TABLE 2 TRAFFIC DATA SUMMARY (*5*)

	EASTBOUND			WESTBOUND			NORTHBOUND		SOUTHBOUND	
	LEFT	*THROUGH*	*RIGHT*	*LEFT*	*THROUGH*	*RIGHT*	*LEFT*	*RIGHT*	*LEFT*	*RIGHT*
Intersection Data										
Number of Lanes	2	2	0	2	2	1	2	2	2	2
Peak Hour Volume	463	1227	308	635	592	583	72	331	568	232
Delay (Seconds)										
Stopped Time Delay	30.7	-	-	32.8	-	-	29.6	-	31.6	-
Travel Time Delay										
at 1000 ft	-	84	-	53.4	44.7	18.5	-	-	-	-
at 800 ft	-	-	-	-	-	-	45.0	-	54.2	-

Some of the advantages of using the PASSER II model are the simplicity of data input and the ability to simulate different types of signal phasing schemes. However, the model has several disadvantages. The main disadvantage is the inability of the user to input the appropriate clearance interval of each phase. One possibility to overcome this shortcoming is to add the clearance interval to the green phase. Regardless of the user input, the model assumes a fixed phase lost time of 4.0 sec, and the user has no way of changing this value. Another disadvantage is that the model has no special treatment for right-turn traffic and no allowance for separate right-turning lanes.

PASSER III–88 Software

PASSER III–88 is the recent release of the Progression Analysis and Signal System Evaluation Routine—Model III for diamond interchanges (*8*). The model was developed to determine the optimal phase patterns, splits, and internal offsets at signalized isolated interchanges. In addition, the model is capable of optimizing system cycle length and progression offsets for one-way frontage roads.

The isolated interchange logic simulates the operation of the interchange into both a right and a left side. Five basic signal phases are allowable. There are two possibilities for off-ramp left turners: either lead or lag to the on-ramp movement. Several phasing combinations can be generated, and the model tests these alternatives to find the optimum pattern and offsets. Webster's delay equation is used for the optimization process.

Because PASSER III–88 is designed exclusively for the conventional diamond, several assumptions had to be made to simulate the SPDI operation. No apparent advantages were identified in using this software. As with PASSER II–87, the clearance interval was overlooked in the analysis process. However, PASSER III–88 provides for a separate analysis of right-turning movement as well as a simulation of separate turning lanes. One severe limitation is that PASSER III–88 is a fixed-sequence program, which limits its application to the five predetermined sequences.

TRANSYT-7F Software

The TRANSYT-7F model is the U.S. version of the Traffic Network Study Tool model developed by the Transport and Road Research Laboratory of Great Britain (*9*). It is a macroscopic, deterministic, time-scan model for optimizing the signalization on arterials and grid networks.

Intersections are represented by nodes in TRANSYT, and links represent streets connecting those intersections. Each link can represent a traffic movement at any given node. The SPDI is represented by coding two nodes with zero offset. The delay model used is a modified version of the Webster formula. The hill-climbing procedure is not applicable for diamond interchanges because there is no offset between the two signals. The model is used only for simulation.

The advantage of using TRANSYT is the ability to simulate each movement separately and to include the clearance interval for each phase. Furthermore, the model can simulate other geometric features such as the location of the stop bar and the location of transit stops. The results of the simulation runs for the SPDI are documented in a later section.

TRAF-NETSIM Software

TRAF is a system of traffic simulation models designed to represent traffic flow on any existing highway facility. The abbreviation "NETSIM" is composed of the prefix "NET" for surface street network and the suffix "SIM" for microscopic simulation. Individual vehicles are simulated through the system along the links, according to specified controls at nodes (intersections), stochastically determined turning movements, and deterministic car following. No set paths are modeled because turning movements are purely random.

NETSIM has a multiplicity of features and user options. Virtually any feasible geometric configuration, traffic control system, traffic management strategy, and demand configuration can be modeled. The type of network may vary from a single intersection to a complex grid network.

The most recent release of TRAF-NETSIM can produce static and dynamic representation of traffic movements (*10*).

The static option produces static graphs that report on links and network-wide measures of effectiveness such as stopped delay and queue length. The dynamic logic produces animation of individual vehicle movements on selected links. Animation is produced for one node only and is limited to a distance of 500 ft on either side of that node.

To simulate an SPDI operation in NETSIM, the interchange is split into two intersections 215 ft apart. Pretimed signal operation was implemented with a three-phase plan. Although NETSIM can simulate actuated signal operation, the SPDI operation was simulated as pretimed for two reasons: (a) the actuated logic requires extensive data collection, which was not possible for this site, and (b) although all other types of macroscopic software (such as PASSER and TRANSYT) have the ability to evaluate actuated signal schemes, they do not simulate the actuated operation in the true sense. More specifically, macroscopic models do not change the green duration of a signal phase from one cycle to the other as do microscopic models. The average phase durations documented in Table 1 were used as NETSIM input. Because of the stochastic nature of NETSIM, 10 runs were implemented with different starting random number seeds, and the average stopped delay was then calculated. The results of the NETSIM runs are documented in a later section of this paper.

The advantages of using NETSIM to simulate the SPDI operation are as follows:

1. Different intersection geometries can be tested.
2. The graphic presentation of this system makes it easy to check the network's geometrical configuration and the effectiveness of the control strategy.
3. NETSIM does not have a delay model like other macroscopic models, so the user does not have to be concerned about the validity of the formulas. The different statistical distributions embedded in NETSIM can be used to make the measures of effectiveness produced by the model match those observed in the field.

The drawbacks of using NETSIM include the following:

1. The model cannot simulate vehicle paths in the intersection. It makes the vehicle disappear from one link right at the stop bar and then recreates it on a downstream link. The advantage of the large turning radii provided by the SPDI is then underestimated.
2. Like any other microscopic model, the interchange is represented by two close intersections, and statistics are collected for the whole network.
3. Vehicles are generated according to a uniform distribution, and the arrival pattern cannot be changed as it can in the TEXAS model.

TEXAS Software

The Traffic Experimental and Analytical Simulation model is a microscopic simulation program developed to analyze alternative designs of isolated intersections (*11*). The model can simulate any intersection configuration controlled by stop signs, yield signs, or traffic signals. Like NETSIM, the TEXAS model can simulate all traffic signal schemes, including two-phase to six-phase pretimed controllers; eight-phase, dual-ring, semiactuated or full-actuated controllers; and permissive or exclusive left-turn control schemes.

The TEXAS model can be used effectively to simulate the SPDI operation. The interchange is coded as an isolated intersection with an extra-wide median between the on- and off-ramps. The user can input different arrival distributions for each approach of the interchange. These options make the TEXAS program more attractive than the NETSIM model. Furthermore, the ability to display traffic movements on the computer screen (animation option) adds more credibility to the model.

The TEXAS model has the following limitations:

1. The interference to traffic caused by pedestrians moving simultaneously with vehicular traffic cannot be simulated.
2. The model does not simulate the effect of approach grade. This can be compensated for somewhat by using different headway distributions.
3. There is no provision for coordination of or the effect of adjacent signals. This factor becomes a critical consideration when the urban interchange is part of a major arterial with a predetermined progression plan.

Results of the SPDI Simulations

Stopped delays were adopted for the comparative assessment of the five computer programs. These observations were made for left-turn movements only on all four approaches. Both NETSIM and TEXAS determine stopped-time delay comparable to the procedure used in the field, recording the queue length every 15 sec. However, the two PASSER programs and the TRANSYT program use a modified Webster delay equation. The original Webster delay equation is essentially based on total vehicular delay, which includes stopped-time delay and delay incurred by vehicles during the deceleration and acceleration cycles. Therefore, delay figures produced by these programs were expected to be relatively higher than the observed values.

Table 3 contains the field results and the simulated results. Using the p.m. peak-hour field observations as a benchmark for comparison, it can be observed that the results of PASSER III–88 were the closest to the field results for the three macroscopic models under evaluation. As for the microscopic models, it appears from the first glance that the NETSIM model produced comparable results to the field data on the basis of the weighted average delay figures. Closer examination of the results reveals that the TEXAS model seems to be more suitable for simulating the SPDI because it produced delay figures close to the field data figures for three out of the four movements. The stopped-time delay data for this site were collected for 1 hr of a given day. More sites and more observations for each site should be collected to provide a more credible assessment.

Initial findings of this research indicate that the TEXAS model has good potential for use as an evaluation tool of the SPDI performance. While this research was being conducted, the Transportation Research Center at the University of Texas (the developer of the TEXAS model) modified the current version of the model to simulate urban interchange operation.

TABLE 3 SIMULATION DELAY RESULTS OF THE SPDI

		East-Bound Left Turn	*West-Bound Left Turn*	*North-Bound Left Turn*	*South-Bound Left Turn*	*Weighted Average of All Four Movements*
PM Peak Hour Traffic Volume		463	635	72	568	
Macro. Models	**PASSER II-87** (sec/veh)	38.3	56.20	27.10	32.80	42.57
	PASSER III-88 (sec/veh)	23.17	39.68	29.37	32.76	32.59
	TRANSYT-7F (sec/veh)	39.8	76.50	32.80	46.80	55.21
Micro. Models	**NETSIM** (sec/veh)	2.82	5.54	36.62	84.12	31.78
	TEXAS (sec/veh)	36.83	100.40	32.73	36.44	59.75
	Observed Data (sec/veh)	30.70	32.80	29.60	31.60	31.71

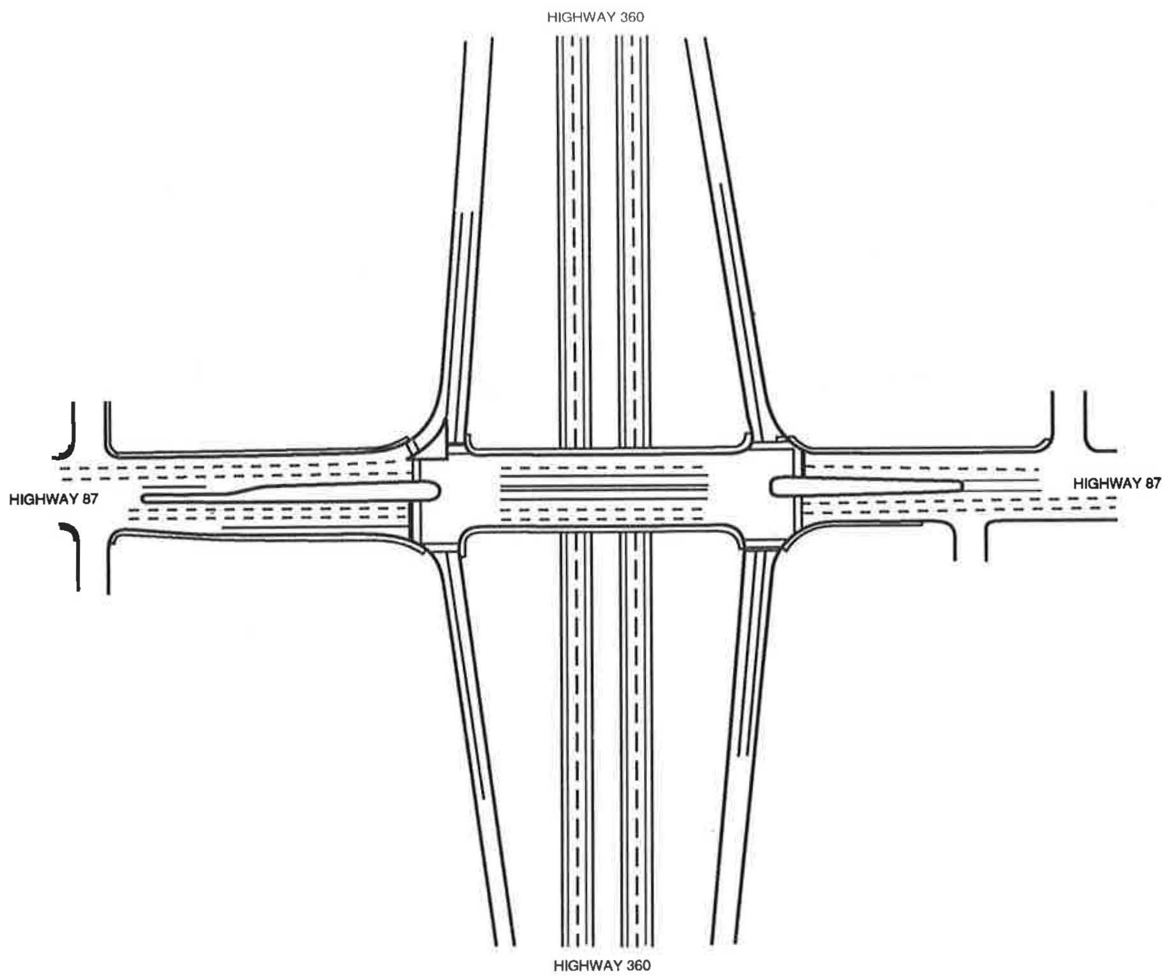

FIGURE 5 Conventional diamond condition diagram.

TABLE 4 TRAFFIC DATA SUMMARY FOR CONVENTIONAL DIAMOND INTERCHANGE (5)

	S.R. 87 (Arterial)						*S.R. 360 Off-Ramps*			
	SOUTH-BOUND			NORTH-BOUND			WEST-BOUND		EAST-BOUND	
	LEFT	*THROUGH*	*RIGHT*	*LEFT*	*THROUGH*	*RIGHT*	*LEFT*	*RIGHT*	*LEFT*	*RIGHT*
Number of Lanes	1	2	1	1	3	1	1.5	1.5	2	1
Volume of P.M. Peak Hour	271	1316	712	302	1307	303	335	281	517	345
Average Phase Length (Seconds)	16.8	30	-	18.5	30	-	25		25	
Yellow	4.5	4.5	-	4.5	4.5	-	4.0		4.0	
Red Clearance	2.0	2.0	-	2.0	2.0	-	2.0		2.0	

Cycle Length = 92.50 Seconds
Lost Time = 19.0 Seconds

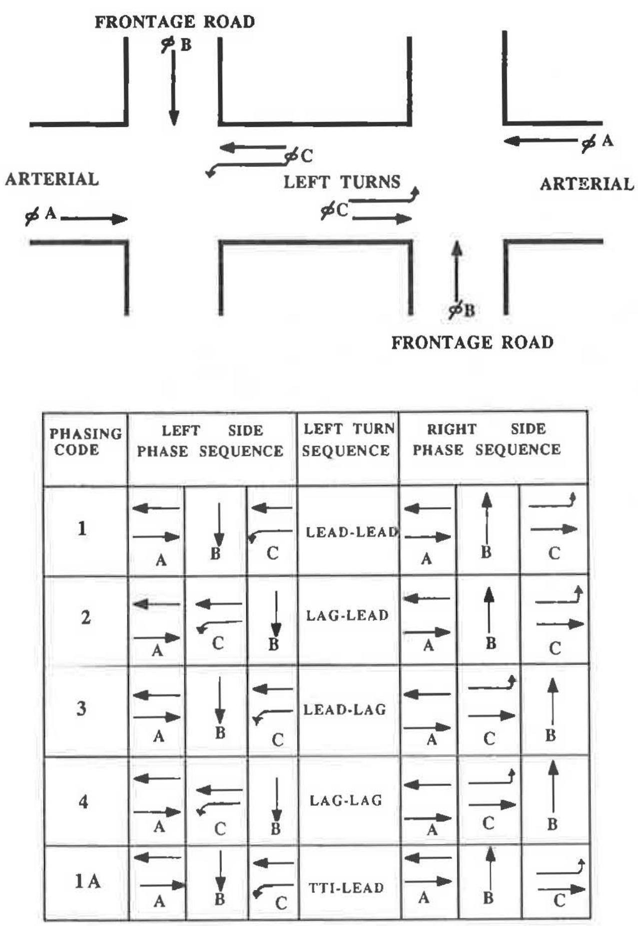

FIGURE 6 Phasing code description used by PASSER III-88.

No public release has yet been made of that version, and the Texas SDHPT is currently evaluating the program.

SIMULATION OF THE CONVENTIONAL DIAMOND

The conventional diamond interchange is the interchange most commonly found in urban areas where the arterial street is carried over or under the freeway facility. Accessibility to and from the freeway is provided by four ramps. The interchange can be treated as two isolated signalized intersections 250 to 350 ft apart. The storage area on the bridge deck is a critical element in determining the interchange capacity. The signal phasing scheme of this type of interchange may specifically affect its performance.

To test the ability of the PASSER II-87, PASSER III-88, TRANSYT-7F, TRAF-NETSIM, and TEXAS models to simulate the conventional diamond interchange operation, an interchange in Mesa, Arizona, was adopted (*5*). Figure 5 shows a layout of the conventional diamond, and Table 4 presents a traffic data summary of this interchange.

As shown in Figure 6, the five phasing schemes adopted by the PASSER III-88 model were used to test the computer software. No delay data were available for the conventional diamond in Mesa; however, the delay statistics produced by the computer models were used to conduct a comparative assessment to determine if models produce comparable results.

The coding of the conventional diamond could not be accomplished for the TEXAS model because the phasing code for this model is limited to a single intersection. Therefore, the assessment was limited to the other four programs. Table 5 presents the delay results in seconds per vehicle as calculated

TABLE 5 DELAY RESULTS OF CONVENTIONAL DIAMOND (sec/veh)

PHASE ORDER	PASSER II-87	PASSER III-88	TRANSYT-7F	NETSIM
Lead-Lead	45.10	39.06	181.97	26.80
Lag-Lead	43.90	38.66	185.79	66.20
Lead-Lag	44.10	37.96	181.04	58.60
Lag-Lag	42.90	37.11	184.87	26.00
TTI-Lead	44.70	62.57*	181.49	42.80

*Simulated for a cycle length of 120 seconds

by the four programs. The three macroscopic models produced comparable results with respect to the five phase orders under consideration. The only exception was observed for the Texas Transportation Institute (TTI) lead phase as applied to the PASSER III program. The difference between the TTI scheme and the lead-lead scheme is that the TTI phase order considers a 12-sec offset between the left side of the interchange and the right side. PASSER III logic did not permit the TTI evaluation for a 95-sec cycle length, and the cycle length had to be increased to 120 sec. This change was probably the reason for the higher delay figure in this phase.

The results of the NETSIM model varied more than those of the other models for different phase orders. This finding was expected for two reasons. The first is that NETSIM is a microscopic model and traffic events are processed in smaller time increments, allowing more accurate calculation of the impact of signal phasing schemes on vehicular delay. The second reason is attributed to the stochastic nature of the model, which would be included in the analysis of the effects of random events.

Another significant issue is the absolute values of the delay figures produced by the models. Both PASSER models produced comparable delay values, but the TRANSYT and the NETSIM models did not. Because of the lack of field data availability at conventional diamond interchanges, no conclusions can be reached concerning which model is more accurate in simulating the conventional diamond.

CONCLUSION

It was concluded that all five computer programs evaluated can simulate the SPDI operation. Each program has unique features that make it more attractive than the others. From the limited number of runs made, it appears that the PASSER III–88 and the TEXAS model results were the closest to the field data.

All programs except the TEXAS model were able to simulate the conventional diamond design. It was not possible to determine the effectiveness of these simulations without extensive field data to conduct successful model validations.

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Passing Operations on a Recreational Two-Lane, Two-Way Highway

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A study was conducted of passing maneuvers in selected passing zones over a 21-mi segment of a principal arterial, two-lane, two-way roadway in northern Wisconsin. Within the project, passing was restricted to 33 percent of the roadway length due to horizontal and vertical geometrics. Five independent sites were monitored using both field observers and directional speed and volume recorders. Timed video-recorders were also placed at two sites. The data were gathered on three peak summer weekends in 1988. Operational data, segregated into 15-min intervals, included information on directional flows, average speeds, and passing operations. The data conformed with AASHTO expectations and indicated that on recreational weekends with traffic volumes of 200 to 250 veh/hr in the major direction and 85 to 175 veh/hr in the minor direction, 25 to 35 percent of all passes were made in the presence of an opposing vehicle, an average of 21 passes per hour were completed with duration of time in the opposing lane (at 60 mph) of 12.2 sec, and pass aborts comprised 0.8 percent of all passes. Under higher flow levels of 330 to 420 veh/hr in the major direction and 70 to 170 veh/hr in the minor direction, 26 to 50 percent of all passes were made in the presence of an opposing vehicle, an average of 16 passes per hour were completed with duration of time in the opposing lane of 11.3 sec, and pass aborts comprised up to 7 percent of all passes where passing was restricted to 33 percent for a 10- to 15-mi segment. The data indicate that (a) the passing driver's decision threshold is negatively affected by the inability to pass, (b) this effect increases significantly at volume levels as low as 500 veh/hr two-way, and (c) passing drivers may be significantly overestimating their ability to complete passing maneuvers safely.

US-63 from Turtle Lake to Cumberland is an 11.1-mi principal arterial extending from the intersection with US-8 in Turtle Lake to the intersection with SH-48 west of Cumberland in Barron County, Wisconsin. Its location is approximately 1 hr northeast of the Minneapolis–St. Paul metropolitan area, and the route is a major recreational highway to northern Wisconsin and Lake Superior. The roadway was originally constructed in 1936 as a 20-ft-wide concrete pavement with 8-ft gravel shoulders. It was resurfaced and widened with bituminous in 1973 to a 24-ft pavement with 10-ft gravel shoulders.

The roadway geometrics are satisfactory for 60-mph horizontal operations, although several long horizontal curves limit passing sight distance. The vertical alignment has 52 vertical curves, of which 46 have stopping sight distance for 55 mph, 5 are adequate for 55-mph stopping sight distance, and 1 is marked for 45-mph stopping sight distance. In general, the combined geometrics offer an MUTCD marked passing opportunity of approximately 32 percent northbound and 33 percent southbound within the 11.1-mi segment.

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The traffic volume in 1985 was recorded as 2,530 veh/day with a peak-hour volume of 24.1 percent of average daily traffic (ADT). However, a nearby average annual daily traffic (AADT) station on US-8 reported that Sunday peak traffic in July was 235 percent in excess of the annual weekday ADT, with averaged summer (May to September) weekend volumes approximately 80 percent above the monthly AADT. In the peak direction on weekends, video-recorded data from this project revealed that passenger vehicles make up approximately 67 percent of the traffic flow, light trucks and vans 26 percent, vehicles with trailers 6 percent, and RVs and large trucks 1 percent of the average peak flow.

From 1978 to 1986, this segment experienced a total of 90 accidents—an average of approximately 11 accidents per year. These accidents resulted in 71 injuries and 6 fatalities, or about 1 fatality and 9 injuries per year. A review of the 1985 through 1987 accidents indicated that 67 percent of them were vehicle-vehicle and 22 percent were passing related. Although the average annual accident rate is reported as only 45 per 100 million vehicle-miles (mvm) for this segment compared with 220 for this roadway type statewide (indicating a safe roadway on average), a severity of 77 fatalities and injuries per 90 total accidents indicates the risk of substantial severity when an accident does occur.

A proposed project on this segment will reconstruct the entire 11.1-mi roadway by improving the substandard curves to a minimum 60-mph stopping sight distance, retaining the same two-way passing sight distances. The typical section will provide 12-ft lanes with 6-ft shoulders, of which 3 ft will be paved. Passing lanes approximately 1 mi long will be added at two locations in each direction, which will increase the passing opportunity to 67 percent northbound and 56 percent southbound. Neither of the passing lanes is adjacent to the other so that a driver will not perceive the presence of a four-lane roadway. To quantify the full effect of the new passing lanes, an in-depth collection of data related to the passing maneuver was performed (*1*).

SITE LOCATIONS AND DATA COLLECTION

Collection of the roadway data was performed on three nonconcurrent weekends—July 4, 15, and 30, 1988. These particular dates were selected because the weekend of July 4 is the peak traffic weekend of the year, and by retaining the weekends in 1 month large shifts in driver population would be minimized. Also, by selecting nonconcurrent weekends, it was hoped that the driver population would not become sensitized to expectancies of traffic studies in progress on the roadway.

Five passing zone observation sites were selected to provide a depth and breadth to the data collection while maintaining control of the data. Sites 2, 3, and 4 were selected for observation within the 11.1-mi project limits, and Sites 1 and 5 were selected for observation significantly beyond the termini of the project at each end. The external sites were selected at a distance of approximately 5 mi from the termini of the project to establish if, as reported by Harwood (*2*), such passing lanes have an impact on operations a significant distance from the project. The external sites were selected as the closest, longest passing zones from each terminus in which passing is relatively easy and safe if flow levels are low. Both zones were about 1 mi in length with relatively flat, tangent geometrics. The internal observation sites were selected at approximately 2- to 3-mi intervals to provide data from the short 2,000-ft passing zones at Sites 2 and 4 to the longer 3,600-ft zone at Site 3. Sites 2 and 4 were selected for placement of the video-recorders because these sites offer a well-balanced perspective of traffic flow at a distance of about 2 mi from each terminus.

Roadside observers trained in the collection of passing data were placed at each site. To minimize disruption to the traffic stream and external influences on driver behavior caused by the presence of the observers, they were either sufficiently hidden from the drivers' view or were dressed as resting bicyclists (complete with easily observed bicycle). The intent of this strategy was to alleviate the threat of driver behavior modification due to the presence of an unusual observer at the roadside. Similarly, the observers were placed at roadside locations as far from the traveled roadway as possible, while retaining their visual capability, so their location would not interfere with driver performance by appearing as a roadside obstacle. Video-recorders with time (in minutes) imprinted on the image were stationed at two of the internal sites for two weekends and at the two external sites for one weekend. The video-recorders were wrapped in black or green plastic wrap, which blended into the natural background at the sites. In general, the bicycle and video-camera disguises appeared extremely effective in maintaining the integrity of the observers as a natural part of the roadway environment, thus preserving the drivers' existing operational and safety characteristics.

Pneumatic road-tubes were placed at each site to record flow and speed in the major direction of travel. On each weekend, the speed and volume recorders were set for the northbound (away from home) direction from noon on Friday to 3:00 p.m. on Saturday and were then transferred to the southbound (to home) direction from 3:00 p.m. on Saturday to 9:00 p.m. on Sunday (or Monday, July 4).

Following is a summary of the site locations and the type of data collected at each.

Site 1

Site 1 is an external location designed to provide stability to the data collection by the observation of substantial passing maneuvers and to measure the effect of passing lanes a significant distance from the termini of the lanes. The observer was located approximately midway within the passing zone, which begins about 4.5 mi west of the intersection of US-8 and US-63 in Turtle Lake. The passing zone, which is on a tangent alignment, is 1.25 mi long (6,600 ft), with a flat grade in both directions at the point of pass initiation. Speed and volume data were collected from road-tube counters on each weekend, and videotape data were recorded on the last weekend.

Site 2

Site 2 is an internal site, with the passing zone located approximately from Station 145 to Station 163 (1,800 ft). Observers were located at Station 158 northbound and Station 163 southbound. Both observation sites are approximately 2.0 mi from the beginning of the segment at US-8 in Turtle Lake, which is Station 56. The horizontal alignment is tangent in the northbound direction and has a slight curve to the right in the southbound direction. This passing zone has a downhill grade northbound (−1.5 percent at the pass initiation point) and a slight downhill grade southbound (−0.5 percent at the pass initiation point). Road-tube speed and volume data were recorded in 15-min increments on each of the three weekends, and videotape data were recorded on the first and second weekends.

Site 3

The passing zone at Site 3, an internal site, is located approximately from Station 412 to Station 452 (3,600 ft). Observers were located at Station 429 northbound and Station 438 southbound. Both observation sites are located approximately 0.5 mi north of the northern limits of the village of Comstock, or 7.2 mi north of the intersection with US-8 in Turtle Lake. The horizontal alignment is tangent, with a downhill grade northbound (−2.0 percent at the pass initiation point) and a slight downhill grade southbound (−0.5 percent at the pass initiation point). Directional speed and volume data were collected on each of the three weekends using road-tubes. No video-recorded data were gathered for this site because it is midway between Sites 2 and 4, and data from those sites could be averaged to approximate the directional flows and vehicle classifications at Site 3.

Site 4

The passing zone at Site 4, an internal site, is located approximately from Station 550 to Station 571 (2,100 ft). The observer was located at Station 558 for both the northbound and southbound passing observations. The observer location was about 9.5 mi north of the intersection with US-8 in Turtle Lake. The horizontal alignment is tangent throughout this site, with an uphill grade northbound (+1.8 percent at the pass initiation point) and a downhill grade southbound (−3.0 percent at the pass initiation point). Directional speed and volume data were collected on three weekends, and video-recording data were taken on the first and second weekends.

Site 5

Site 5 is an external site. The passing zone is located approximately 7.1 mi north of the northern terminus of this project at SH-48, which is about 4.5 mi north of the northern limits of the village of Cumberland. This passing zone was selected for observation because, at approximately 1.1 mi (5,800 ft) in length, it is the first significantly long passing zone north of the project limits. This site is on a slight uphill grade northbound (+0.5 percent at the pass initiation point) and a downhill grade southbound (−1.5 percent at the pass initiation point). The horizontal alignment is essentially tangent throughout the passing zone. Three weekends of directional road-tube speed and volume data were collected, and videotape data were recorded on the last weekend.

PASSING DATA COLLECTION PROCEDURES

The field observers placed at the five sites over the three weekend periods were instructed to observe all passing maneuvers and record the following data.

Time in the Opposing Lane

This characteristic of the passing maneuver was estimated by using a stopwatch to record the elapsed time between the crossing of the centerline by the passing vehicle's left front tire and the return of the vehicle's left rear tire to the lane of origin. This time corresponds to the AASHTO definition of the time ($t2$) during which the driver occupies the left lane (*3*). Although it is often difficult to measure this event precisely because of the distance and skew between the vehicle and the observer, it becomes easier with experience to recognize which vehicles intend to pass and to record the event with relative precision.

Pass with No Opposition

This type of pass represented those completed with no opposition to the pass observed at the so-called "critical position" (alongside the passed vehicle) and with no difficulty or conflict encountered in the completion of the pass. The critical position is assumed to be the point at which the passing driver evaluates the outcome of the pass attempt and either proceeds or aborts the pass.

Pass with Opposition Greater than 10 sec

In this type of pass, the pass was completed with no conflict but with an opposing vehicle 10 sec or more from the completion of the pass (return of the left rear tire to the original lane) at the critical position. The time between the passing vehicle's return to the original lane and the point at which the vehicle trajectories would meet was estimated using verbal counting (i.e., "1001, 1002, 1003"). Although verbal counting is admittedly an inaccurate data collection technique, it does present a relative measure of the clearance time between vehicles, which is an estimate of the AASHTO $t3$ clearance interval. Funding constraints precluded a more refined measurement of the $t3$ characteristic.

Pass with Opposition Between 5 and 10 sec

This type of pass represented those completed with an opposing vehicle in sight at the critical position and with a vehicle clearance within the range of more than 5 sec but less than 10 sec from the completion of the pass to the opposing vehicle. This is referred to as a slight passing conflict because the passing driver is taking a greater risk than in other passing maneuvers.

Pass with Opposition Less than 5 sec

In this type of pass, the pass was completed with an opposing vehicle in sight at the critical position and with a vehicle clearance of less than 5 sec from completion of the pass to the opposing vehicle. This is referred to as a major conflict because these drivers are taking a significant risk with the potential of a severe outcome to themselves and their passengers should the pass attempt fail. Roadside observers were instructed to record as much detail as possible about the vehicles and character of this type of passing event.

Pass—Full Abort

This type of pass represented the most serious and life threatening of passing maneuvers with the vehicle completely entering the opposing lane and then retreating to the lane of origin after concluding (due to the presence of an opposing vehicle) that the pass could not be completed safely. This type of event is a clear indication of great risk taking on the part of the passing driver and indicates the trade-off between primacy (which accepts safety as the ultimate goal of driving) and the presence of overpowering delay (which alters the passing driver's disposition to pass from a normal risk-taking level to a level of significantly elevated risk).

Multiple Pass

This type of pass represented a pass extension, in which one vehicle passed two or more vehicles, two or more vehicles passed one or more vehicles, or any similar combination of passes occurred. It was assumed that multiple passes are an undesirable by-product of inefficient passing operations on two-lane roadways and that they indicate elevated risk taking on the part of the passing driver. Roadside observers were instructed to record the character of the multiple passes with as much detail as possible, including the number and character of vehicles involved.

PASSING DATA COLLECTION AND ANALYSIS

On the basis of the procedures described in the preceding section, passing, speed, and flow data were collected at 15-min intervals from each site as follows:

- *Site 1.* At this site, 135 intervals of data were developed and 391 passes were observed in the northbound direction.

In the southbound direction, 74 intervals of data were collected and 456 passes were observed.

- *Site 2.* At this site, 156 intervals of data were produced and 273 passes were observed in the northbound direction. In the southbound direction, 95 data intervals were recorded and 242 passes were observed.
- *Site 3.* At this site, 156 data intervals were recorded for the northbound direction, with 489 passes observed. In the southbound direction, 81 data intervals were recorded and 466 passes were observed.
- *Site 4.* At this site, 112 data intervals were developed northbound and 210 passes were observed. In the southbound direction, 96 data intervals were recorded along with 318 observed passes.
- *Site 5.* At this site, 175 data intervals were produced in the northbound direction and 578 passes were observed. In the southbound direction, 101 data intervals were recorded along with 730 observed passes.

In summary, the three internal sites (2, 3, and 4) produced 421 data intervals in the northbound direction, recording major direction flows, speeds, and observations of passing maneuvers for 970 completed or aborted passes. In the southbound direction, 276 data intervals were recorded at the internal sites, with 1,026 observations of completed and aborted passes. At the two external control sites (Sites 1 and 5), 310 data intervals were recorded in the northbound direction, with 969 observed passing maneuvers. In the southbound direction, 175 data intervals were recorded at the external sites, with 1,577 completed or aborted passes observed.

Table 1 presents an overall summary of the percentage of passes, cross-classified by the risk taking in the passing maneuver in the northbound and southbound directions. The data are based on 1,182 data intervals (obtained in the three weekend data collection periods) and on 4,153 observed passing maneuvers.

A comparison of the data gathered in this study with other research indicated conformance in the following:

- The overall number of passes with no opposition was recorded as 70 percent. The actual site conditions included variations from 50 to 81 percent, which may have been caused by the individual characteristics of the sites and the traffic flows experienced at the time of observation. Because previous passing research has found that 60 percent of all passes occur with no opposition in sight, it can be concluded that the passing data conform with previous results (*4*).
- The time in the opposing lane was recorded for each passing maneuver and averaged 10.6 sec over the 4,000+ passing observations at an average operating speed of approximately 59 mph. This finding conforms with AASHTO *t*2 results of 10.7 sec at 60-mph operating conditions.

Tables 2 and 3 present a summary of the data recorded at each site by direction in 15-min data intervals. The northbound data intervals included both Friday and Saturday peak and off-peak data records, with an average of approximately 150 to 180 data intervals at each site. The southbound data intervals included more highly peaked 15-min intervals recorded only on Sundays (or Monday, July 4), with an average of approximately 75 to 100 data intervals at each site. In general, the northbound data collected from each site can be compared directly with data from other northbound sites; however, caution must be used when comparing northbound and southbound data summaries because the quantity of off-peak data intervals in the northbound direction may tend to depress the mean values.

TABLE 1 RISK TAKING IN PASSING MANEUVER (PERCENTAGE OF ALL PASSES BY DIRECTION)

	NO OPPOSITION		TIME TO OPPOSING VEHICLE IN PASSING: >10 sec.		5<10 sec.		< 5 sec.		PASS ABORTS	
	NB	SB	NB	SB	NB	SB	NB	SB	NB	SB
SITE 1	76	50	6.1	17.8	9.2	12.9	7.7	11.4	1.3	7.9
SITE 2	81	76	4.0	0.4	6.2	9.9	8.1	7.0	1.1	6.6
SITE 3	63	69	14.1	11.6	10.6	11.6	9.8	6.9	2.0	1.1
SITE 4	64	81	26.2	13.2	5.7	1.6	1.0	2.8	3.3	1.6
SITE 5	65	75	19.4	7.3	9.5	9.7	5.4	7.3	0.7	0.8
AVERAGE	70	70	14.0	10.1	8.2	9.1	6.4	7.1	1.7	3.6

A comparison of the northbound average flow rates suggests that Sites 1 through 4 are not significantly different from one another, but the northbound average flow levels at Site 5 are significantly lower than those at Sites 1 through 4. This suggests that the northbound flows within Sites 1 through 4 develop from the same parent population and are maintained throughout the project, but the northbound flow at Site 5 represents a distinctly lower population. A statistical comparison of the flow levels in the southbound direction suggests the same conclusion; i.e., Sites 1 through 4 have similar flow levels, whereas Site 5 has significantly lower flow levels than Sites 1 through 4.

From the data, three distinct passing relationships were noted, as discussed in the following paragraphs.

Low-Flow-Level-Passing Characteristics

The northbound and southbound data shown in Tables 2 and 3 from Site 5 can be examined to help determine the expected characteristics of passing operations in reduced flow levels and less pressured passing operations than those found in Sites 1 through 4. Although these data compare two differently collected data sets, they do indicate that both flow rates are statistically similar in the major direction of passing operations and slightly different in the minor direction. A summation of the last three passing types from Table 1 is 15.6 and 17.8 percent, respectively, for the northbound and southbound directions. This result indicates the stability of the first three passing characteristics as a whole at this site. It also indicates that, if the major direction flow remained constant and more vehicles were added to the opposing flow at Site 5, the first two passing characteristics might be the only ones significantly altered because sufficient major direction volume may not exist to create a severe passing hazard (such as more pass aborts).

TABLE 2 NORTHBOUND PASSING DATA SUMMARY

VARIABLE	15 MINUTE INTERVAL MEAN INDIVIDUAL SITES				
	1	2	3	4	5
NORTHBOUND FLOW (veh/15-minutes)	71.70	74.80	77.72	81.53	54.82
SOUTHBOUND FLOW (veh/15-minutes)	50.70	25.91	28.07	29.31	38.97
TOTAL FLOW (veh/15-minutes)	121.20	107.20	112.60	110.84	93.79
TOTAL NUMBER OF PASSES (15 minutes NB)	3.00	1.81	3.33	2.04	3.27
t2 (average time in opposing lane)	10.45	10.93	9.40	7.98	11.63
PASS WITH NO OPPOSITION	2.26	1.47	2.10	1.34	2.14
PASS WITH OPPOSITION > 10 SEC.	0.18	0.07	0.47	0.55	0.64
PASS WITH OPPOSITION 5 - 10 SEC.	0.27	0.11	0.35	0.12	0.31
PASS WITH OPPOSITION < 5 SEC.	0.23	0.14	0.32	0.02	0.17
PASS ABORTS	0.04	0.02	0.07	0.07	0.02
MULTIPLE PASSES	0.66	0.28	0.56	0.13	0.63

Given this apparent stability in the sum of the most critical of passing events at the Site 5 flow rates and directional splits, the directional passing characteristics at Site 5 can be averaged to present a picture of low-flow-level recreational weekend passing operations as follows:

Where the major directional flow is in the range of 200 to 250 veh/hr, with the minor flow in the range of 85 to 175 veh/hr,

1. Approximately 21 passes occur each hour.
2. The time spent in the opposing lane passing the lead vehicle is approximately 12.2 sec at an operating speed of approximately 60 mph.
3. Passes aborted after the passing vehicle fully enters the opposing lane are approximately 0.75 percent of all passing operations.
4. Passes in which the pass completion is less than 5 sec from the opposing vehicle are approximately 6.3 percent of all passes.
5. Passes in which the pass completion is approximately 5 to 10 sec from the opposing vehicle are approximately 9.6 percent of all passes.
6. Although the extent of passing with no opposition in sight or with opposition greater than 10 sec from pass completion appears to be a function of the flow rate in each direction of travel, the average percent of passes completed in the presence of no opposition is 65.1 to 74.9 percent.

TABLE 3 SOUTHBOUND PASSING DATA SUMMARY

VARIABLE	15 MINUTE INTERVAL MEAN INDIVIDUAL SITES				
	1	2	3	4	5
SOUTHBOUND FLOW (veh/15-minutes)	92.00	98.40	86.06	88.41	56.10
NORTHBOUND FLOW (veh/15-minutes)	41.80	19.40	25.17	22.73	23.22
TOTAL FLOW (veh/15-minutes)	133.80	117.80	112.48	111.14	79.32
TOTAL NUMBER OF PASSES (15 minutes NB)	5.55	2.39	4.86	3.23	7.17
t2 (average time in opposing lane)	12.00	10.71	11.45	10.06	12.87
PASS WITH NO OPPOSITION	3.08	1.94	3.38	2.65	5.42
PASS WITH OPPOSITION > 10 SEC.	1.10	0.01	0.57	0.43	0.52
PASS WITH OPPOSITION 5 - 10 SEC.	0.80	0.25	0.57	0.05	0.70
PASS WITH OPPOSITION < 5 SEC.	0.70	0.18	0.34	0.09	0.52
PASS ABORTS	0.49	0.17	0.05	0.05	0.06
MULTIPLE PASSES	0.93	0.39	1.00	0.23	1.42

These conclusions represent the conditions under which passing operations occur on an apparently typical summer recreational weekend within the volume levels observed.

High-Flow-Level Passing Characteristics

It is also useful to examine the operational and passing characteristics of the higher volume sites (Sites 1 through 4). A comparison of sites in Table 3 shows that the best indicator of the effect of the inability to pass in the presence of higher flow levels occurs at Sites 2 and 1 southbound. Although the volume levels in the opposing direction at these two sites are different, their passing percentages can be averaged to present a picture of high-flow-level recreational weekend passing operations as follows:

Where the major directional flow is in the range of 330 to 420 veh/hr, with the minor flow in the range of 70 to 170 veh/hr,

1. Approximately 10 to 22 passing operations will occur each hour.
2. The time spent in the opposing lane passing the lead vehicle is approximately 11.3 sec at an operating speed of 60 mph.
3. Passes aborted after the passing vehicle fully enters the opposing lane are approximately 7.2 percent of all passing operations.
4. Passes in which the pass completion is less than 5 sec from the opposing vehicle are approximately 9.2 percent of all passes.
5. Passes in which the pass completion is 5 to 10 sec from the opposing vehicle are approximately 11.4 percent of all passing operations.
6. Although the extent of passing with no opposition in sight or with opposition greater than 10 sec from pass completion appears to be a function of the flow rate in each direction of travel, the average percent of passes completed in the presence of no opposition is 50.0 to 75.7 percent.

TABLE 4 MULTIPLE-PASS PERCENTAGE OF TOTAL PASSES

	NORTHBOUND(%)	SOUTHBOUND(%)	TOTAL(%)
SITE 1	21.8	16.8	19.3
SITE 2	15.2	16.3	15.7
SITE 3	16.8	20.6	18.6
SITE 4	6.4	7.0	6.8
SITE 5	19.3	19.8	19.6
AVERAGE	15.9	16.1	16.0

These conclusions represent the conditions under which passing operations occur on a typical high-peak-volume recreational weekend in the summer. The conclusions, especially with regard to abort passes and hazardous passes, can be expected to exceed those to be found under nonrecreational conditions of a similar volume in which passing is restricted to 33 percent in both directions.

ANALYSIS OF MULTIPLE PASSES

As explained previously, multiple passes are an undesirable by-product of inefficient passing operations in which one vehicle passes two or more vehicles or two or more vehicles pass one or more vehicles. Although the level of risk varies among multiple passes, all of these maneuvers may be a surrogate measure for the pressure to pass exerted in each individual passing zone. Table 4 presents the multiple pass percentage of total passes occurring at each site by direction. Ideally, no multiple passes should occur; however, a range of multiples from 6 to 21 percent indicates substantial passing pressure in the passing zones.

SUMMARY

Although internal spot speeds were not collected at all sites, they were recorded as 58.1 mph northbound and 59.9 mph southbound with floating-car travel times of 12.14 and 11.87 min, respectively. Table 5 presents a summary of the change in passing operations over the lower and higher flow levels. These data indicate that, where flow levels are sufficiently elevated, the passing driver's risk-taking decision threshold may increase by up to 100 percent; in other words, the passing driver is twice as willing to accept risk when the opportunity to pass is restricted to 33 percent of normal operating conditions. In the more typical recreational weekend passing operations, only 25 to 35 percent of all passes are made with an opposing-lane vehicle in sight. However, when the volume levels increase by only another 100 to 125 veh/hr in the major direction, and with the opposing flow approximately the same, passing drivers accept up to 50 percent of passing operations in the presence of an opposing vehicle.

Within this additional risk-taking event, the data also indicate that the passing driver accepts an approximate 2 percent increase in less significant passing conflicts (in which the opposing vehicle is 5 to 10 sec from the pass completion) and a 3 percent increase in more significant pass conflicts (in which the opposing vehicle is less than 5 sec from the pass completion). Most surprising, the incidence of pass aborts increased approximately 7 percent from its original condition of approximately 1 percent of all passes.

Passing drivers appear to be creating a far greater change in the primacy associated with passing aborts than with the less hazardous maneuvers. This suggests that passing drivers may be significantly overestimating their ability to complete passing maneuvers safely, which may be causing the inordinate

TABLE 5 COMPARISON OF PASSING OPERATIONS AT OBSERVED FLOW LEVELS

MAJOR FLOW (vph)	200 - 250	330 - 420
MINOR FLOW (vph)	85 - 175	70 - 170
PASSES/HR	21	16
TIME t2 IN OPPOSING LANE (seconds @ 60 mph)	12.2	11.3
VEHICLE OPPOSITION TO PASS (percent of total passes)	65 - 75	50 - 76
PASS COMPLETION 5-10 SEC. FROM OPPOSITION (percent of total passes)	9.6	11.4
PASS COMPLETION LESS THAN 5 SEC. FROM OPPOSITION (percent of total passes)	6.3	9.2
PASS ABORTS (percent of total passes)	0.8	7.2

increase in passing aborts compared to the less serious conflicts at higher flow levels.

CONCLUSIONS

Over 4,000 normally occurring passing operations were observed in this research, which conformed with both AASHTO and previous research results relating to passing maneuvers. The volumes observed are not significant in terms of the capacity of a two-lane, two-way roadway because the major direction carries only 200 to 420 veh/hr, with a minor direction volume in the range of 70 to 175 veh/hr, or a total two-way maximum flow of only 500 to 550 veh/hr. However, even at these relatively low flow levels, passing operations were observed to be negatively affected by the absence of passing opportunity when contrasted to normal passing zones, and this negative effect increased significantly as the traffic flow levels increased.

A reduced passing opportunity to only 33 percent also appears to have a significant effect on the passing driver's delay tolerance and the relationship with primacy, which presumes that drivers value safety above all other driving goals. The shift in primacy for a variety of passing characteristics (including aborts) from less than 1 percent to 6 and 7 percent of all passes at two individual sites indicates that the passing driver's decision threshold can be significantly altered by the previous inability to pass as volume levels increase and that passing opportunity at flow levels of only 400 to 500 veh/hr appears to be an integral part of the safe operation of two-lane, two-way rural highways.

RECOMMENDATIONS FOR FUTURE RESEARCH

This effort has developed a significant amount of data related to the operation and safety characteristics of rural two-lane, two-way roadways in peak and off-peak flow conditions. Due to the volume of both field-observed and video-recorded data, the following issues deserve more detailed attention:

1. What type of lead vehicle is causing passing to occur?
2. Are peak and off-peak passing operations similar, or do passing characteristics follow an exponential distribution as traffic flows increase such that increased flows add substantially to degraded passing operations and safety?
3. How do the passing data from this study correspond to data generated by the two-lane traffic simulation model ROADSIM?
4. The significance of close-following platoons should be compared to the *Highway Capacity Manual* (HCM) rural capacity procedures in a before-and-after study of the passing lanes to establish a relationship between the HCM and the safety risk-taking effects observed in this study. This relationship, along with a peaking-sensitive economic analysis procedure, may aid in the determination of need for passing lanes on rural two-lane, two-way roadways.

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Field Observations of Truck Operational Characteristics Related to Intersection Sight Distance

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Several pilot field studies were conducted to test a data collection methodology for the evaluation of AASHTO Case III–B and C sight distances for trucks at stop-controlled T intersections. The data collection plan used a combination of three traffic observation techniques: video recorders, human observers, and portable traffic data collectors. Specific findings included estimates for the gaps (time and distance) that trucks on a minor road accept during a turn maneuver onto a two-lane roadway, the average acceleration rate for the turn maneuver of a truck on a minor road, and the average deceleration rate of vehicles on a major road during a truck's turn maneuver from a minor road. The speed reduction by a vehicle on a major road during the truck's turn maneuver and the minimum separation distance between the turning vehicle and an oncoming vehicle were also estimated.

Recent sensitivity analyses have demonstrated that the application of AASHTO's intersection sight distance (ISD) procedures to trucks can result in required sight distances that could exceed 3,000 ft (*1*). Such long sight distances are probably not practical or required by truck drivers. To quantify actual truck performance, pilot field studies were conducted to observe truck operational characteristics at three stop-controlled T intersections. Video data were collected at three intersections where trucks exited from a minor two-lane roadway and turned left or right onto a major two-lane roadway. The data collection methodology successfully established estimates of truck gap acceptance values, truck acceleration rates, vehicle deceleration rates and speed reductions on the major roadway, and a resulting minimum separation between the accelerating truck and an oncoming vehicle on the major roadway. Specific findings were compared to vehicle performance characteristics described in AASHTO's 1984 *A Policy on Geometric Design of Highways and Streets* (Green Book) (*2*) and other related literature.

The field data provided guidance for future efforts. Another study on a larger scale is needed to fully evaluate field performance characteristics such as acceleration, deceleration, and minimum separation. The gap acceptance concept should also be further examined for a broader range of vehicle and driver types, intersection geometrics, approach speeds, and traffic volume on the major and minor roads. A gap acceptance sight distance procedure provides a means to simultaneously consider driver behavior, vehicle performance, and operational characteristics at an unsignalized intersection. Knowledge gained regarding the various distributions of the interrelated parameters would establish an empirical basis from which current ISD criteria could be modified. The resulting ISD procedure would specifically permit an analysis of differently designed vehicles commensurate with the intended functional requirements of the intersection.

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DATA COLLECTION

Intersection Selection

Potential intersections were identified through phone calls or discussions with individuals associated with trucking associations, planning commissions, municipalities, police, and state departments of transportation. Candidate intersections satisfied the following conditions or criteria:

- Unobstructed sight distance is present (goal of 1,000 ft).
- Between 5- and 10-percent truck traffic exists on the major road.
- The minor road is associated with a truck generator or with a high percentage of truck traffic.
- Both the major and minor roads are two-lane roadways meeting as a T intersection (preferably without turn lanes).
- The minor road is controlled by a Stop sign.
- The posted speed limit for the major road is greater than or equal to 40 mph.
- The candidate intersection is a minimum of 1,000 ft from a signalized intersection.
- The candidate intersection is standard with regard to geometry (i.e., its approach roads intersect at an approximately 90-degree angle with relatively flat approach grades).

Candidate intersections identified during the initial contacts were visited. Photographs were taken, sketches were drawn, and geometric and operational information was obtained during the initial visits.

Intersections with acceleration lanes, separate left-turn lanes, apparently low truck traffic on the minor road, or apparently low volume on the major road (large headways) were eliminated. Traffic counts were conducted for a minimum of 24 hr at each candidate location. On the basis of information from the initial site visits and the traffic counts, three intersections in Pennsylvania were selected. One intersection was an asphalt and aggregate plant driveway (Central Valley Asphalt Plant), the second was a truck stop exit (Truck Stop 64), and the third was near an industrial park (Trindle and Railroad). Table 1 presents the characteristics of each intersection.

Data Collection Plan

The primary objective of this pilot study was to develop, test, and recommend an effective data collection approach. A video camera data collection procedure was used at each site. The video equipment recorded the movement of the vehicles at each intersection. This procedure permitted the collection of all data needed for evaluation. Traffic data collectors recorded an estimate of the running speed on the major road and a point speed for the decelerating or accelerating vehicle, as well as the traffic volume during the study. Figure 1 shows the equipment layout for a typical data collection effort.

Data collection began by properly orienting the video cameras. The cameras were positioned to maximize the length of road filmed without jeopardizing the resolution of the vehicles on the film. Typically, one camera recorded the overall operations at the intersection (100 ft on either side of the center of the minor road approach); several other cameras recorded the major roadway approaches. Approximately 300 ft of road was discernible from each approach leg camera.

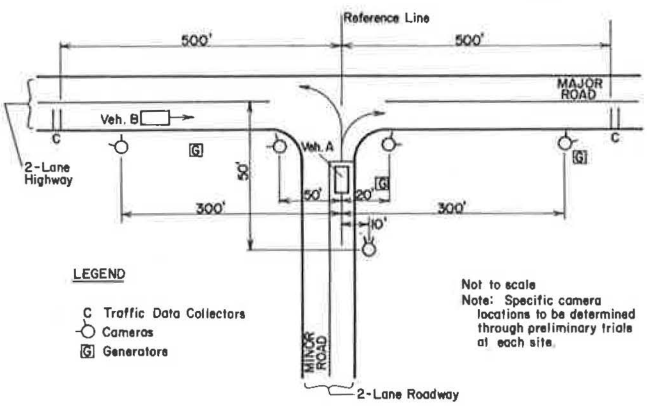

FIGURE 1 Typical setup for video data collection plan.

An internal clock was started when videotaping began. The cameras superimposed the time on the video recording so that the times of specific events could be identified during the data reduction process. Each internal clock was synchronized with a master clock. Researchers used a lap-top portable computer with continuous time display to coordinate the counters and video cameras.

To determine the position of a vehicle at a given time, reference points were documented on the videotape. This was accomplished by flagging vehicles as they passed selected points in the study area. The flagging procedure began at the center of the minor road approach (the reference line shown in Figure 1). Points were then painted in 100-ft increments along the major road up to a distance of 600 ft from the reference line in each direction. The flagging process established the reference scale for eventual data reduction.

Two traffic data collectors (Diamond Traffic Products, Model TT-2001) recorded major roadway approach or departure speeds for individual vehicles. The collectors were generally 500 ft from the intersection on the near-lane legs with tubes spaced 10 ft apart to measure speed, axle classification, and volume. Because the collectors could display and store individual vehicle speeds, manual recording of the speed of each approaching or departing vehicle was not required.

DATA REDUCTION

The data reduction was accomplished by drawing 100-ft increment lines on a clear sheet of acetate taped to a television monitor. The process began by reviewing the videotape frame by frame to determine the exact location of a vehicle crossing a 100-ft increment line.

The videotape of the minor road approach was commonly referred to as the "reference line tape" and was used in every data reduction. Data reduction, although relatively simple, was time consuming and tedious. To reduce the required information (gap, acceleration, deceleration, and minimum separation), several different videotapes were viewed simultaneously. The video equipment had the following capabilities: slow motion, freeze frame, and frame-by-frame advancement.

TABLE 1 SELECTED INTERSECTION CHARACTERISTICS

Intersection			Volume[a]		% Trucks[a]		Speed Limit[b]	85th Percentile Speed	Descriptive Profile	
No.	Major Road	Minor Road	Major	Minor	Major	Minor	(mi/h)	(mi/h)	Major	Minor
1	RT 26	Central Valley Asphalt Plant	14,000	175	15	90	45	47	level	level
2	RT 64	Truck Stop 64	7,000	500	20	95	40	51	level	level
3	Trindle	Railroad	20,000	2,000	20	25	40	40	level	level

[a]Values are unadjusted ADT count volumes obtained in September 1988 during site selection.

[b]Major roadway approach.

Gap Acceptance

The reference line tape was used to reduce the gap information. Data were eliminated if the vehicle on the minor road did not stop, a turning vehicle caused the gap, or the gap-causing vehicle turned onto the major road from a nearby driveway. To obtain the gap data, a record was made of the time the truck on the minor road stopped, the times the subsequent vehicles on the major road crossed the reference line until the departure of the truck, the departure time of the truck, and the time the next vehicle on the major road crossed the reference line.

Acceleration

The equipment setup required for acceleration was more complex than the gap setup. For most sites, three tapes were reviewed simultaneously to obtain complete acceleration information for a particular truck. Acetate sheets were marked in 100-ft increments using the flagging procedure. The monitors were used to follow the truck from screen to screen as the turn maneuver was accomplished. The departure time and the times at which a truck passed the 100-ft increment lines were read from the video screen and recorded in a computer worksheet.

Data for vehicles were eliminated for the following reasons: the vehicle did not stop completely at the intersection, certain 100-ft data points were not discernible, the vehicle slowed to make a turn within the study area, or other factors were present that would bias the findings.

Deceleration

Vehicles on the minor road identified from the gap data that did not stop at the intersection were eliminated from the potential deceleration data set. Deceleration data for vehicles on the major road reacting to a turning truck that accepted a gap greater than 20 sec were also eliminated. (Initial field observations indicated that the vehicle on the major roadway did not decelerate during these truck turning maneuvers.) The potential deceleration data were then divided into the following groups:

- Vehicles in the near lane of the major road responding to right-turning trucks,
- Vehicles in the near lane of the major road responding to left-turning trucks, and
- Vehicles in the far lane of the major road responding to left-turning trucks.

Minimum Separation

Minimum separation is the distance between the rear bumper of the turning truck and the front bumper of a vehicle approaching on the major roadway. Minimum separation can be determined by comparing the acceleration data for the truck on the minor road with the deceleration data for the vehicle approaching on the major roadway. The minimum time (or distance) difference between estimated acceleration and deceleration curves was eventually determined from a plot of the data. A sample of data sets that included both acceleration and deceleration data was selected for the minimum separation evaluation.

ANALYSIS

Gap Acceptance

The quantity of the proposed data to be collected for the gap acceptance analysis was a compromise between a reasonable, realistic data collection effort for a pilot study and the need for adequate data for numerical analysis. Several combinations of vehicle and maneuver types at the intersections had fewer than 50 data points, which was the goal established at the beginning of the data collection efforts. Analysis was conducted only for combinations of at least 15 truck turning maneuvers from the minor road.

The acceptance and rejection data for two possible maneuvers (left or right turns) and two vehicle types (five-axle trucks and trucks with fewer than five axles) were determined for each day. Field observations indicated that the maneuvers made during the filming were typical; for example, no accidents or other unusual situations occurred during the filming period. The individual numerical data files for each day were later merged into three intersection site files.

Several difficulties and biases arose in the measurement of the critical gap. For example, the actual critical gap of an observed single driver cannot be measured. Such difficulties have resulted in the development of different methods for identifying a critical gap. Three methods were initially selected for this study: the Greenshield and Raff methods and the logit model (*3–5*). The logit model was used for the comparison analysis, because it produced descriptive results. Certain results from the Greenshield analyses must be interpreted with caution because of small sample sizes. One small accepted gap length can determine the average minimum acceptable time gap if none or only one of the drivers rejected the same gap size. The Raff method produced results similar to the Logit model.

Choice modeling (such as whether to accept a gap) may be done by using a logistic model to estimate the probability of taking a certain action. Logistic or logit models have been used in previous studies to model the probability of accepting gaps of varying lengths (*6,7*). The simple, dichotomous choice logistic function is

$$P = \frac{1}{1 + e^{-(\beta_0 + \beta_1 X)}} \tag{1}$$

where

P = probability of accepting a gap,
β_0, β_1 = regression coefficients, and
X = variable related to the gap acceptance decision (gap length).

The mean response is a probability when the dependent variable is a 0 or 1 (accept or reject) indicator variable. The logistic function can be easily linearized with the following transformation:

$$P' = \log_e\left(\frac{P}{1 - P}\right) = \beta_0 + \beta_1 X \quad (2)$$

where P' equals the transformed probability.

A sample logistic curve and equation for five-axle trucks turning right at the Trindle and Railroad intersection are shown in Figure 2. The probability of accepting a gap is found by solving Equation 2 for a particular time value. The time gap for a 50-percent probability can be found by substituting 0.5 for P as follows:

$$\log_e\left(\frac{0.5}{1 - 0.5}\right) = -9.58 + 1.12 \times X_{50\%} \quad (3)$$

where $X_{50\%} = 8.52$ sec. Fifty percent of the truck drivers at the Trindle and Railroad intersection accepted a gap of 8.52 sec, and 85 percent accepted a gap of 10.06 sec. The logit model was similarly applied to the remaining combinations of vehicle and maneuver types. Table 2 presents the results from the preceding analysis.

Each intersection's unique characteristics influenced the truck driver's gap acceptance. Right-turning trucks at the Central Valley Asphalt intersection waited for the passing of platoons formed at a signalized intersection 2,000 ft upstream. Truck drivers at the Truck Stop 64 intersection frequently waited for large gaps (greater than 20 sec) that were readily available because of the low volume on the major road. Also, these drivers may have accepted larger gaps than usual, because the majority of the drivers would only accelerate for a short distance before slowing to make a turn onto Interstate entrance ramps 500 and 1,000 ft downstream. Truck drivers at the Trindle and Railroad intersection were pressured to accept smaller gaps than those accepted at the other sites. The frequency of gaps greater than 20 sec was small; long queues occasionally formed on the minor road behind the truck.

Findings from similar studies (*6,8*) for trucks and passenger cars and findings from the *Highway Capacity Manual* (*9*) for passenger cars are presented in Table 3. In 1981, Wennell and Cooper (*8*) reported on their studies conducted at five

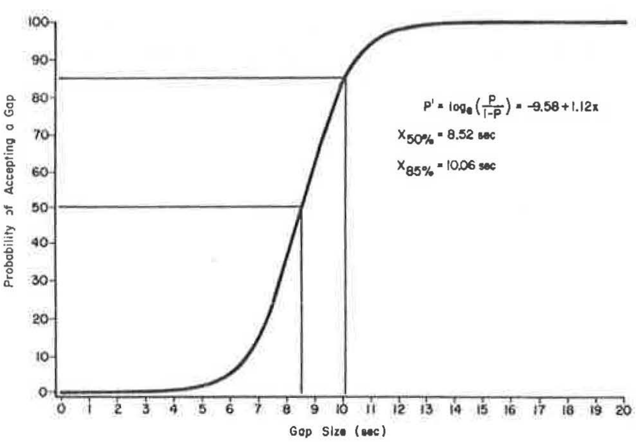

FIGURE 2 Sample logit model plot for five-axle trucks turning right at Trindle and Railroad.

intersections in England. Data for vans and trucks were combined into an all-goods category when the data were sparse. The all-goods vehicle findings are several seconds shorter than the findings in this study. Gap lengths for passenger cars are more than 1 sec shorter than the critical gaps found in other studies and the *Highway Capacity Manual* (*9*).

Radwan et al. (*6*) conducted field studies to estimate gap acceptance distribution for drivers crossing or merging from a minor road onto a four-lane major road. Truck data for all maneuvers (right, through, and left) were combined, because the number of data points was small. The findings at the high-volume Trindle and Railroad intersection were similar to Radwan's findings for all truck maneuvers.

Distance Gaps

Distance gap is the distance between one vehicle on the major road and the next, as the first vehicle passes the intersection (see Figure 3). The distance gap was determined at the Trindle and Railroad intersection for situations in which the vehicle was within the camera limits and was not eliminated for some other reason (such as decelerating to make a turn or entering the major road from a driveway within the study area). The distance gaps accepted, approximated to the nearest 25 ft, are presented in Table 4. Determining the distance gaps required several cameras and was only possible when the vehicle on the major road was within 500 ft of the intersection. Only 22 percent of the vehicles on the major road at the Trindle and Railroad intersection were within these camera limits.

The distance gap accepted by a vehicle on the minor road is a measure preferable to the time gap. Distance gaps are

TABLE 2 FINDINGS FROM GAP ACCEPTANCE ANALYSIS FOR TRUCKS

		Logit model at the following percent probability of accepting a gap	
Intersection	Data Sets	50 Percent (sec)	85 Percent (sec)
LEFT-TURNING 5-AXLE TRUCKS			
Central Valley Asphalt	1	--*	--
Truck Stop 64	5	--	--
Trindle and Railroad	16	8.27	9.84
RIGHT-TURNING 5-AXLE TRUCKS			
Central Valley Asphalt	0	--	--
Truck Stop 64	134	12.43	14.78
Trindle and Railroad	91	8.52	10.06
LEFT-TURNING LESS-THAN-5-AXLE TRUCKS			
Central Valley Asphalt	58	11.16	13.89
Truck Stop 64	2	--	--
Trindle and Railroad	8	--	--
RIGHT-TURNING LESS-THAN-5-AXLE TRUCKS			
Central Valley Asphalt	23	13.17	15.86
Truck Stop 64	7	--	--
Trindle and Railroad	26	7.25	8.87

*Insufficient data. Analyses were not performed for data sets containing less than 15 accepted gaps (i.e., 15 minor road vehicles).

TABLE 3 FINDINGS FROM SIMILAR STUDIES

Study	Turn Maneuvers	Gap (sec)		
		Median Accepted Gap[a]		
Wennell and Cooper, 1981 *(8)*	Left turns (UK conditions)	Site	Cars	Goods[b]
		1	3.91	4.63
		3	3.66	5.33
		4	4.31	4.99
		5	4.41	4.91
		Critical Gap[c]		
Radwan, et al., 1980 multilane, divided highways *(6)*	PC, Right turns	6.73 sec		
	Trucks, all possible maneuvers	8.40 sec		
Highway Capacity Manual, 1985 *(9)*		Running Speed (Major)		
		30 mi/h		55 mi/h
	Right turn from stop, 2 lanes	5.5 sec		6.5 sec
	Left turn from stop, 2 lanes	6.5 sec		8.0 sec

[a]Gap that has a 50 percent probability of acceptance (probit analysis).

[b]Goods category included vans and trucks when data were "sparse".

[c]Critical gap was determined using Logit model.

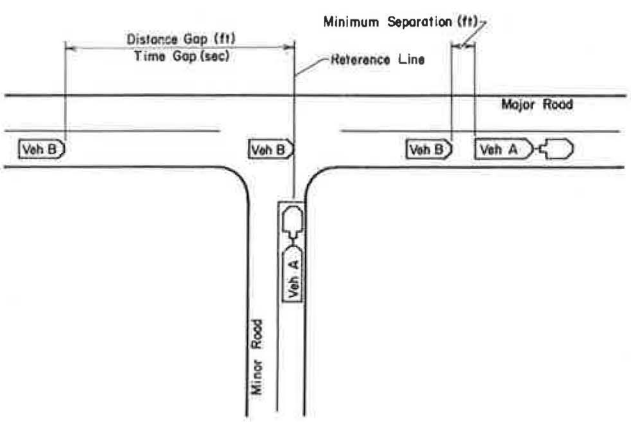

FIGURE 3 Gap and minimum separation dimensions.

directly comparable to sight distance, and time gaps can vary if a gap is accepted and the vehicle on the major road is forced to decelerate. Unfortunately, measuring distance gaps is much more difficult than observing time gaps.

Sight distances based on critical time gaps and 85th-percentile speed result in calculated sight distance values greater than the field-observed distance gap. For example, if the critical gap is 8.5 sec and the major roadway speed is 40 mph, the calculated sight distance is approximately 500 ft. A predicted distance gap from Table 4 for an 8.5-sec accepted time gap would be between 300 and 400 ft.

Acceleration

The times at which an accelerating truck left the intersection and arrived at each 100-ft increment line were read from the clock superimposed on the videotapes. These times were recorded in a computer spreadsheet program in hours, minutes, and seconds. In order to analyze the vehicles' distance-versus-time curves, the raw data were standardized so that all vehicles left the minor road at time zero.

TABLE 4 DISTANCE GAPS FOR TRUCKS AT TRINDLE AND RAILROAD

Vehicle No.	No. Axles	Time Gap (sec)	Distance Gap (ft)
RIGHT-TURNING TRUCKS, GAPS TO THE LEFT			
E 50	2	6.31	200
F 44	5	8.64	200
F 48	5	8.96	200
F 46	5	9.05	200
F 98	3	9.56	200
E 66	2	6.14	300
E 46	5	6.97	300
E 69	5	7.44	300
F 21	5	8.91	300
E 10	5	7.01	350
F 47	5	8.64	350
E 13	5	8.11	375
E 8	5	11.58	400
E 9	2	6.36	500
F 52	3	7.24	500
E 32	2	7.84	500
F 64	5	8.68	500
E 65	5	10.34	500
RIGHT-TURNING TRUCKS, GAPS TO THE RIGHT			
F 16	4	8.74	375
F 57	5	9.48	400
F 58	5	11.71	400
F 74	5	7.95	400
F 84	5	11.88	400
F 87	5	8.35	300
F 97	5	7.17	300
LEFT-TURNING TRUCKS, GAPS TO THE LEFT			
E 56	5	8.34	300
LEFT-TURNING TRUCKS, GAPS TO THE RIGHT			
E 67	2	11.55	300
F 43	5	11.67	400
F 60	5	12.13	400

Average accelerations were calculated using average velocities and average time required to traverse a given distance. Because the data were based on 100-ft increments, average speed was calculated for each 100-ft segment. Table 5 presents a summary of the average acceleration findings.

Presented in Table 6 are average acceleration rates determined from distance-versus-time plots reported by Hutton *(10)*. Hutton measured the acceleration of trucks with weight-to-horsepower ratios of 100, 200, 300, and 400 lb/hp on a level and straight roadway. Table 6 also includes average acceleration rates calculated using Figure IX–22 in the Green Book *(2)*. Distance and speed were taken directly from the Green Book *(2)* figure. The time required to reach a given speed or distance was calculated in 5-mph increments. The acceleration rate for a specific distance was calculated by dividing speed attained by the time required to reach the given speed.

The signalized intersection south of the Central Valley Asphalt exit did not appear to affect the acceleration behavior of the turning trucks. The calculated average acceleration rates were within the rates calculated from the Hutton curves. The median acceleration rate for trucks turning left was between Hutton's 100- and 200-lb/hp values (*10*). The median acceleration rate for right turns was near the 300-lb/hp values.

The acceleration rates at the Truck Stop 64 intersection were lower than at the other sites, because the majority of the trucks turning right eventually turned onto nearby Interstate entrance ramps. The median acceleration rate was significantly lower than those calculated at other sites. This rate was also lower than the 400-lb/hp values presented by Hutton (*10*), but it was similar to the rate calculated from AASHTO (which represents an older truck fleet with higher weight/horsepower ratios).

The median acceleration rate at the Trindle and Railroad intersection was near the value for the 100-lb/hp ratio in Table 6 for the 0- to 490-ft distances. The urbanized setting and high traffic volume on the major road influenced the acceleration rates at this site.

The rates calculated using the Green Book (*2*), Figure IX–22, were considerably lower than those of the vehicle with the poorest performance (400 lb/hp) examined by Hutton (*10*) and most rates calculated for the three study intersections. As presented in Table 6, the AASHTO acceleration rates are relatively constant (approximately 0.77 mph/sec) for the specific distances.

Deceleration

The deceleration data from the different cameras were also adjusted to a common time base. The data files from different filming days were similarly merged into three intersection files. These files were segregated by maneuver type (right or left turns). Because of the limited number of data points, the data associated with five-axle right-turning trucks at Truck Stop 64 and Trindle and Railroad intersections were combined into one data set. The number of data sets for left-turning trucks was small; therefore, only a limited analysis could be conducted.

An average speed was calculated for each 100-ft increment. These average speeds were then examined to identify where a maximum deceleration rate or speed reduction occurred. Vehicles were not considered in the analysis if they had less than a 5-mph speed reduction through the observation area or if the data displayed erratic or extreme speed variations.

The 50th and 85th percentiles for the deceleration rate and speed reduction occurring before the intersection were determined for vehicles on the major road reacting to turning five-axle trucks. These values typically represented a 200- to 400-ft total deceleration distance ending 50 to 150 ft before the intersection. Fifty percent of the vehicles impeded by five-axle trucks turning onto the major road had deceleration rates of 3.67 mph/sec or less. Eighty-five percent of the vehicles had deceleration rates of 5.85 mph/sec or less. Fifty percent of the vehicles had speed reductions of approximately 21.2 mph when impeded by five-axle trucks turning onto the road. Eighty-five percent of the vehicles had speed reductions of 38.1 mph or less.

Table 7 presents the speed reduction for each vehicle grouped by initial speed. The estimated speed reduction for each 5-mph rounded initial speed increment is also presented. The

TABLE 5 CALCULATED AVERAGE ACCELERATION RATES

Intersection	Maneuver	No. Axles	Distance (ft)	No. of Vehicles	Average Acceleration Rate (mi/h/sec) Cumulative Probability 50 Percent	85 Percent
Central Valley Asphalt	Left turn	3 & 4	0-290	25	1.27	1.58
Central Valley Asphalt	Right turn	3 & 4	0-490	8	1.04	1.21
Truck Stop 64	Right turn	5	0-350	43	0.80	1.20
Trindle and Railroad	Right turn	5	0-510	41	1.37	1.74

TABLE 6 HUTTON AND AASHTO ACCELERATION RATES (mph/sec) (*2,10*)

Distance (ft)	Hutton 100 lb/hp	200 lb/hp	300 lb/hp	400 lb/hp	AASHTO[a]
0-290	1.57	1.14	1.03	1.01	0.75
0-350	1.53	1.13	1.03	1.01	0.76
0-490	1.38	1.11	1.01	0.90	0.78
0-510	--[b]	--	--	--	0.79

[a]Values based on Green Book Figure IX-22.

[b]Data not available, curves terminate at 500 ft.

TABLE 7 SPEED REDUCTIONS FOR VEHICLES ON A MAJOR ROAD REACTING TO A FIVE-AXLE TRUCK TURNING RIGHT

Vehicle ID No.	Gap Accepted (sec)	Initial Speed (mi/h)	Speed Reductions (mi/h)	Deceleration Rate (mi/h/sec)	Rounded Initial Speed (mi/h)	Estimated Speed Reductions (mi/h)
76 F	10.78	68	40.4	7.83	70	40
116 C	18.95	64	43.2	4.46	65	35
147 C	18.45	63	29.7	6.56		
99 C	13.12	62	40.7	5.85	60	35
47 F	8.64	62	38.5	7.74		
261 C	14.65	60	36.5	3.68		
49 F	12.81	57	27.9	5.10	55	30
46 F	9.05	52	38.1	4.84	50	25
78 F	10.55	52	26.9	6.75		
84 F	11.88	52	26.1	4.37		
130 C	16.92	52	5.4	0.74		
97 F	7.17	51	17.7	2.79		
48 F	8.96	49	38.1	4.46		
57 F	9.48	49	25.1	3.85		
18 F	14.85	49	20.9	3.85		
41 F	15.90	48	21.4	3.66		
176 C	19.35	47	24.6	2.30		
17 F	13.22	46	23.0	3.56	45	20
117 C	18.55	46	17.9	2.57		
232 C	15.95	46	9.1	1.72		
201 C	14.75	45	21.5	2.25		
27 F	12.56	45	19.6	3.11		
65 E	10.34	45	18.6	4.09		
24 E	19.45	43	20.1	2.98	40	15
209 C	12.97	43	16.4	1.69		
28 E	10.81	43	15.3	4.09		
25 F	9.88	41	20.5	4.07		
30 E	15.35	41	11.9	3.23		
22 E	13.74	41	11.8	3.16		
36 F	12.98	40	14.2	2.88		
38 E	14.14	37	9.9	2.43	35	15
48 E	12.27	36	6.4	1.65		
49 E	15.61	35	19.9	3.43		

speed reductions ranged from 40 mph at a 70-mph initial speed to 15 mph at initial speeds of 40 and 35 mph.

Only limited data were available for left turns. The findings for each vehicle are presented in Table 8. A review of the findings did not reveal any differences in speed reductions or deceleration rates for vehicles in the far lane as compared with vehicles in the near lane during a left-turn maneuver. More data would be required to draw conclusions on whether drivers in the opposing lane respond differently to left-turning vehicles.

Green Book (*2*), Figure II–13, presents deceleration distances for passenger vehicles approaching intersections. The distances, which are based on comfortable deceleration rates, are determined from the speed when brakes are applied and the final speed reached. Curves are provided for the following final speeds: 50, 40, 30, 20, and 0 mph. Table 9 presents the deceleration rates based on the Green Book (*2*) figure.

Table 10 presents the observed normal deceleration rates for passenger cars on dry pavement from the *Transportation and Traffic Engineering Handbook* (*11*). The Handbook states that deceleration rates up to 5.5 mph/sec are reasonably comfortable for passenger car occupants.

The majority of the deceleration rates observed in the field are within the comfortable rates from both the Green Book (*2*) and the Handbook (*11*). Vehicles with deceleration rates greater than the Green Book (*2*) rates had initial speeds higher than 62 mph. These high-speed vehicles had to reduce their speed between 25 and 41 mph when a truck entered the traffic stream. With a 25-mph reduction, the vehicles were driving near the speed limit of the road. The speed limit and 85th-percentile speed were 40 and 51 mph at Truck Stop 64 and 40 and 40 mph at Trindle and Railroad, respectively.

Minimum Separation

The minimum separation analysis required information on both the accelerating truck and the decelerating major road vehicle. These data sets were compared to determine the number of potential minimum separation data sets available. Nine sets from the Truck Stop 64 intersection and eight sets from the Trindle and Railroad intersection were available. Two sets from each intersection were eliminated when the minimum separation location was found to be outside the camera limits. Only right-turning vehicles were selected for this analysis.

Among the various parameters estimated, the information on minimum separation was the most limited. Nonetheless, an attempt at establishing a probable range of values was made.

All time adjustments made to the acceleration and deceleration data sets (discussed in previous sections) applied to the minimum separation data sets. A plot of speed versus distance was used to estimate the location at which the vehicles were at minimum separation. A sample data set (Vehicle 99) from the Truck Stop 64 intersection is shown in Figure 4.

Minimum separation occurs when both the accelerating truck and the major-road vehicle are traveling at approximately the same speed. If the major-road vehicle is moving at a higher speed than the accelerating truck, minimum separation will occur beyond the camera's field of view. The sample major-road vehicle in Figure 4 reached its minimum speed approximately 250 ft beyond the intersection. Its speed was 11 mph, and the acceleration truck's speed was estimated at 10 mph. The headway time (t) between the vehicles was determined by finding the difference between the two vehicles' arrival times at a point 250 ft beyond the intersection. The headway

TABLE 8 DECELERATION RATES AND MAXIMUM SPEED REDUCTIONS FOR LEFT TURNS

Vehicle No.	No. Axles	Gap Accepted (sec)	Vehicle Type	Speed Reduction (mi/h)	Deceleration Rate (mi/h/sec)
TRINDLE AND RAILROAD, EASTBOUND					
36 E	3	7.78	PC	18.2	6.33
45 F	3	11.21	PC	6.3	1.94
53 F	5	9.03	PC	15.8	2.79
55 F	5	12.32	PC	18.1	2.86
56 E	5	8.34	PC	29.4	2.80
58 E	5	7.27	PC	12.6	3.25
61 E	3	11.51	PICKUP	15.2	5.37
TRINDLE AND RAILROAD, WESTBOUND					
22 F	5	15.48	PC	13.0	3.83
23 F	5	17.75	PC	15.9	4.56
31 F	5	16.59	PC	12.4	3.15
43 F	5	11.67	PC	10.7	2.18
45 F	3	11.21	PC	18.7	5.83
53 F	5	9.03	PC	13.9	3.28
56 F	5	8.24	PC	27.1	6.24
58 E	5	7.27	PC	27.7	2.53
60 F	5	12.13	PC	12.8	2.76
61 E	3	11.51	PC	21.3	3.46
CENTRAL VALLEY ASPHALT, SOUTHBOUND					
3 D	2	16.35	5-AX	13.6	3.18
7 D	3	11.35	PC	14.0	3.51
27 D	3	11.85	2-AX	26.2	5.47
30 D	2	7.81	PC	20.9	5.20

TABLE 9 DECELERATION RATES (mph/sec) FROM THE GREEN BOOK (*2*)

Initial Speed (mi/h)	Speed Reached (mi/h) 50	40	30	20	0
70	6.08	6.47	6.19	6.30	6.25
60	5.78	5.88	5.67	5.74	5.57
50		5.29	5.11	5.15	5.10
40			4.12	4.64	4.70
30				4.08	3.78
20					3.92

NOTE: Deceleration rates are based on information from Green Book Figure II-13. The rates were calculated with the following equation:

$$\text{deceleration rates} = \frac{V_f^2 - V_i^2}{2 \times \text{distance}}$$

TABLE 10 DECELERATION RATES FROM THE ITE (*11*)

Speed Change (mi/h)	Deceleration rate (mi/h/sec)
15 - 0	5.3
30 - 0	4.6
40 - 30	3.3
50 - 40	3.3
60 - 50	3.3
70 - 60	3.3

NOTE: Rates are observed normal deceleration rates for passenger cars on dry pavements. Deceleration rates up to 5.5 mi/h/sec are reasonably comfortable for car occupants. (*11*) time for the sample vehicle was 0.63 sec. The minimum separation distance can be estimated from the plots or from the following equation:

$$MS = (1.47Vt) - L \qquad (4)$$

where

MS = minimum separation distance (ft),
V = velocity of turning vehicle (mph),
t = headway time between vehicles (sec), and
L = length of turning vehicle (ft).

When the calculated minimum separation distance was less than 25 ft, the minimum separation between vehicles was set at 25 ft. The minimum distance of 25 ft was selected on the basis of observations from the videotapes.

The findings for the data sets are presented in Table 11. Minimum separation between vehicles typically occurred between 200 and 400 ft downstream of the intersection. The speeds of the vehicles at minimum separation were between 10 and 24 mph at the Truck Stop 64 intersection and between 17 and 34 mph at the Trindle and Railroad intersection. These speeds are much lower than the 85th-percentile approach speeds of 51 mph for Truck Stop 64 and 40 mph for Trindle and Railroad.

The headway time between the vehicle on the major road and the turning trucks at the Truck Stop 64 intersection was 2.4 sec or less, corresponding to a minimum separation distance of approximately 25 ft. The drivers on the major road at the urban intersection of Trindle and Railroad maintained larger minimum separations. These drivers typically had 4- to 7-sec headways or 50- to 150-ft minimum separations.

The minimum separation at an urban intersection would be expected to be smaller than the minimum separation at a low-volume rural intersection. However, the values for the Truck Stop 64 intersection were lower than those obtained for the Trindle and Railroad intersection. The truck drivers at the Truck Stop 64 intersection did not accelerate as fast as the truck drivers at the urban intersection, and the running speed on the major road was also higher at the rural intersection. The results of the speed difference were that drivers on the major road closed the gap faster on trucks turning from Truck Stop 64 than on trucks turning at Trindle and Railroad. This observation was supported by the location of minimum separation at the intersections. Several of the minimum separation locations were more than 300 ft downstream from the Trindle and Railroad intersection.

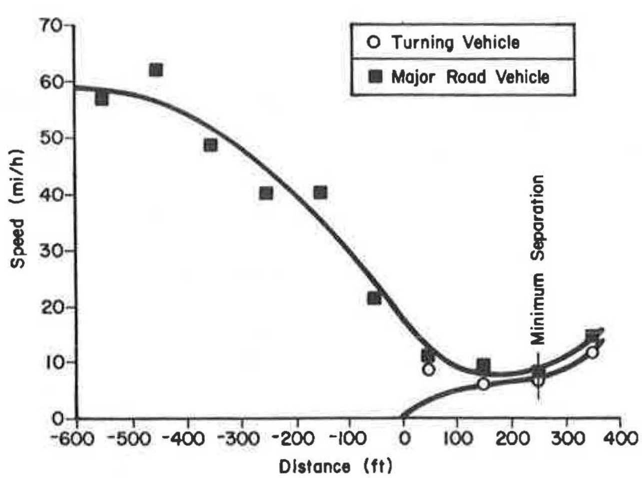

FIGURE 4 Plot of minimum separation (speed versus distance) for a sample vehicle.

The intersection sight distance criteria in the Green Book (*2*) incorporate a dimension known as "tailgate distance," which is equivalent to the minimum separation distance discussed here. However, the Green Book (*2*) does not provide guidance on the values to use for tailgate distance. When the Green Book (*2*), Figure IX-27, B-2a and Ca curve is reproduced using distance and time values approximated from Green Book (*2*), Figure IX-22, the tailgate distance, or minimum separation distance, is approximately 1 sec multiplied by the major-road speed (*12*). This figure represents the distance between the rear bumper of the turning vehicle and the front bumper of the vehicle on the major road. A 1-sec tailgate or minimum separation time represents a 15-ft minimum separation distance of 10 mph and a 30-ft distance at 20 mph.

The general findings from this limited analysis revealed a minimum separation time value of approximately 1 sec for the Truck Stop 64 intersection but higher values at the Trindle and Railroad intersection. Drivers attempted to have a larger separation distance between their vehicle and the turning vehicle if available, but they accepted 1 sec or less on some occasions.

SUMMARY OF FINDINGS

The median gaps accepted by truck drivers turning onto a major road ranged from 7.25 to 13.17 sec, depending on the intersection, turning maneuver, and truck type considered. The range of time gaps accepted with 85 percent probability was 8.87 to 15.86 sec. Table 12 presents a summary of the gaps on the basis of the logit model.

The 50th-percentile average acceleration rates ranged from 0.80 to 1.33 mph/sec, and the 85th-percentile average acceleration rates ranged from 1.20 to 1.74 mph/sec. The specific rates for the predominant truck type for left and right turns at the Central Valley Asphalt intersection and for right turns for the other two intersections are presented in Table 13.

Table 14 presents the deceleration rates and speed reductions for vehicles on the major road at the Trindle and Railroad and Truck Stop 64 intersections that were impeded by right-turning five-axle trucks. The 50th- and 85th-percentile deceleration rates were 3.67 and 5.85 mph/sec, and the speed reductions were 21.2 and 38.1 mph, respectively.

The minimum separation findings, although limited, are presented in Table 15. The headway times ranged from 0.63 to 2.38 sec at the Truck Stop 64 intersection and from 4.13 to 5.24 sec at the Trindle and Railroad intersection. The minimum separation distance between vehicles at the Truck Stop 64 intersection was assumed to be 25 ft, whereas at the Trindle and Railroad intersection the minimum separation ranged from 57 to 143 ft.

These pilot field studies are a first step toward acquisition of the data needed, either to revise the AASHTO procedures to include realistic deceleration by the vehicle on the major road, or to replace the current procedures with an alternative procedure on the basis of a gap acceptance policy.

TABLE 11 MINIMUM SEPARATION FOR FIVE-AXLE TRUCKS TURNING RIGHT

Vehicle No.	No. Axles	Gap Accepted (sec)	Headway Time (sec)	Minimum Separation Distance (ft)	Minimum Separation Time (sec)	Speed at Minimum Separation Major (mph)	Speed at Minimum Separation Minor (mph)	Distance Beyond Intersection (ft)
TRUCK STOP								
94	5	11.18	1.00	25[a]	1.55	12	11	250
99	5	13.12	0.63	25	1.89	10	9	250
116	5	18.95	2.17	25	1.89	10	9	300
130	5	16.92	1.33	25	0.81	20	21	350
147	5	18.45	1.07	25	0.74	24	23	300
176	5	19.35	2.38	25	0.81	20	21	350
209	5	12.97	0.86	25	0.71	24	24	350
TRINDLE AND RAILROAD								
22	5	13.74	5.01	109	3.22	23	23	300
24	5	19.45	4.38	91	2.69	24	23	500
28	5	10.81	4.13	143	2.95	34	33	500
38	5	14.14	4.80	88	2.85	21	21	250
45	5	10.31	4.53	57	2.15	17	18	200
59	5	8.88	5.24	75	3.00	18	17	250

[a]Minimum separation between vehicles was assumed as 25 ft based on observations from the videotapes.

TABLE 12 TIME GAPS ACCEPTED

Intersection	Turn Maneuver	Truck Type	Probability 50 Percent	Probability 85 Percent
Central Valley Asphalt	Left	Less-than-5-axles	11.16 sec	13.89 sec
Central Valley Asphalt	Right	Less-than-5-axles	13.17 sec	15.86 sec
Truck Stop 64	Right	Five-axle	12.43 sec	14.78 sec
Trindle and Railroad	Left	Five-axle	8.27 sec	9.84 sec
Trindle and Railroad	Right	Five-axle	8.52 sec	10.06 sec
Trindle and Railroad	Right	Less-than-5-axles	7.25 sec	8.87 sec

TABLE 13 AVERAGE ACCELERATION RATES

Intersection	Turn Maneuver	Truck Type (No. Axles)	Distance (ft)	Cumulative Probability 50 Percentile	Cumulative Probability 85 Percentile
Central Valley Asphalt	Left	3 & 4	0-290	1.27 mi/h/sec	1.58 mi/h/sec
Central Valley Asphalt	Right	3 & 4	0-490	1.04 mi/h/sec	1.21 mi/h/sec
Truck Stop 64	Right	5	0-350	0.80 mi/h/sec	1.20 mi/h/sec
Trindle and Railroad	Right	5	0-510	1.33 mi/h/sec	1.74 mi/h/sec

TABLE 14 DECELERATION RATES AND SPEED REDUCTIONS FOR MAJOR VEHICLES ON THE MAJOR ROAD REACTING TO RIGHT-TURNING FIVE-AXLE TRUCKS

	Cumulative Probability 50 Percentile	Cumulative Probability 85 Percentile
Deceleration Rates	3.67 mi/h/sec	5.85 mi/h/sec
Speed Reductions	21.2 mi/h	38.1 mi/h

TABLE 15 MINIMUM SEPARATION

Intersection	Headway Time (sec)	Minimum Separation Distance (ft)
Truck Stop 64	1.00	25
	0.63	25
	2.17	25
	1.33	25
	1.07	25
	2.38	25
	0.86	25
Trindle and Railroad	5.01	109
	4.38	91
	4.13	143
	4.80	88
	4.53	57
	5.24	75

SUGGESTED IMPROVEMENTS IN DATA COLLECTION AND REDUCTION METHODS

Future studies should include elevated (and concealed) video cameras. More than five cameras may be necessary for acceleration or distance gap information beyond 500 ft. Reliable and durable video equipment is necessary to minimize later adjustments to timing operations and camera coordination. Each camera's field of view should overlap the next, and each pair of cameras should include a distinguishable common reference point. An alternative approach for marking the 100-ft increment points along the approach legs is to place contrasting colored tape on the road, shoulder, or curb so that reference markings are readily discernible on the videotape. Because video cameras are sensitive to moisture and extreme ambient temperatures, primary field activities must be scheduled to avoid adverse weather conditions.

ACKNOWLEDGMENT

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Comparison of International Practices in the Use of No-Passing Controls

J. ALAN PROUDLOVE

Minimization of no-passing control has assumed greater importance since the publication of the latest Highway Capacity Manual in 1985, so it is important that correct standards be used when making policy for design of alignment or examining the performance of two-way highways. Comparison of some international standards on geometric passing distance requirements revealed widely differing values in published figures. North American and Australian performance in truth differed only slightly, despite different values in their design manuals. Britain, however, used much shorter no-passing distances than the other countries. Differences between countries in their use of no-passing controls were also observed, but the absence of coordination between geometric standards and those for control of passing where those standards were not met was almost universal. The abandonment of current no-passing controls and their replacement by new warrants and application methodology, on the basis of revised geometric standards, is recommended. The resulting effect on lengths and positions of no-passing control zones is expected to be small, but level of service values could be changed. An extension of the present form of no-passing-zone pavement marking is also recommended. Such an extension would include special marking of the approach to the barrier line, in particular to mark the last safe point to begin a passing maneuver, to make the beginning of the barrier line more conspicuous from the point of no return for higher-design-speed roads.

In 1988, the Indonesian government introduced laws supporting the introduction of the double-line, no-passing form of pavement marking controls. Driver behavior in Indonesia was undisciplined; many severe accidents were caused by drivers, particularly of trucks and intercity buses, passing on blind curves. Pavement markings outside cities were almost nonexistent, except on the toll road system being developed.

However, the Indonesian legislation did not contain information that would enable field engineers to determine where double lining should begin and end. Neither were there warrants for the placement of double lines. In researching these matters to prepare advice for the Directorate General of Highways of the Indonesian Department of Public Works, the widely differing practice among English-speaking countries quickly becomes evident, together with the inadequacy and lack of logic of much of that practice. Indonesian practice in geometric design borrowed from AASHTO; Indonesian road signs were an amalgam of the European and Australian standard models. But to simply adopt any other country's warrants without fully understanding the consequences can be dangerous.

The result ot the research was to recommend the adoption of a practice that was closest to that used in Australia, but also using and extending an additional British pavement marking technique. The two essential features of the recommendation were that the start of the double line should be related to the point where minimum passing sight distance is lost (rather than to some arbitrary warrant) and that road users should be advised of the last safe points for starting and aborting a passing maneuver, by the introduction of a new form of pavement marking and perhaps new road signs. This recommendation might be considered for adoption by the United States as a step towards rationalizing current disparate practices. The Indonesian research also revealed an inconsistency in the derivation of the American figures for minimum geometric passing sight distance requirement, suggesting that a term appeared to be missing from the computation.

Correct use of passing sight distance controls is a matter that should be of increased concern to highway and traffic engineers. Introduced several decades ago in North America and Europe, the use of a barrier line prohibiting passing was initially a valuable traffic control device to increase safety on existing two-way roads. As traffic densities increased, the effect of passing restrictions on levels of service became evident, and the proportion of a highway's length without passing opportunity had a major impact on the level of service that could be attained, much more so than other geometric considerations.

Now a third area of significance has emerged in Britain, as part of a coordinated approach to the geometric design of two-way roads that recognized the effects of geometrics on service performance. After several decades in which highway design was dominated by divided highway and particularly limited-access highway design, the need to improve nondivided but high-volume roads came into new prominence, particularly as high design flows became accepted as attainable on this type of road. Central to the new philosophy of design was the minimization of the length of highway with passing restrictions in accord with the principles contained in the 1985 *Highway Capacity Manual* (*1*). These restrictions included those involving either the absence of safe passing visibility or the presence of other prohibitions on passing, such as left-turning protection zones or islands. Three techniques were used to maximize the length of highway with safe passing visibility: minimizing the length of each sight-restricting element by the use of minimum acceptable standards of curvature, combining the horizontal and vertical sight-restricting elements of alignment, and combining the location of alignment and intersection restrictions on passing. These objectives would be supported by the correct sizing and siting of passing restrictions in relation to traffic speeds.

Australia and Britain refer to the maneuver as "overtaking"; the United States and Canada call it "passing." In this

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paper, no distinction has been made between the use of the two terms. Because each country uses two sets of passing standards, one for geometric design and the other for traffic control, the standards will be referred to as either "geometric requirements" or "warrant sight distances," to distinguish one from the other.

CURRENT PRACTICE IN SOME ENGLISH-SPEAKING COUNTRIES

The geometric design manuals (*2–5*) and the traffic control codes (*6–9*) of Australia, Britain, Canada, and the United States revealed differences in values recommended for both geometric passing sight distance minima and sight distance warrants for the use of control devices. In addition, Australia positioned the start of its barrier line differently from the other countries, not at the point where the warrant sight distance is lost. Australia and Britain used a more explicit definition of design speed for geometric design than did either the United States or Canada, although the latter countries shared with the former the 85th-percentile speed definition used for traffic control purposes. In addition, Australia and Britain recognized that overtaking performance followed a statistical distribution; thus, they included consideration of the fraction of the driver population accommodated in their derivation of geometric overtaking sight distance standards.

Meaningful comparisons of standards for minimum safe geometric passing sight distance were difficult to make. For example, values for a 100-km/hr design speed were 1,010 m (Australia), 680 m (Canada and the United States), and 580 m (Britain). Likewise, the warrant distances for use of barrier lines ranged from 185 to 400 m. Each country published its control-warrant standards for determining the need for double-line pavement markings separately from its geometric design standards dealing with the engineering of plan and profile in relation to minimum sight distances. This separation obscured the lack of compatibility between the two sources of advice and may have perpetuated some fundamental errors in the deviation of the recommended values. The differences between countries in the values published, both in warrants for the use of double lines and in the specification of a safe passing sight distance for geometric design, are shown in Figure 1. The reasons for the differences included fundamentals, such as definitions of eye height, object height, and maximum safe rates of deceleration. Britain used a single value (0.25*g*) for the coefficient of deceleration *f*, independent of speed, in contrast to the other countries. Australia used the widest range, with *f* values between 0.33*g* and 0.65*g* for design speeds between 130 and 50 km/hr, respectively, and an eye height of 1.15 m compared with Britain's 1.05 m. More esoteric differences were also involved, such as the meaning of design speed or the definition of minimum safe passing sight distance, or Britain's desirable and absolute minimum standards, and departures below those standards. Britain requires sight distances of 820, 580, and 410 m to allow safe overtaking for 99, 85, and 50 percent of the driver and vehicle population, respectively, for a 100-km/hr design speed. AASHTO (*5*) was unclear about its definition of design speed, discussing "the maximum safe speed that can be maintained" without defining it specifically, such as the 99th-percentile free-running speed.

Australia and Britain both equated design speed to the 85th-percentile speed of highway users. A design speed was individually assigned to each geometric element of alignment as the coordinating value of horizontal and vertical alignment for that particular element. These assignments were placed within an overall speed environment for the highway link that was related to the link length and cross section, physical environment, and highway function. Within this field of disparate definitions and parameter values, double-line warrants also differed considerably.

Table 1 presents sample values of the warrants from the four countries' manuals on traffic control devices. The second half of the table presents corresponding values of minimum passing sight distance requirements from the geometric design manuals of the four countries.

Relationships between the two sets of figures were not obvious although some arbitrary basis had been chosen for the warrants determining the need for double-line markings. The justification for a separate method to determine the need for and positioning of the barrier line, rather than using the position where geometric passing sight distance is lost at the design speed, was not made clear in either the geometric design policies (*2–5*) or the uniform devices manuals (*6–9*) of the different countries, although several made apologetic explanations about the avoidance of unduly restrictive controls on overtaking.

MINIMUM REQUIREMENTS FOR PASSING SIGHT DISTANCE

To be able to understand the different values quoted in the design manuals for the minimum passing sight distance

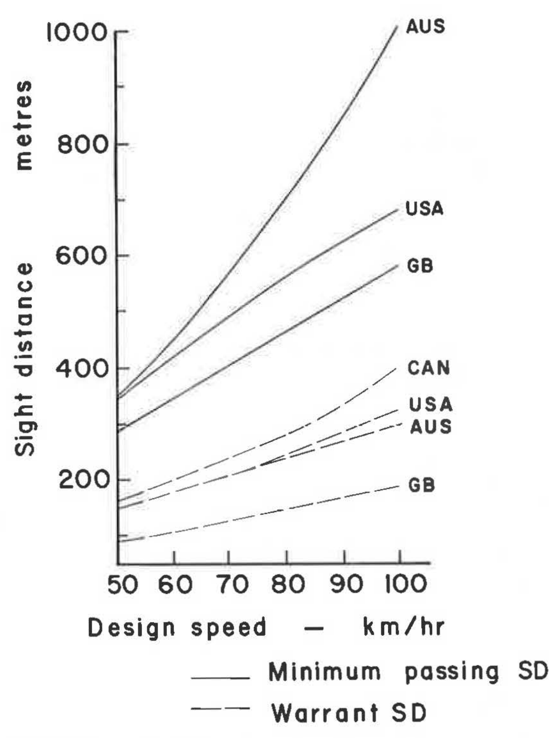

FIGURE 1 Variations in values for minimum passing sight distance, warrants for the use of double-line markings, and stopping sight distance (*1–5*).

requirement, the components of the minimum overtaking maneuver must be analyzed. In diagrams such as Figure III–2 of AASHTO's policy manual (*5*), shown here in a modified form as Figure 2, the complete maneuver was broken into two parts: the perception, analysis, and reaction (*AB*), and the overtake and return to right lane (*BC*). The decision point (*B*), the last point for aborting the maneuver, is at the boundary between the first and second phases.

The minimum passing sight distance requirement could be expressed in two ways, corresponding either to the length of the complete maneuver (*AE*), starting from the trailing position (*A*), or to the distance (*BD*) required to complete the maneuver from the point of no return (*B*). The former was called the establishment sight distance (ESD) in Australia (*2*), and the full overtaking sight distance (FOSD) in Britain (*3*), although the ESD included d_1 from Figure 2. These terms defined the minimum sight distance that should be available when the decision to attempt an overtaking maneuver is made, and therefore the sight distance needed before an overtaking restriction should be terminated. Most authorities seemed to require a minimum passing sight distance based on this version, the minimum length of clear road required to begin an overtaking maneuver as the geometric requirement. In Australia (*2*), the shorter sight distance required for safe completion of the maneuver beyond the point of no return was called the continuation sight distance (CSD), with computed values less than half those for the ESD; in Britain it was the abort sight distance (ASD), taken as half the FOSD. These terms represent the distance that should be the basis of barrier line installation. Figure 3 shows these different definitions.

The different dimensional standards for these minimum overtaking sight distance geometric requirements may have been the result of the different values assigned to variables such as relative speeds of overtaking, overtaken, and oncoming vehicles; clearance interval; and reaction plus perception time. Britain, for example, assessed the overtaking sight distance requirement for 50, 85, and 99 percent of the car and driver population. Troutbeck (*10*) discussed this topic on the basis of experimental work carried out in Australia.

TABLE 1 PUBLISHED SIGHT DISTANCE WARRANTS FOR THE USE OF DOUBLE LINES AND STANDARDS FOR MINIMUM PASSING SIGHT DISTANCES USED IN GEOMETRIC DESIGN

	Design Speed (km/hr)		
	50	70	100
Double-Line Warrants (m)			
Australia	150	210	300
Britain	90	125	185
Canada	160	240	400
United States	150	210	320
Minimum Passing Sight Distance Requirements (m)			
Australia[a]	350	570	1,010
Britain[b]	290	410	580
Canada[c]	340	480	680
United States[d]	360	490	680

[a]Defined as establishment sight distance.
[b]FOSD for 85 percent of the car and driver population.
[c]Minimum passing sight distance.
[d]Derived from AASHTO (*5*).

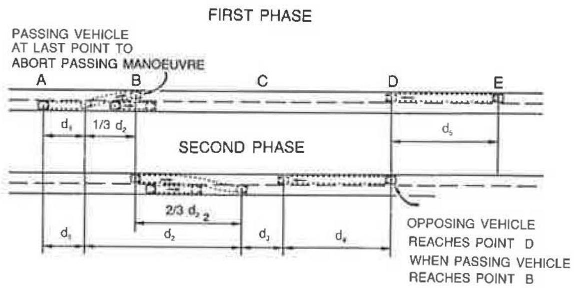

FIGURE 2 Movements of the three vehicles involved in the passing maneuver during the first and second phases (before and after, the point of no return), based on AASHTO (*5*), Figure III–2, modified to include movement *ED* of the opposing vehicle during the first phase.

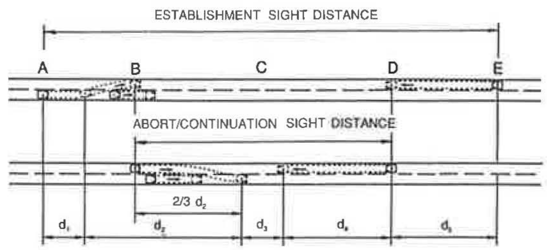

FIGURE 3 The two forms of passing sight distance requirement: the ESD, needed for the complete passing maneuver, the clear distance required before terminating a barrier line; and the ASD or CSD, needed at the point of no return, loss of which should be the warrant for the use of a barrier line.

The AASHTO policy handbook (*5*) was ambiguous about the derivation of its figures for minimum safe passing sight distance; its definition was between the long and the short versions. Figure III–2 showed the minimum passing sight distance requirement as *AD* in Figure 2, equal to $d_1 + d_2 + d_3 + d_4$, but this sum represented the addition of distances between vehicles in noncontemporaneous positions. Consider passing on a long, straight road. A decision to overtake, at point *A*, would be based on the road being clear through to *E*. If, when at *B* alongside the vehicle to be overtaken, an opposing vehicle appeared, the maneuver would be aborted if the vehicle was closer than *D* but continue if the vehicle was beyond *D*.

The length *BD* was the same as the Australian CSD. The equivalent of the Australian ESD was measured from the point when the overtaking vehicle was in the trailing position at *A*, at which time the opposing vehicle would be at *E*, still some 7 or 8 sec from point *D*. To be consistent, AASHTO should quote either $d_2 + d_3 + d_4$ for continuation conditions or $d_1 + d_2 + d_3 + d_4 + d_5$ for establishment conditions, both of which represent contemporaneous positions of the overtaking and oncoming vehicles.

Australian and American values for these two versions of minimum safe passing sight distance are presented in Table 2. The American figures are based on metrication of values given by AASHTO (*5*) in its Figure III–2. Figure 4 shows that, after making allowances for the point raised above and plotting distances against speed of the passing vehicle rather than against the nominal design speed, the two sets of figures were similar. The Canadian figures (*4*) were apparently based on AASHTO (*5*) so they may need to be amended in the

TABLE 2 COMPARISON OF AUSTRALIAN, BRITISH, AND EQUIVALENT AMERICAN VALUES FOR PASSING SIGHT DISTANCES

	85th-percentile speed (km/hr)						
	50	60	70	80	85[a]	90	100
Australia							
CSD (m), 75 percent	165	205	245	332	-	-	-
CSD (m), 85 percent	200	240	285	345	-	410	490
ESD (m)	350	450	570	700	-	840	1,010
Britain							
ASD (m), 85 percent	145	170	205	-	245	-	290
FOSD (m), 85 percent	290	345	410	-	490	-	580
United States							
Equivalent CSD (m)	185	250	315	380	-	445	515
Equivalent OSD (m)	260	350	445	-	580	-	725
Equivalent ESD (m)	345	470	590	715	-	840	965

[a]Britain uses 85 km/hr as a standard design speed.

same way. Table 2 and Figure 4 also show British overtaking sight distance minimum geometric requirements (*3*) for occupation of the passing lane and abort distance. These values were derived from rounded observed values of passing duration, taken as the occupancy of the passing lane (d_2). Unlike those used by the other countries, the British passing lane occupancy figures were found to be relatively unaffected by vehicle speed, and 50 percent of the overtaking maneuvers were completed in under 7 sec, 85 percent in under 10 sec, and 99 percent in under 14 sec. AASHTO (*5*) used a 9- to 11-sec range; Australia quoted 8 to 14 sec over the 50- to 100-km/hr speed range.

If each country had found the same passing lane occupation time, their passing sight distances might also have been the same, except for small differences in details such as clearance (d_3). The uniform passing time with speed of the British findings produced a linear relationship between passing sight distance and speed; the wide range of the Australian passing times, with speed, led to the curved relationship shown in Figures 1 and 4.

In the British computations, the FOSD required at the beginning of the passing lane occupation equated to 2.05 times the 85th-percentile speed multiplied by the time to complete the overtaking maneuver (i.e., the time the passing lane is occupied). In this way, the FOSD could be calculated for different overtaking populations at each design speed. This variable was another that led to differences between the national figures and made comparison difficult. The Americans and Canadians did not state, for example, whether their safe passing sight distance requirements would allow for all passings or for only the more adventurous to be accommodated, but AASHTO (*5*) gave the time of the passing lane occupation as close to 10 sec for d_2 across the 50- to 100-km/hr range. This figure corresponded to the British value for the 85th percentile; the Australian figures generally provided for 85 percent of their driving population.

The British Department of Transport's equivalent of the Australian CSD, called the ASD, was taken as half the FOSD, a proportion that was reasonably consistent with Australian and AASHTO (*5*) findings. However, the values were considerably different, perhaps as a result of different assumptions on relative overtaking and overtaken speeds. More field research is needed to establish whether these differences are the result of fundamentally different driver behavior in the various countries or of the different assumptions made in the computations.

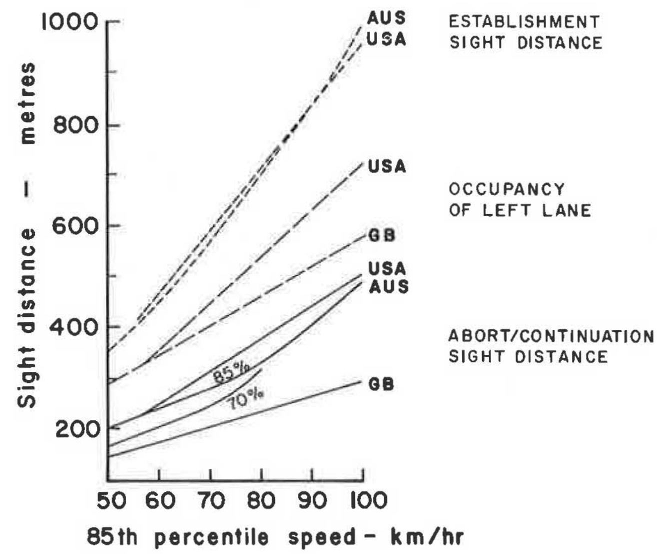

FIGURE 4 Different passing sight distance definitions: full requirement, occupancy of left lane, and minimum at point of no return.

Table 3 presents British, Australian, and American values for CSD and the minimum stopping distance values used in each country. The Australian CSD was almost identical to twice the stopping distance plus 5 sec at the 85th-percentile speed, and this was the basis for evaluating Australia's intermediate sight distance, now replaced by the CSD definition. American figures were even closer, but the British values did not conform to the pattern at all, despite similar stopping distance values. The reason that British values for ASD should be so different to those of the other countries was not clear.

TABLE 3 RELATIONSHIP BETWEEN STOPPING DISTANCES AND CSD OR ASD VALUES

	85th-percentile speed (km/hr)						
	50	60	70	80	85[a]	90	100
Australia							
Stopping distance (m)	45	60	75	95	–	120	155
2 × SD + 5 sec (m)	160	200	250	300	–	365	450
CSD (m)	165	205	245	320	–	410	490
Britain							
Stopping distance (m)	50	70	90	–	120	–	160
2 × SD + 3.6 sec (m)	150	200	250	–	325	–	420
ASD (m)	145	170	205	–	245	–	290
United States							
Stopping distance (m)	65	90	120	150	–	180	210
2 × SD + 3.6 sec (m)	180	240	310	380	–	450	520
Equivalent CSD (m)	185	250	315	380	–	445	515

[a]Britain uses 85 km/hr as a standard design speed.

SITING OF DOUBLE LINES IN RELATION TO POINT WHERE SAFE PASSING VISIBILITY IS LOST

Table 4 presents warrants for the use of double-line overtaking controls with the minimum CSD geometric requirements in the three countries. Most important to this investigation was the relationship, or absence of any explicit relationship, between minimum geometric sight distances and warrant distances. The differences between the two sets of sight distance standards shown in Table 4 introduced further inconsistencies, giving rise to the problem of reconciling the differences between geometric requirements for minimum passing sight distance, and those given in the warrants for the use of barrier lines. The similarities between the American and Australian figures are apparent in Figure 5, yet the positioning of the beginning of the barrier line was quite different.

British and American warrants both required the barrier line to start at the point where sight distance is lost, as specified in the warrant for the 85th-percentile speed at that point. The Australian warrant, with speed and sight distance figures almost identical to those of Americans, required the line to start at a given distance beyond this point of visibility loss. The Australian method immediately commended itself for its rationality. The significance of the distance an overtaking driver could see when he had completed the overtaking maneuver—for this was the requirement of the American and British method—was unclear. The American and British method started the barrier line, that is, the point where overtaking is complete at the point where the warrant sight distance is lost (point *C* in the figures). This distance may have been used as a proxy for some other. The significant sight distances that must be available to the passing driver are point *A*, when he decides to begin a passing maneuver, and point *B*, when he must decide whether to complete or abort the maneuver at the point of no return. The British and American approach suggested that they were using a different definition of no-passing barrier-like markings than the Australians.

America and Britain, in fact, both used a short-zone definition of barrier-line meaning (defined in following paragraphs), yet started the barrier line at the point the warrant sight distance is lost, a practice that fits the long-zone use of

TABLE 4 RELATIONSHIP BETWEEN ASD OR CSD VALUES AND WARRANTS FOR THE APPLICATION OF DOUBLE-LINE CONTROLS

	85th-percentile speed (km/hr)						
	50	60	70	80	85[a]	90	100
Australia							
CSD (m)	165	205	245	320	–	410	490
Warrant (m)	150	180	210	240	–	270	300
Britain							
ASD (m)	145	170	205	–	245	–	290
Warrant (m)	90	105	125	–	155	–	185
United States							
Equivalent CSD (m)	185	250	315	380	–	445	515
Warrant (m)	155	175	210	240	–	280	315

[a]Britain uses 85 km/hr as a standard design speed.

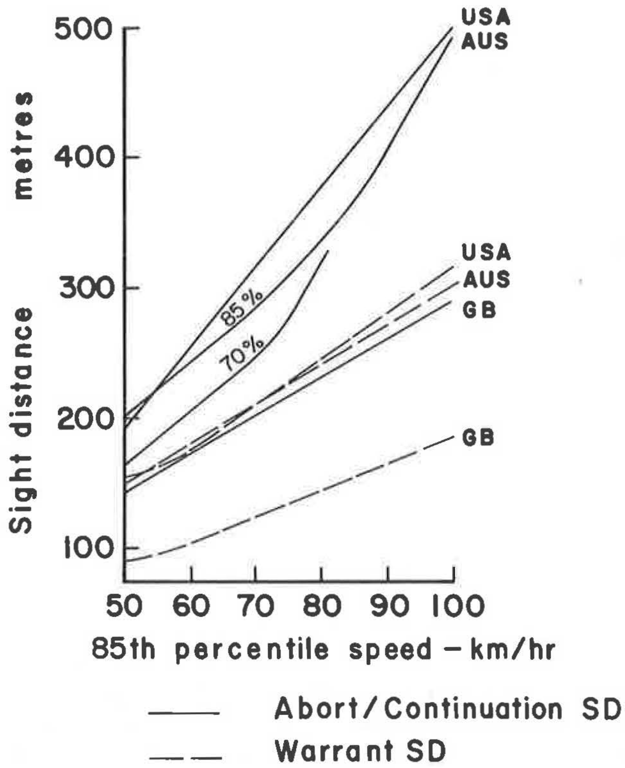

FIGURE 5 Comparison of traffic control warrants with minimum geometric passing sight distance requirements.

barrier lines. Australian practice started the barrier line at a distance about half the ASD beyond the point of loss of the ASD. American practice did not recognize the ASD as significant, although the calculation of passing sight distance is half-based on this concept. But because the ASD was a geometric design concept, with values much higher than those of the warrant sight distance, installing the barrier line in accordance with the warrants produced results similar to what would result from installing it by the previously outlined geometric principles.

No simple relationship between the two approaches could be foreseen, because of the effect of alignment at specific sites. In some locations, both ASD and warrant sight distance might be on the same horizontal or vertical arc, whereas at others the latter may be on the arc but the ASD may include both arc and tangent lengths.

These warrants may have been found to be satisfactory through custom and practice, but some more rational method would be preferable. Only the geometric design approach has the rationality of being based on behavioral research. A single basis for determining passing requirements, and for installing barrier lines where those requirements were not met, would be an improvement.

The assumption will be made that the present warrants should be replaced by the more rational approach of beginning the barrier line at some point related to the loss of CSD or ASD. Troutbeck's treatise (*10*) is the only known example in which the beginning of the barrier line was recommended to be positioned explicitly with respect to the point of loss of geometric overtaking visibility.

Before examining the detail of an alternative to the conventional approach of using independent warrants, the legal meaning attached to the barrier line should be understood. Troutbeck (*10*) identified two legal forms of barrier-line control. In the first, legislation required that the overtaking maneuver must not be begun, that is, the vehicle must not move from right to left lane, after the beginning of the barrier line (Point *B* in Figure 6). Returning to the right lane by crossing the barrier line to complete an overtaking maneuver was permitted. Troutbeck called this method the long-zone definition, when the barrier line started at the point where overtaking sight distance is lost. Few states use this definition in their legal codes. Troutbeck's short-zone definition fixed the beginning of the barrier line at the point by which the vehicle must have completed the overtaking maneuver and returned to the right lane (Point *C*), so that no crossing of the barrier line would be permitted in any circumstance.

In the long-zone form, the barrier line could be unduly restrictive, especially if based on loss of an FOSD that would permit a large percentage of the overtaking-performance spectrum to take place safely in shorter distances than the geometric design minimum, on the basis of the overtaking requirements of 99 or 85 percent of the driver population. Also, the long-zone form gave no guidance to overtaking drivers about the point by which the maneuver must be completed and was unsatisfactory for enforcement purposes.

In the short-zone form, the beginning of the barrier line advised drivers of when the passing maneuver must be completed. This form also avoided ambiguity in the meaning of the marking; a barrier line, as the right-hand member of a pair of lines, must never be crossed. However, this definition gave the driver no advice about the last safe point to begin an overtaking maneuver or about the point of no return. A further disadvantage was that the beginning of the double lining was some distance ahead of the last safe point for beginning a maneuver at the higher design speeds, probably beyond the distance at which a double line could be distinguished by most drivers, so they might be unaware of its presence when starting to pass. To a certain extent, the pennant road sign used in the United States served this purpose by providing a more visible marker, but the sign gave no indication of the point of no return and would be difficult to apply where, as on many older two-way roads, there was considerable roadside activity. Troutbeck suggested that this limit of conspicuity of a pavement marking was around 80 to 100 m, so for design speeds above 50 km/hr the presence of an upcoming double-line marking would be inconspicuous at the point of no return.

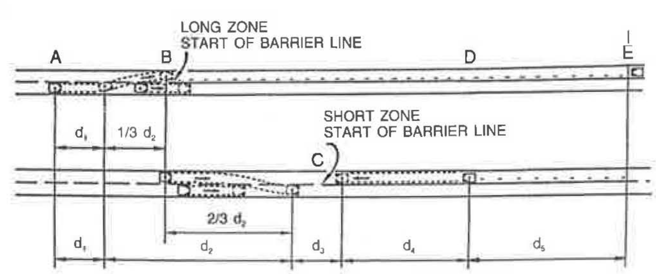

FIGURE 6 Alternative long- and short-zone definitions of the no-passing zone in relation to the minimum passing distance requirement.

This situation would be unrealistic, and some advanced warning of approach to a barrier line should be provided. Whether this should be given at the last point for beginning a maneuver or at the point of no return is debatable.

No provision for this kind of advance-warning road sign or pavement marking was included in the uniform codes of Australia (*6*), Canada (*8*), or the United States (*9*), or in the Vienna Convention and Protocol on international standardization of road signs and pavement markings. The British code (*7*), which follows the European Agreement with certain reservations and additions, contained a pavement marking for use in this situation. However, its use was not carried through in the rational way that would come from following the recommendations of Troutbeck.

In Britain, three types of directional-driving pavement-marking lines are used: advisory center lines; mandatory double lines; and a warning line for use where crossing the center line was hazardous but not forbidden. The warning line, as intermediate between the advisory center line of short marks and long gaps (or sometimes equal marks and gaps) and the continuous line of the interdictory double-line marking, consisted of long marks and short gaps, in a 2:1 ratio. Unfortunately, the British code required the warning line to be used in a rather arbitrary way at the approaches to a double-line zone, instead of using it to indicate the zone between the loss of either FOSD or CSD and the start of the barrier line (i.e., for the length of either *AC* or *BC*).

Adoption of this pavement marking system, with or without pennant signs, would allow points of loss of both FOSD and CSD to be advised to the driver in a way that would not penalize those drivers requiring a shorter passing time, while continuing the use of the unambiguous mandatory message of the continuous barrier line as the required point for completion of all passing maneuvers.

Figure 7 shows how the pavement marking system that is being recommended here would work. For each direction of travel, after the 85th-percentile speed has been measured and an appropriate design speed for the sight-restricting element of alignment has been determined, four salient points on the road's center line must be located:

1. The point at which FOSD for the design speed is lost (having first decided whether this distance is to accommodate 99 percent or a smaller percentage of the driving population),

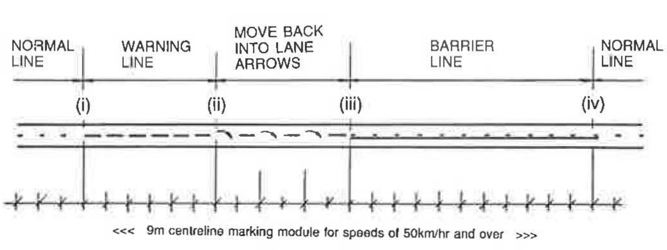

FIGURE 7 The recommended pavement marking system resulting from the short-zone definition and European and Australian pavement marking practice, which bases warrants and the start of the barrier line on loss of CSD, and uses a new marking system to indicate the approach to a no-passing zone.

2. The point at which ASD or CSD is lost,
3. The point by which overtaking maneuvers must be completed, which is half the ASD or CSD ahead of Point 2, and
4. The point at which sight distance again exceeds the FOSD.

The pavement markings will be warning lines between Points 1 and 3 and barrier lines between Points 3 and 4 or until certain checks on the alignment beyond Point 4 are satisfied. Between Points 2 and 3, every second mark of the warning line, counting back from the start of the barrier line, is replaced by a move back into lane arrow, to indicate the imminence of the start of the barrier-line marking. The warning line consists of a 6-m line followed by a 3-m gap where the design speed is 50 km/hr or faster; otherwise, a 4-m line followed by a 2-m gap. Over those lengths where the available visibility, in either direction, is less than that required for FOSD but never falls to as little as the ASD or CSD, the center line is to be marked with the warning line, to indicate the more hazardous situation for overtaking.

At present, many sites in the United States appear to be too restrictively marked for no passing, perhaps as a result of following the misleading tabulation of passing sight distances against design speed in the AASHTO policy manual (*5*). A rectification of this situation might lead more road users to recognize the validity of the passing prohibitions.

CONCLUSION

The investigation of four countries' methods for the application of controls to sites where sight distances were inadequate for safe passing revealed that the apparent differences in published geometric standards between Australia and the United States could be reconciled when like situations were compared, and that the AASHTO (*5*) analysis appeared to be in need of review. British values were found to be signif-

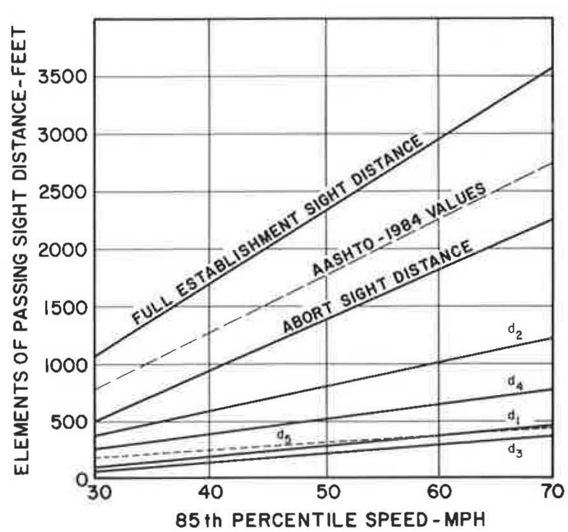

FIGURE 8 Amended Figure III–2 (*5*) to show U.S. equivalent of ESDs and CSDs with the average speed of passing vehicle taken to be the 85th-percentile speed of the highway.

icantly smaller than those of the other countries, although some of the differences were caused by different assumptions about the components of the passing maneuver or the population that is accommodated. AASHTO (*5*) might be recommended to revise its Figure III–2 and dependent Table III–5 to use the average passing speed as the design speed and to recompute FOSD and ASD requirements as described, as an interim measure until further field work verifies the various increments of time in the passing maneuver. An interim revised AASHTO (*5*), Figure III–2, might look like Figure 8, indicating both FOSD and ASD requirements. When a sight distance in one direction was between that required for the FOSD and the ASD value, a warning center line should be used; when less than the latter, a barrier line should begin half the ASD beyond the point where the ASD distance is lost.

The study's central objective was to seek a standard method for installation of double-line pavement markings, but no consensus was found. Australia, using much the same warrants to establish locations requiring the double-line markings as does America, located the beginning of the double lining quite differently. Britain shared the American and Canadian method to find the start of the barrier line but used smaller warrant lengths. Only Australia positioned the start of the double lines at a point explicitly related to the loss of a geometric requirement for passing sight distance.

Most current methods of determining the position of double-line pavement markings to indicate overtaking prohibitions had some of the following deficiencies:

- Incompatibility with actual geometric overtaking sight distance requirements,
- Possible ambiguities of interpretation by highway engineers, and
- Inadequacy in conveying advance warning of the overtaking prohibition and the point of no return to road users.

Present methods separating traffic control techniques and their warrants from geometric design requirements for passing sight distance are unsatisfactory. The present warrant approach is recommended to be discarded in favor of one based on geometric principles, with an additional form of pavement marking to indicate the approach to a passing-restricted zone.

The recommended method of determining the positions of double-line pavement markings has the advantage of rationality over methods apparently in use in some states of Australia, Britain, Canada, and the United States. The method is independent of each country's differences in values for minimum geometric overtaking sight distance standards, which could still be used in the manner of application described, and would replace the warrants currently in use. To be able to carry out the suggested rationalization of their double-lining system, Australia, Canada, and the United States would have to adopt some new style of pavement-marking warning line, but that warning line would be an entirely consistent extension of current practice and one that, in a slightly different form, has already been recommended for adoption in Australia.

The conclusion that the present method of identifying and marking sections of two-way highway where no overtaking should take place should be replaced does not claim that present methods are, in general, unsatisfactory or unsafe. Rather, they could be improved and made more logical, particularly to relate them more closely to the geometric design of highways. This may be a good time for a new study on passing behavior that would show whether AASHTO's longstanding values for duration of the passing maneuver are still representative of contemporary vehicle performance and today's drivers.

ACKNOWLEDGMENT

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Safety Factors for Road Design: Can They Be Estimated?

FRANCIS P. D. NAVIN

When asked to state the safety of a particular design, highway engineers are at a loss to give a single meaningful measure, as is possible in structural or geotechnical engineering. The question of a meaningful road safety design measure led to a method to estimate the margin of safety and safety index for isolated highway components. The method uses the basic highway alignment design equations and, on the assumption that the variables are independent, normal, and random, the expected value of the mean and variance are estimated and the margin of safety and safety index are derived. The proposed measures of road geometric design safety are the margin of safety and safety index. The variables included in the safety measures represent the characteristics of driver, vehicle, and road surface. Calculations based on preliminary information indicate that the safety index is the most meaningful safety measure for road design. A general method to specify the design parameter's value is also proposed. This method is based on factors that represent the strategic importance of a road, the number of road users, the type of vehicles, the quality of the drivers, the expected environmental conditions, the terrain, and the general standard of design or construction. These factors are all found implicitly in current design procedures. The apparent advantage of the proposed method is that the designer must explicitly specify the importance of the modifying factors. Research is required to make the method useful. The research must clearly develop the mean, variance, and distribution of the variables used in the basic geometric design equations. Further information will eventually be needed on the interaction of the variables, to remove the independence requirement and to permit an estimate of the road system reliability over a specified road link.

Many Canadian authors, including Navin (*1*), Hauer (*2*), and Hutchinson (*3*), have criticized the current road geometric design procedures for failing to meet the established operational safety standards for certain vehicles. In particular, a problem appears to be developing around the differing highway geometric demands for cars and large trucks.

The road geometric design standards by the Roads and Transportation Association of Canada (RTAC) (*4*) or AASHTO (*5*) and operational standards such as *Uniform Traffic Control Devices for Canada* (*6*) and the ITE Handbook (*7*) all work on the implicit assumption that if the published standards have been correctly applied, the road has an adequate margin of safety. This assumption is also accepted by the courts when ruling on the designer's liability for vehicular accidents where road geometry or operation is suspected. One method by which meaningful measures of road safety may be estimated will be outlined.

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The basic assumption is that all the variables used to estimate the design parameters in the elementary geometric design components are independent, normal, random variables. For example, when estimating the stopping sight distance design parameter, speed, perception-reaction time, and coefficient of friction are independent, normal, random variables. The published values of these variables are used in conjunction with expected value methods to obtain estimates of the mean and variance of the design parameters. The expected values are in turn used to calculate measures of road safety.

DEFINITIONS

The fundamental ideas behind the proposed method are those found in limit states design, as used by structural engineers. This approach requires the designer to think of the demand D_0 by driver-vehicle systems for a particular highway design parameter and also of the supply S_0 of this parameter provided by the current highway design standard. The general arrangement for such a system is shown in Figure 1. The driver-vehicle system demand is some random distribution about the mean value of D_0 and that supplied by the highway is the value S_0. In this example, when the supply S_0 is exceeded by the demand D_0 the system is considered to have failed. Failure is thus defined by the engineer or appropriate standard and need not actually result in an accident. A few additional definitions that will be used require explanation.

The simplest measure of safety is the central factor of safety [SF(Central)], which is defined as the ratio of the average

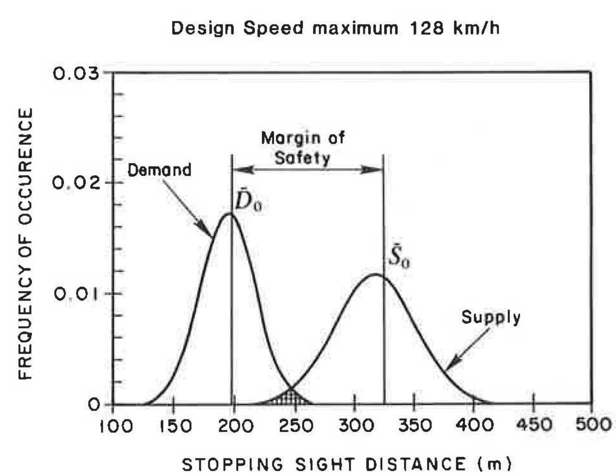

FIGURE 1 Stopping sight distance.

supply $\bar{S}_0$ and the average demand $\bar{D}_0$ (*8*). This value, given in Equation 1, is rarely used.

$$\text{SF(Central)} = \bar{S}_0/\bar{D}_0 \quad (1)$$

The most common measure is the conventional factor of safety. The ratio of the average demand is increased and the supply is reduced, by some multiple of the standard deviation. This approach implies that designers are uncertain about the exact values and allow for more demand and less supply of the parameter, as seen in Equation 2.

$$\text{SF(Conventional)} = \frac{\bar{S}_0 - k\sigma_{S0}}{\bar{D} + k\sigma_{D_0}} \quad (2)$$

where k is any multiple of the standard deviation (σ).

Extending this idea of uncertainty further, a finite chance exists that the demand will exceed supply; for example, the stopping sight distance required by the driver-vehicle system exceeds that provided by the highway design. The reasons why the demand exceeded supply are not important at this point, but this event may occur, and its occurrence is ascribed to random events rather than gross human error. Ang and Tang (*9*) give the method for deriving the expected value and variance of a design parameter and the derivation of measures of safety. The first measure is the margin of safety M, which is the difference between the expected value of the supply and the expected value of the demand, given in Equation 3. The ratio of the margin of safety and the combined variance, expressed in Equation 4, is defined as the reliability index or safety index (β).

$$M = E\langle S_0\rangle - E\langle D_0\rangle \quad (3)$$

$$\beta = \frac{M}{\sqrt{\text{Var}\ (S_0) + \text{Var}\ (D_0)}} \quad (4)$$

The chance of failure given by the safety index may be evaluated by normal probability methods if the variables in the basic equation are a linear combination. If they are not, then the correct chance of failure must be estimated by methods given by Ang and Tang (*9*).

To be accepted, the derived equations must have variables that are easily obtained and useful to both the road designer and road operator. Also, the minimum number of variables should be included in any parameter to keep them reasonably simple. The parameters should be easily understood both by the engineer and nonengineer. Finally, system elements such as road characteristics, driver behavior, and vehicle capabilities must be explicitly considered.

EARLY RESEARCH

The pioneering work by Moyer and Berry (*10*) on marking highway curves with safe speed indications gives the clearest insight into how early highway engineers thought about relative safety. Moyer and Berry (*10*) summarized the research, "The safe speed has largely been determined on the basis of retaining control of the car on the curve." The safe speed on a curve was defined by a 10-degree ball bank reading. They continued, "The general acceptance of this value is rather surprising because it is, after all, an arbitrary value at which the driver of a car senses some discomfort and where the hazard of skidding off the curve becomes apparent." The authors also took driver's attitude into consideration by recommending "14 degrees for speeds below 20 mph, 12 degrees for speeds of 25 and 30 mph, and 10 degrees for speeds of 35 mph and higher." The corresponding friction factors were 0.21, 0.18, and 0.15, for 20 mph, 25 to 30 mph, and 35 mph or higher, respectively. Satisfactory speed levels were indicated by acceptance by "a percentile value of 85 percent for curves of 30 mph or less and 90 percent for curves with speeds of 35 mph." Moyer and Berry (*10*) gave suggestions for rough roads and nighttime speeds. "The only condition in which the ball bank angle of 10 degrees or higher will not indicate the safe daylight speed is when the surfaces are slippery when wet, or ice or snow covered."

Moyer and Berry (*10*) addressed the relative safety of their recommendation.

> While it is true that friction values are lower on wet surfaces than on dry surfaces, . . . there is still a large margin of safety on wet surfaces properly constructed and maintained if the low value of $f = 0.1$ at a ball bank reading of 10 degrees is used . . . .

Further,

> asphalt, concrete and similar types with a gritty surface texture or sandpaper finish provide a wide margin of safety against skidding for speeds with a ball bank value of 10 degrees. . . . This analysis shows that drivers can drive safely at the posted speed on properly constructed and maintained surfaces when wet and even when covered with snow free from ice.

Figure 2 supports these statements by showing the coefficient of friction versus speed, tire conditions, and road surface type. Moyer and Berry (*10*) did not estimate the margin of safety. The method presented here will show how such estimates may be made.

The development of the AASHTO vertical curve design standard contrasts with the pragmatic research of Moyer and Berry (*10*). An excellent summary of the history of stopping sight distance in the United States is given by Hall and Turner (*11*). Neuman (*12*) gives the following overview of the AASHTO policy.

> The minimum sight distance available should be sufficiently long to enable a vehicle traveling at or near the likely top speed to stop before reaching an object in its path. While greater length is desirable, sight distance at every point along the highway should be at least that required for a below-average operator or vehicle to stop.

Vertical curve design reduces to simply determining the driver's eye height and the height of an obstacle on the road, given a budget and a driver's reaction time. The U.S. standard, used in 1940 and adopted in 1954, was an object of 4 in. and an eye height of 4 ft 6 in. Increasing the object from 0 to 4 in. reduced the length of the vertical curve by 40 percent, but above the 4-in. object height, little economic gain was derived. During the 1950s, the driver's eye height dropped,

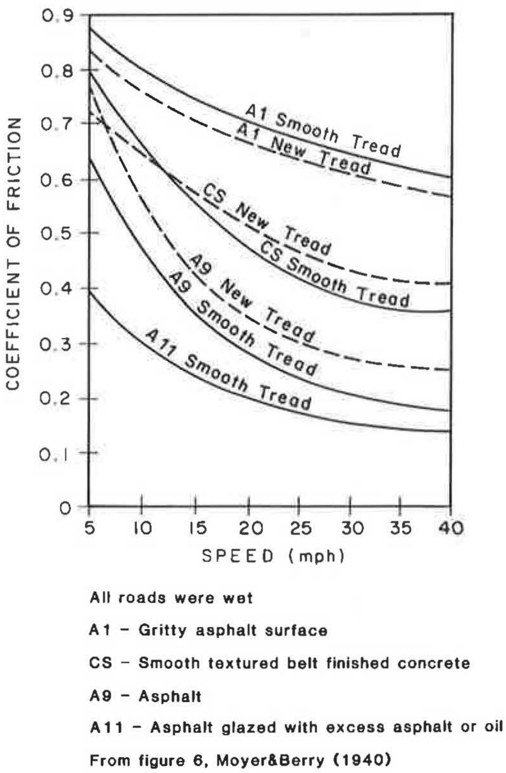

FIGURE 2 Effect of surface texture and surface condition on coefficient of friction for new and smooth-tread tires.

so the 1965 AASHTO Blue Book specified an object height of 6 in. and a driver's eye height of 3 ft 9 in.

The 1965 AASHTO committee thought the "standards adopted in 1954 were somewhat liberal." An object 7 in. high would give a stopping sight distance equal to that of the 1954 standard. The reduction of 1 in. was adopted, because it would be "wise to provide a factor of safety. . . ." In Canada, RTAC 1976 (*4*) recommends a maximum object height of 15 in. and a desirable object height of 6 in. with an eye height of 3 ft 5⁵⁄₁₆ in.

These two examples illustrate the concern that highway engineers have for highway safety and how they have developed design policies, constrained by budgets and the inability to estimate factors such as the margin of safety and a safety index for isolated components of a highway.

STOPPING SIGHT DISTANCE

Theory

Stopping sight distance is fundamental to all geometric design. To calculate the distance, suitable values as set by policy are assigned to the variables of Equation 5. This equation represents the stopping sight distance supplied by the highway as the sum of the driver's perception-reaction distance plus the vehicle's braking distance.

$$\mathrm{SSD}_H = V_H T + \frac{V_H^2}{2\alpha_x^H} \tag{5}$$

where

SSD = stopping sight distance (m),
V = velocity (m/sec),
T = perception-reaction time (sec),
α = deceleration rate (m/sec²),
H = highway,
x = longitudinal axis of highway,
D = driver, and
v = vehicle.

For the driver-vehicle system, which is assumed to have small random errors, the corresponding expected values of the mean and variance are

$$\mathrm{E}\langle \mathrm{SSD}_v\rangle = V_D T_D + \frac{V_D^2}{2\alpha_x^v} + \frac{V_D^2\left(\sigma_{\alpha_x^v}^2\right)}{2(\alpha_x^v)^3} + \frac{\sigma_{V_D}^2}{2\alpha_x^v} \tag{6}$$

$$\mathrm{Var}\langle \mathrm{SSD}_v\rangle = V_D^2\sigma_{TV}^2 + \frac{V_D^4\left(\sigma_{\alpha_x^v}^2\right)}{4(\alpha_x^v)^4} + \left(T_D^2 + 2\frac{V_D T_D}{\alpha_x^v} + \frac{V_D^2}{(\alpha_x^v)^2}\right)\sigma_{V_D}^2 \tag{7}$$

where σ denotes the standard deviation of the corresponding distribution.

The stopping sight distance demanded by the driver-vehicle system depends on the speed V_D that the driver selects, his or her perception-reaction time T_D, and the stopping capabilities of the vehicle. No relationship is assumed between the driver's ability to brake the vehicle and the vehicle's ability to stop. This complexity may be included and will no doubt influence the numerical results, but adds little to the arguments being presented.

The general relationship between the stopping sight distance supplied by highway design and the distance demanded by the driver-vehicle system may be either a single value from the design manual or the actual value supplied after construction and changes over time with the quality of the road. A single design value has been assumed for simplicity of the arguments, even though most of the equations are derived for the more general case. Failure is defined as when the demanded stopping sight distance exceeds the distance supplied. To compare the supply and demand, the measures of interest are the margin of safety and the reliability or safety index, as previously defined.

The margin of safety for stopping sight distance is the difference between the stopping sight distance supplied by the highway and that demanded by the driver-vehicle system, given by the following equations:

$$M(\mathrm{SSD}) = E\langle \mathrm{SSD}_H\rangle - E\langle \mathrm{SSD}_v\rangle \tag{8}$$

$$\mathrm{Var}\langle M(\mathrm{SSD})\rangle = \mathrm{Var}\langle \mathrm{SSD}_H\rangle + \mathrm{Var}\langle \mathrm{SSD}_v\rangle \tag{9}$$

The safety index for stopping sight distance is as follows:

$$\beta\,(\mathrm{SSD}) = \frac{E\langle \mathrm{SSD}_H\rangle - E\langle \mathrm{SSD}_v\rangle}{\sqrt{\mathrm{Var}\,(\mathrm{SSD}_H) + \mathrm{Var}\,(\mathrm{SSD}_v)}} \tag{10}$$

If the AASHTO standard is used for the highway, then the safety index is as follows:

$$\beta\ (\mathrm{SSD}) = \frac{\mathrm{SSD}_H - E\langle \mathrm{SSD}_v \rangle}{\sqrt{\mathrm{Var}\ (\mathrm{SSD}_v)}} \tag{11}$$

The evaluation of the probability of failure from Equations 10 and 11 requires methods explained by Ang and Tang (*9*), because of the nonlinear combination of variables. The safety index used represents the situation in which the demand is random and the supply is fixed as a single value.

Estimated Values

The mean and standard deviation of the variables used in the stopping sight distance calculation are presented in Table 1. Few sources of good, basic data are available to precisely define the statistical nature of the variables. The graphical results for the lower AASHTO standard are shown in Figure 3. If these values are reasonable, the resulting margin of error and safety index are as follows:

- AASHTO high values (variance set to zero) compared to driver-vehicle system: SSD_H is 198 m, margin of safety is 61 m, safety index is 1.22, and chance of failure is about 1 in 10.
- AASHTO low values (variance set to zero) compared to driver-vehicle system: SSD_H is 160 m, margin of safety is 23 m, safety index is 0.42, and chances of failure are about 3 in 10.

Failure has been defined as the driver-vehicle system's demanding a stopping sight distance greater than that prescribed by the highway design. Failure may or may not result in a serious physical outcome, depending on particular circumstances.

The distribution of margin of safety is the normal distribution, shown in Figure 4. The two values plotted are AASHTO low, which represents the highway supply, and the driver-vehicle system, which represents the demand. The margin of safety distribution $f(M)$ is as follows:

$$f(M) = \frac{1}{[2\pi\ \mathrm{Var}\ (M)]^{1/2}} \exp\left\{-\left[\frac{(M-\overline{M})^2}{2\ \mathrm{Var}\ (M)}\right]\right\} \tag{12}$$

TABLE 1 VALUES FOR STOPPING SIGHT DISTANCE (SSD) VARIABLES

Variable	Unit	Highway, AASHTO		Driver/Vehicle	
		High	Low		Source
V	km/h	95	85	80	AASHTO (5)
T	s	2.50	2.50	1.35	Olsen (13)
α_x	g	0.29	0.29	0.24	Navin (1)
σ_V	km/h	0	0	8.0	Olsen (13)
σ_T	s	0	0	0.18	Olsen (13)
σ_α	g	0	0	0.06	Navin (1)
SSD calculated	m	188	154	137	
SSD design	m	198	160		

The distribution of Figure 4 gives an indication of how an acceptable value of margin of safety might be estimated and then explained to those who set policies for acceptable safety index values.

HORIZONTAL CURVE

Theory

This analysis is similar to that for the stopping sight distance. The failure mode is assumed to be a vehicle rollover measured by the radius of turn. The highway's radius of curve is given by the following equation:

$$R_H = \frac{V_H^2}{\alpha_y^H} = \frac{V_H^2}{(e^H + f_y^H)g} \tag{13}$$

where

R_H = radius of curve (m),
V_H = highway design speed (m/sec),

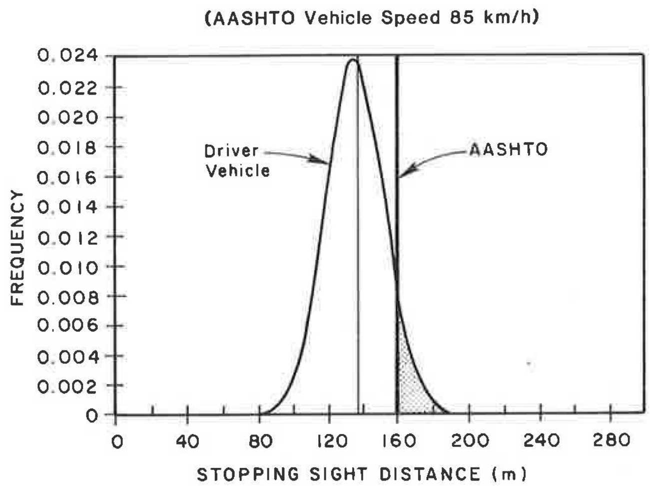

FIGURE 3 Stopping sight distance (SSD) distribution.

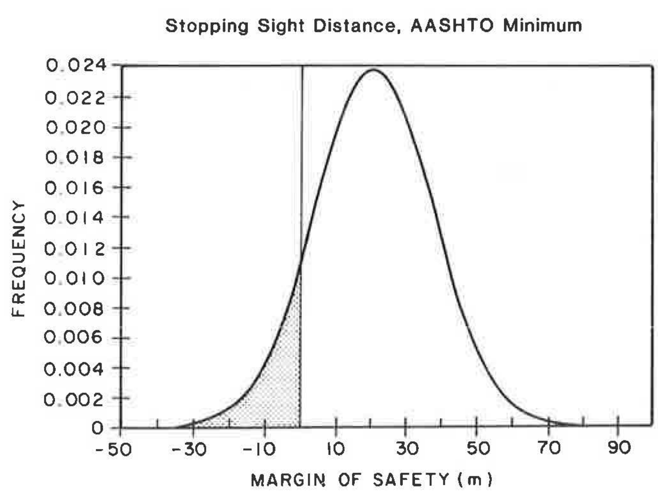

FIGURE 4 Margin of safety (*M*) distribution.

g = acceleration of gravity (9.8 m/sec^2),
α_y^H = highway design lateral acceleration (m/sec^2),
e^H = highway superelevation (m/m), and
f_y^H = highway lateral friction factor.

The expected radius demanded by the driver-vehicle system (R_D), assuming the vehicle velocity and rollover threshold acceleration are normally distributed, independent, random variables, is

$$E\langle R_D\rangle = \frac{V_D^2}{\alpha_y^v} + \frac{V_D^2\left(\sigma^2_{\alpha_y^v}\right)}{(\alpha_y^v} + \frac{\sigma^2_{V_H}}{\alpha_y^v} \tag{14}$$

The variance is

$$\mathrm{Var}\langle R_D\rangle = \frac{V_D^4\left(\sigma^2_{\alpha_y^v}\right)}{(\alpha_y^v)^3} + 4\frac{V_D^2}{(\alpha_y^v)^2}\sigma^2_{V_D} \tag{15}$$

The margin of safety for radius is defined as the difference between the supply of the highway and the demand of the driver-vehicle system.

$$M(R) = R_H - E\langle R_D\rangle \tag{16}$$

The difference between the radii of turn is the suggested measure, even though this measure is not as intuitive as, for example, the stopping sight distance. The radius was selected because it is a physical design parameter that is easily related to the vehicle's velocity and threshold rollover acceleration. The lower the radius demanded by the driver-vehicle speed or stability, the safer the turning maneuver. Only when the radius demanded by the driver-vehicle system exceeds that provided by the highway should the curve fail insofar as the available radius has been exceeded.

Setting R_H as the single AASHTO value, the safety index for radius becomes

$$\beta\,(R) = \frac{R_H - E\langle R_D\rangle}{\sqrt{\mathrm{Var}\langle R_D\rangle}} \tag{17}$$

Equations 14 and 17 form the base of the remaining measures. These equations have factors that represent the highway design elements (H), the driver (D), and the vehicle (v). These are the elements considered important for both design and analysis.

Estimated Value

The failure of a driver-vehicle system on a highway curve may be either a rollover or a slide-out. Cars will usually slide out and trucks roll over. The rollover mode of failure is assumed. The values of the variables are from Moyer and Berry (*10*) for the original design decisions and from the University of Michigan Transportation Research Institute (*14*) and Navin (*1*) for modern tractor-trailer rollover.

Moyer and Berry, in discussing the margin of safety of their proposal to use a ball bank reading of 10 degrees, stated,

> As is evident . . . considerably higher friction values than 0.15 and higher speeds than that for a ball bank reading of 10 degrees are possible. . . . This is most evident on the sharper ones, such as the 61-ft and the 100-ft radius curves. On these curves, friction values close to 0.5 were developed at speeds almost double the safe speed based on a ball bank reading of 10 degrees. The ride at these speeds was far from comfortable and the limit of steering control was not far off; in fact, the path of the car was increasingly uncertain as the top speeds of these curves were approached.

Using the values of R, f_y, and V given by Moyer and Berry (*10*) in their Figure 5, the safety margins for the rollover failure mode are presented in Table 2. The very sharp curve (R = 18.6 m) has an estimated 2.6-m margin of safety at the recommended safe speed, and the slightly longer one has a 29.1-m margin of safety. At the limiting speed of the curves, the margin of safety is −0.9 m or more for the sharpest curve and 6.9 m for the longer. This simple analysis does not prove that the definition of margin of safety is correct, but it gives it some credibility on the basis of Moyer and Berry's (*10*) description of vehicle handling.

When the design speed is set at 95 km/hr with f_y equal to 0.12 and e_H equal to 0.06, and the operating speed is set at 85 km/hr (σ_v = 8.0 km/hr), and a vehicle's α_y^v equals 0.45 g (σ_α = 0.08g), the results shown in Figure 5 are determined.

If tractor-trailer rollover is the design criteria, the AASHTO design safe-speed radius of turn is 410 m and $E\langle R_{\text{veh}}\rangle$ is 116 m. The average margin of safety is 294 m and the chance of a random rollover failure is remote. Another method of calculating a more realistic failure probability of about 1 in 100,000 is given in Navin (*15*). Similar calculation for a car places β such that failure by rolling over is remote. These computations show how the process may be used to arrive at acceptable estimates of a safe curve. They do not necessarily represent actual margins of safety.

DECISION SIGHT DISTANCE

Theory

The decision sight distance is associated with high-speed roads where stopping is not permitted and decisions must be made while speeds are maintained. The failure mode in such circumstances is assumed to occur when the driver requires a distance greater than that provided by the highway. The fact that a failure may occur by technical definition does not

TABLE 2 ESTIMATED MARGIN OF SAFETY (M) ON A CURVE

		Curve 1		Curve 2	
Variable	Unit	Safe Speed	Driving Limit	Safe Speed	Driving Limit
f_y		0.14	0.46	0.14	0.48
V	m/s	5.4	8.9	11.2	17.9
R_v	m	22.2	17.1	90.1	67.9
R_H	m	18.6	18.7	61.1	61.0
M	m	2.6	-0.9	29.1	6.9
M/R_H	%	14	-5	48	11

necessarily mean that the physical result is an accident. The design relationship used to estimate decision sight distance is

$$DSD_H = (T_P + T_D + T_M)V_H \tag{18}$$

where

DSD = decision sight distance (m),
T_P = perception time (sec),
T_D = decision time (sec), and
T_M = maneuver time (sec).

The expected value and variance for the driver-vehicle system are given by the following equations:

$$E\langle DSD_D\rangle = (T_P + T_D + T_M)V_D \tag{19}$$

$$\mathrm{Var}\langle DSD_D\rangle = (\sigma_{TP}^2 + \sigma_{TD}^2 + \sigma_{TM}^2)V_D^2 + (T_P + T_D + T_M)^2\sigma_{VD}^2 \tag{20}$$

Using Equations 3, 4, 19, and 20, the safety margin and safety index for decision sight distance are as follows:

$$M(DSD) = DSD_H - E\langle DSD_D\rangle \tag{21}$$

$$\beta\,(DSD) = \frac{DSD_H - E\langle DSD_D\rangle}{\sqrt{\mathrm{Var}\langle DSD_D\rangle}} \tag{22}$$

The safety index is easily computed in this case, because the variables in the basic equation are a linear combination and the normal probability tables may be used.

Estimated Value

The values from Table 3 and Equations 21 and 22 are used to produce the following results. The mean and variance of the various times are estimated from data spread throughout the ITE handbook (*7*). The safety margin and safety index are calculated using the decision sight distance recommended for design purposes.

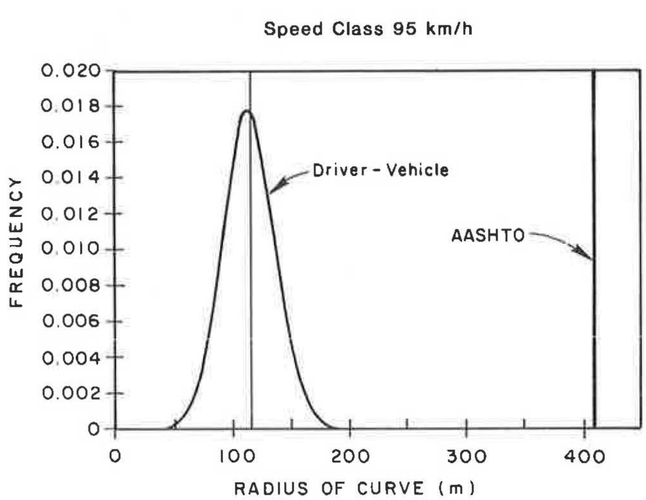

FIGURE 5 Radius (*R*) of horizontal curve.

TABLE 3 VALUES FOR DECISION SIGHT DISTANCE (DSD) VARIABLES

Variable	Unit	Highway, ITE		Driver/Vehicle	
		High	Low		Source
V	km/h	95	85	80	ITE(7)
T_P	s	3.0	2.0	2.5	ITE(7)
T_D	s	7.0	4.7	5.9	ITE(7)
T_M	s	4.5	4.5	4.5	ITE(7)
σ_V	km/h	0	0	10.0	Olsen (13)
σ_{TP}	s	0	0	2.0	McGee* (16)
σ_{TD}	s	0	0	5.7	McGee (16)
σ_{TM}	s	0	0	1.7	McGee (16)
SSD calculated	m	382	296	287	
SSD design	m	389	305		

* Used ratio of mean to standard deviation from observed data, also used Triggs (17).

- ITE high values (variance set to zero) compared to driver-vehicle system: DSD_H is 389 m, margin of safety is 102 m, safety index is 1.38, and chances of failure are about 4 in 10.
- ITE low values (variance set to zero) compared to driver-vehicle system: DSD_H is 305 m, margin of safety is 18 m, safety index is 0.06, and chances of failure are about 5 in 10.

PASSING SIGHT DISTANCE

Theory

Passing sight distance is associated with the design of two-lane roads and helps determine their level of service. The determination of the passing sight distance is based on observation and calculated as follows:

$$PSD_H = (V - m)t_1 + \tfrac{1}{2}\alpha t_1^2 + \tfrac{5}{3}Vt_2 + d_3 \tag{23}$$

where

PSD = passing sight distance (m),
V = speed of passing vehicle (m/sec),
m = speed difference between vehicles (m/sec),
α = acceleration of passing vehicle (m/sec²),
t_1 = preliminary delay time (sec),
t_2 = time that vehicle occupies passing lane (sec), and
d_3 = safety distance (m).

This formulation accounts for the preliminary delay distance when the faster vehicle must decide to pass the slower, the overtaking distance, a safety distance between the faster vehicle and the approaching vehicle, and the distance traveled by the approaching vehicle during much of the maneuver. Assuming all the variables are independent, random, normal variables results in the expected value and variance, as follows:

$$E\langle PSD_v\rangle = (V_v - m_D)t_1 + \tfrac{1}{2}\alpha t_1^2 + \tfrac{5}{3}V_D t_2 + \tfrac{1}{2}\alpha\sigma_{t1}^2 + d_3 \tag{24}$$

$$\mathrm{Var}\langle \mathrm{PSD}_D \rangle = (V_D - m_D + \alpha t_1)^2\sigma_{t1}^2 + (\tfrac{5}{3}V_D)^2\sigma_{t2}^2 + (\tfrac{1}{2}t_1^2)^2\sigma_\alpha^2 + t_1^2\sigma_{mD}^2 + (t_1 + \tfrac{5}{2}t_2)^2\sigma_{VD}^2 + \sigma_{d3}^2 \quad (25)$$

The margin of safety and safety index for passing sight distance are estimated by

$$M(\mathrm{PSD}) = \mathrm{PSD}_H - E\langle \mathrm{PSD}_D \rangle \quad (26)$$

$$\beta\ (\mathrm{PSD}) = \frac{\mathrm{PSD}_H - E\langle \mathrm{PSD}_D \rangle}{\sqrt{\mathrm{Var}\langle \mathrm{PSD}_D \rangle}} \quad (27)$$

The value of the safety index comes from a nonlinear combination of variables, and the probability of failure must be estimated by methods given by Ang and Tang (*9*).

Estimated Values

Values from Table 4 and Equations 24 through 27 are used to produce the following results. The mean and variance of the various times are estimated from data in Lay (*18*) and Hobbs and Richardson (*19*). The passing speed for design was set at the highway's design speed. The average for the passed vehicle was set lower. According to Hobbs and Richardson (*19*), the passing sight distance specified by design is able to accommodate 95 percent or more of all the passing operations. The safety margin and safety index are calculated using the passing sight distance recommended for design purposes.

- ITE high values (variance set to zero) compared to driver-vehicle system: PSD_H is 640 m, margin of safety is 102 m, safety index is 1.28, and chance of failure is about 1 in 10.
- ITE low values (variance set to zero) compared to driver-vehicle system: PSD_H is 305 m, margin of safety is −233 m, and safety index has a value of −4.07, which implies that it is inadequate about 99 times out of 100.

TABLE 4 VALUES FOR PASSING SIGHT DISTANCE (PSD) VARIABLES

Variable	Unit	Highway		Driver/Vehicle	
		High	Low		Source
V	km/h	95	88	80	Hobbs (19)
m	km/h	16	16	16	Hobbs (19)
α	g	0.19	0.19	0.15	Hobbs (19)
t_1	s	4.3	4.3	4.3	Hobbs (19)
t_2	s	10.7	10.7	10.0	Rockwell (20)
d_3	m	76	76	76	
σ_V	km/h	0	0	8.0	Olsen (13)
σ_m	km/h	0	0	4.0	estimated*
σ_α	g	0	0	0.04	estimated
σ_{t_1}	s	0	0	1.0	estimated
σ_{t_2}	s	0	0	1.0	estimated
σ_{d_3}	m	0	0	20	estimated
PSD calculated	m	671	616	538	rounded
PSD design	m	640	305		

* The estimated values come from a general reading through the Human Factors literature, Rockwell (20). The low value is used for pavement markings. Additional information may be found in Ohene (21).

VERTICAL CURVE

Theory

The design of vertical curves is based on stopping sight distance as determined by eye and object height for crest curves and headlight beam for sag curves. The following formulation is for crest curves with the stopping sight distance shorter than the length of the vertical curve; the curvature is estimated by the factor K_H, given as

$$K_H = \frac{(\mathrm{SSD}_H)^2}{100F^2} \quad (28)$$

where

$F = (2h_1)^{1/2} + (2h_2)^{1/2}$,
h_1 = eye height (m), and
h_2 = object height (m).

Given that all the variables are normal, independent, and random, the expected value and variance are as follows:

$$E\langle K_D \rangle = \frac{S_D^2}{100}F^{-2} + \frac{2}{100}F^{-2}\sigma_{sD}^2 + \frac{S_D^2}{100}F^{-3}[3F^{-1}(2h_1)^{-1/4} + 2(2h_1)^{3/2}]\sigma_{h1}^2 + \frac{S_D^2}{100}F^{-3}[3F^{-3}(2h_2)^{-1/4} + 2(2h_2)^{-3/2}]\sigma_{h2}^2 \quad (29)$$

$$\mathrm{Var}\langle K_D \rangle = \left[\frac{S_D^2}{100}F^{-3}(2h_1)^{-1/2}\right]^2\sigma_{h1}^2 + \left[\frac{S_D^2}{100}F^{-3}(2h_2)^{-1/2}\right]^2\sigma_{h2}^2 + \left[2\frac{S_D^2}{100}F^{-2}\right]^2\sigma_{sD}^2 \quad (30)$$

The margin of safety for the crest vertical curve is

$$M(K) = K_H - E\langle K_D \rangle \quad (31)$$

The following safety index must be evaluated by the methods given by Ang and Tang (*9*), because the basic relationship is not a linear combination of variables.

$$\beta\ (k) = \frac{K_H - E\langle K_D \rangle}{\sqrt{\mathrm{Var}\langle K_D \rangle}} \quad (32)$$

Estimated Value

The values from Table 5 and Equations 29 through 32 are used to produce the following results. The mean and variance

TABLE 5 VALUES FOR VERTICAL CREST CURVE VARIABLES

Variable	Unit	Highway		Driver/Vehicle	
		High	Low		Source
S	m	eqn 5	eqn 5	eqn 6	
h_1	m	1.07	1.07	1.07	AASHTO (5)
h_2	m	0.15	0.15	0.15	AASHTO (5)
σ_s	m	0	0	eqn 7	
σ_{h1}	m	0	0	0.007	ITE'65 (22)
σ_{h2}	m	0	0	0.01	estimated*
K calculated	m	88	61	47.2	
K design	m	95	58		

*The estimated values come from a general reading of the driver, Chapter 3, Figure 3.6 of the Traffic Engineering Handbook, Washington, D.C., 1965 (22).

eye height are estimated from data from ITE (*22*). The safety margin and safety index are calculated using the stopping sight distance recommended for design purposes.

- AASHTO high values (variance set to zero) compared to the driver-vehicle system: K_H is 95, margin of safety is 48, safety index is 2.72, and chances of failure are about 3 in 1,000.
- AASHTO low values (variance set to zero) compared to driver-vehicle system: K_H is 58, margin of safety is 11, safety index is 0.49, and chances of failure are about 3 in 10.

The margin of safety in this context is not as intuitive as that for stopping sight distance, but the safety index when stated as a chance of failure is easily understood.

RESULTS

The preceding calculations are summarized in Table 6. The standard design parameter values supplied by the highway are given in the first column. The expected demands by the driver-vehicle system are in the second column. The difference between the highway and driver-vehicle system is the margin of safety. An estimate of the safety index for the isolated components is given in the last column (chance of failure).

The margin of safety is a convenient safety measure for design parameters, such as stopping sight distance or radius of turn, but not for geometric elements such as vertical curvature, K. The safety index appears to be a more useful measure for comparative purposes. When stated as a probability of failure, the safety index is an effective measure. Failure is defined simply as the driver-vehicle system demand exceeding the highway's supply of a particular design parameter. In this case, the consequence of a failure may be speculated, and, given its probability, the risk may be estimated as the product of the consequence and its probability of occurrence. If all the consequences of failure are identical, then, to have a highway with uniform risk, the components would be designed to have an identical chance of failure.

Actually designing highways by a procedure that is similar to that used by structural engineers for buildings appears to be feasible. The procedure is simple in concept. If the mean, variance, and distribution of a particular driver-vehicle design parameter are known, this value can be increased to reflect uncertainty and the importance of the road link. The procedure is summarized in the design Equation 33 for parameter P.

TABLE 6 HIGHWAY AND DRIVER-VEHICLE DESIGN PARAMETERS

Design Parameter		Unit	AASHTO or ITE	Driver Vehicle	Safety Margin	Safety Index	Chance of Failure
SSD	high	m	198	137	61	1.22	1:10
SSD	low	m	160	137	23	0.42	3:10
R	truck	m	410	116	294	55	remote
	car	m	410	79	331	--	remote
DSD	high	m	389	286	102	1.35	4:10
DSD	low	m	305	286	18	0.06	5:10
PSD	design	m	640	538	102	1.28	1:10
PSD	paint	m	305	538	-233	-4.07	99:100
K	desirable		95	47	48	2.72	3:1,000
(crest)	low		59	47	11	0.46	3:10

$$\phi P_H \geq SETDetd(P_{D/v}) \quad (33)$$

where

ϕ = performance factor,
S = highway system importance,
E = exposure factor,
T = traffic mix,
D = driver mix,
e = environmental factor,
t = terrain factor, and
d = desired design or construction standard.

The value of the parameter $P_{D/v}$ is taken from observation of the driver-vehicle system. Factors alter this observed value on the basis of the strategic importance of the road, the number of road users, the types of vehicle, the quality of drivers, expected environmental conditions, the terrain, and the overall design and construction standard required. The performance factor ϕ reduces the roads' supplied characteristics to some acceptable level. A general organization of the relationship between the demanded driver-vehicle parameter and the highway supply is shown in Figure 6. The highway supply may also be subject to changes experienced over time. Also, the driver-vehicle system demand need not be independent of the supply. Most of the factors are already implicitly considered in road design. The strength of the proposed procedure is that it requires the designer to explicitly specify the factors.

The problems that remain are (a) determining that there are advantages to explicitly defining the design parameter of a highway, and (b) actually developing the basic driver-vehicle system information and the factors by which it may be modified.

REQUIRED RESEARCH

The margin of safety and safety index for isolated geometric sections of a highway can be estimated. However, basic infor-

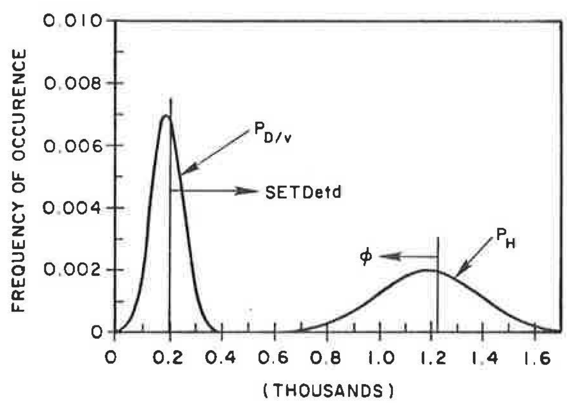

FIGURE 6 General highway design parameters (*P*).

mation on limits to various driver and vehicle performance measures is lacking. To correct this, detailed experiments and onsite observations must be undertaken to establish performance limits for normal operations, human tolerance, and vehicular road limits. Also, the statistical nature and interaction of phenomena such as operating speed, perception reaction time, vehicle deceleration rates, vehicle lateral acceleration rates, passing speeds, and others as indicated in the equations and as related to road geometry need to be studied.

In addition to these statistical distributions, consideration must be given to the appropriate values to use in the equations, as well as the acceptable safety margin and safety index for various types of roads. Finally, some method must be devised to convey the information on relative levels of safety to the various vehicle populations in operating conditions.

CONCLUSIONS

Early researchers such as Moyer and Berry (*10*) explicitly recognized the problems of margin of safety. Using the ball bank indicator, car driver reactions, and observations, they developed a procedure that provided, in their judgment, a reasonable margin of safety. Moyer and Berry and other researchers of the day were limited by instrumentation, in particular reliable accelerometers, and could not estimate the margin of safety. If they could have, they would no doubt have included variables representing the driver, the vehicle, and the road.

The margin of safety and safety index may be estimated using the methods outlined for all isolated geometric sections of a road. The equations also allow individuals or agencies to set, as a policy, the accepted chance of failure at an isolated component. Once such a policy is accepted, it is possible to calculate the correct design value, provided the demand function and supply variance are known.

Further research should be undertaken to explore the usefulness of these equations. The equations appear to hold some promise that a reasonable measure of safety may be estimated for isolated components of the road.

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Review of AASHTO Green Book Procedures for Sight Distance at Ramp Terminals

KAY FITZPATRICK AND JOHN M. MASON, JR.

Policy described in the AASHTO Green Book (1984 edition) concerning sight distance at ramp terminals is reviewed. Ramp terminal design procedures should be commensurate with design principles for at-grade intersections and required sight distance values should be similar to those for stop-controlled intersection sight distance values. However, sight distance values for ramp terminals are over 21 percent lower than those obtained from the B–1 curve procedure because of different distance-traversed assumptions. Available sight distance has been recalculated at ramp terminals assuming the Green Book values for stopping sight distance, driver eye height, and object height. Equations were developed to reproduce the B–1 curve and then truck characteristics were substituted for passenger car characteristics to determine the sight distance requirements for a stopped truck. *K* values needed to produce vertical curves that will provide the required sight distances (i.e., B–1 curve sight distances) both for passenger cars and for trucks are generally greater than Green Book *K* values used for vertical curve design. Therefore, development of alternative approaches for establishing sight distance values at ramp terminals (and at-grade intersections) should be investigated.

AASHTO's 1984 *A Policy on Geometric Design of Highway and Streets* (Green Book) (*1*) contains several procedures that can be used to determine intersection sight distance for a stopped vehicle. One procedure (Case III) is described for stop controls on secondary roads. The Green Book (*1*) states that "the driver of the vehicle on the minor road must have sufficient sight distance for a safe departure from the stopped position even though the approaching vehicle comes in view as the stopped vehicle begins its departure movements." Other procedures are described for arterial highways, railroad-grade crossings, and ramp terminals.

In the ramp terminal section, the Green Book (*1*) states that "although ramp terminals may be considered part of interchange design . . . the terminals should be planned in accordance with design principles for at-grade intersections and with particular attention to sight distance characteristics." As such, the ramp terminal procedure should produce results that are similar to Case III procedures. But sight distance values presented in Table 1 for ramp terminals are different from those found using Case III procedures (shown in Figure 1). If the ramp terminal procedure is only concerned with sight distance to the left for a left-turning vehicle, then the ramp terminal procedure should yield results similar to those produced using the B–1 curve procedure (see Figure 1). However, the ramp terminal sight distance values are over 21 percent lower than the B–1 curve values. When the ramp terminal sight distance values are compared with the values obtained using other procedures (e.g., turning vehicle attains average running speed or design speed), the differences are much greater. Using the values in Table 1 at a ramp terminal will result in sight distances that are less than the sight distances at other at-grade intersections that use Case III procedures.

Parameters are identified that are necessary to calculate both the sight distances at ramp terminals and the relevant Case III curve. An evaluation of the parameters explains the differences in the sight distance values betwen the ramp terminal and Case III, B–1 curve procedures. Development of specific equations permits the calculation of sight distances for other conditions, such as when the stopped vehicle is a truck, or when a vehicle turning onto a road must clear more than one lane.

GREEN BOOK (*1*) POLICY

Sight distance criteria for ramp terminals are intended to ensure that a vehicle stopped at the terminal will have adequate time to turn left and clear the intersection without colliding with a vehicle coming from the left. Ramp terminals should be designed on the basis of the same sight distance design elements as those used for other at-grade intersections. An added sight distance consideration is the location of bridge parapet walls or bridge railings.

The Green Book (*1*) indicates that the primary difference between this condition and the Case III–A (crossing maneuver) procedure is the increase in the time and distance traveled by vehicles negotiating the left turn rather than crossing the highway. Distances cited in the Green Book (*1*) for a turning vehicle to clear the intersection are 60 ft for the passenger car, 90 ft for the SU design vehicle, and 120 ft for the WB–50 design vehicle. Other assumptions include

- Front of the stopped vehicle is 10 ft from the edge of the through pavement (i.e., D = 10 ft);
- Turning vehicle follows its minimum turning path;
- Turning vehicle enters a two-lane, two-way roadway;

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TABLE 1 REQUIRED SIGHT DISTANCE ALONG THE CROSSROAD AT TERMINALS OF RAMPS AT INTERCHANGES [GREEN BOOK (*1*), TABLE IX-9]

Assumed Design Speed on Major road	Sight Distance Required to Permit Design Vehicle to Turn Left from Ramp to Crossroad[a]			Sight Distance Available to Entering Vehicle When Vertical Curve on Crossroad is Designed for Stopping Sight Distance[b]	
	Design Vehicle Assumed at Ramp Terminal				
(mi/h)	P (ft)	SU (ft)	WB-50 (ft)	P (ft)	SU or WB-50 (ft)
30	320	460	620	310	350
40	420	610	820	420	480
50	530	760	1,030	540	600
60	630	910	1,230	730	820
70	740	1,060	1,430	920	1,040

[a]Sight distance measured from height of eye of 3.50 ft for P, SU, and WB-50 design vehicle to an object 4.25 ft high.

[b]Minimum available stopping sight distance based on the assumption that there is no horizontal sight obstruction and that S<L.

- Time to accelerate can be determined from Figure 2; and
- Perception and preparation time is 2.5 sec.

Figure 3 shows the sight distance at ramp terminals. The Green Book (*1*) criteria indicate that both the horizontal sight triangle (Figure 3a) and the vertical curvature (Figure 3b) should be checked to ensure that the required critical sight distance from Table 1 is provided. Further, the privileged vehicle (traveling unimpeded) must have adequate stopping sight distance to a vehicle stopped at the ramp terminal.

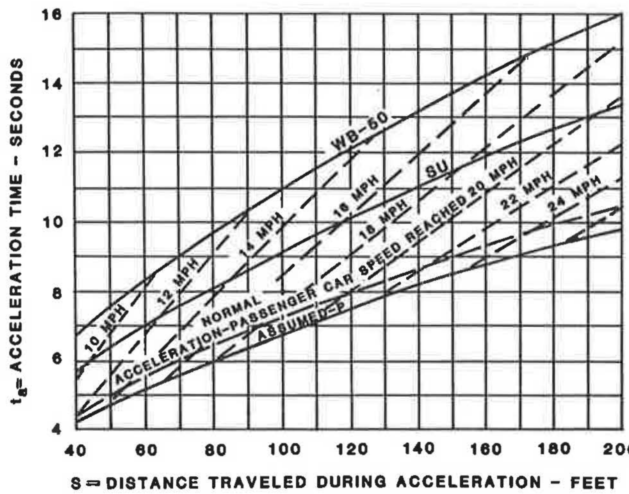

FIGURE 2 Sight distance at intersections [Case III, acceleration from stop, Green Book (*1*), Figure IX-21].

REPRODUCTION OF GREEN BOOK (*1*) SIGHT DISTANCE VALUES

Vehicle Acceleration From a Stopped Position

A major influence on sight distance is the acceleration capability of the vehicle. Two figures in the Green Book (*1*) can be consulted for vehicle acceleration information. Figure 2 shows time-distance curves depicting passenger car normal acceleration and the recommended assumed acceleration for

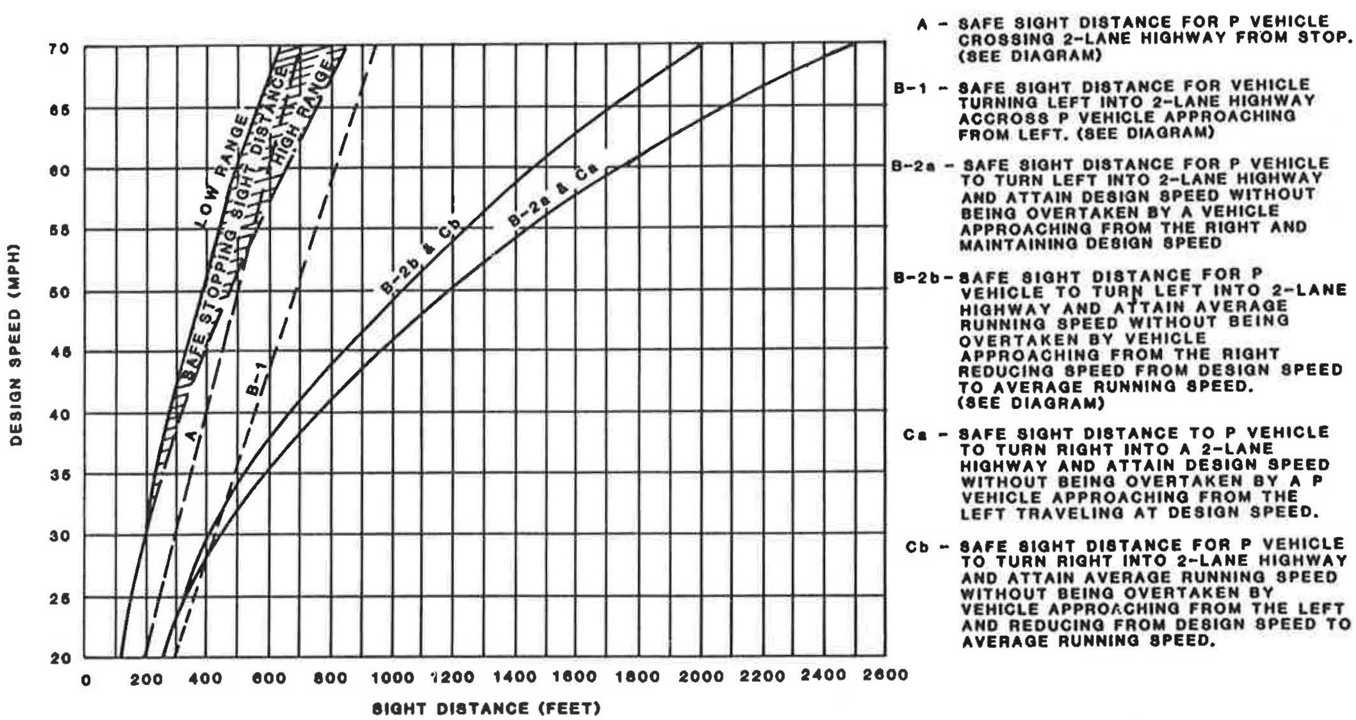

FIGURE 1 Intersection sight distance at at-grade intersections [Case III, B and C, Green Book (*1*), Figure IX-27].

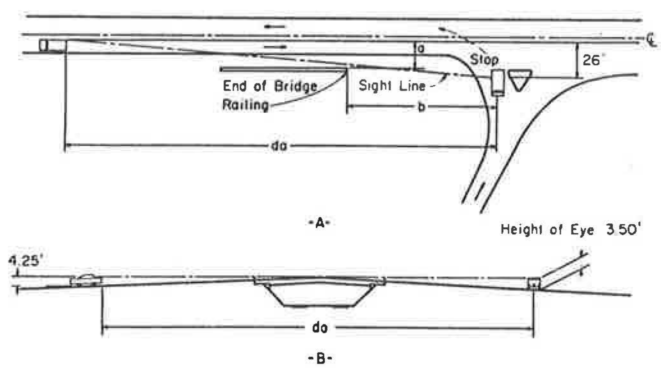

FIGURE 3 Measurement of intersection sight distance at ramp terminals [Green Book (*1*), Figure IX-29].

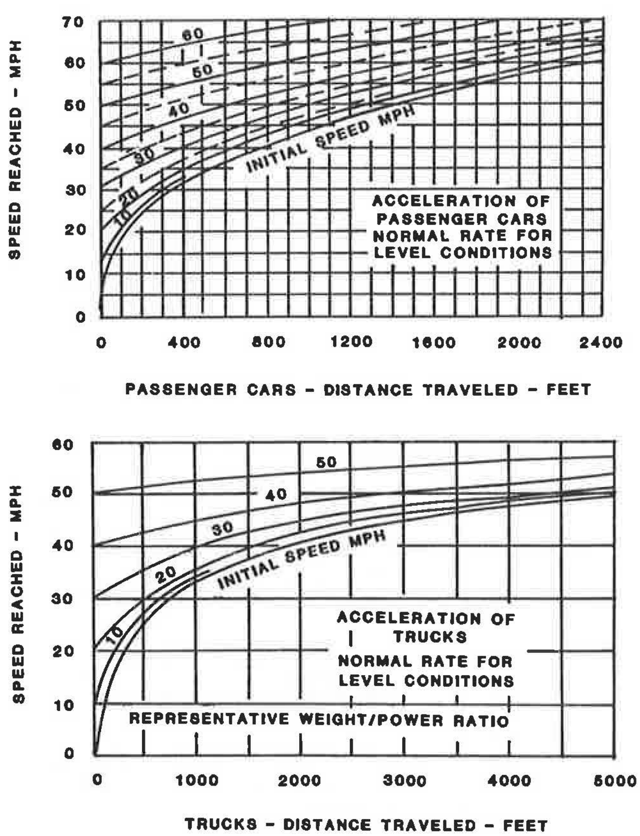

FIGURE 4 Acceleration curves [Green Book (*1*), Figure IX-22].

passenger cars, SU trucks, and WB-50 trucks. These curves represent 40- to 200-ft distances traveled during acceleration. Figure 4 presents speed versus distance curves for passenger car normal-rate acceleration for level conditions from 0 to 2,400 ft and "normal rate for level conditions, representative weight-power ratio" truck acceleration from 0 to 5,000 ft. Truck acceleration curves in Figure 4 represent SU and WB-50 design vehicles and were determined from truck performance studies.

Acceleration times can be estimated on the basis of Figure 4. Table 2 presents the distance values read from the figure in 5-mph increments. The distance increment column is the distance traveled during the previous 5-mph increase. The time increment column was calculated using the following equation:

$$\text{Time Increment} = \frac{2 \times \text{Distance Increment}}{V_{\text{initial}} + V_{\text{final}}} \tag{1}$$

where V_{initial} and V_{final} are the vehicle's initial and final speeds, respectively. Acceleration time is the sum of the previous time increments. An inherent assumption in using either Figure 2 or Figure 4 is that the acceleration rates for right- and left-turning maneuvers are not significantly different.

Sight Distance at Ramp Terminals

The Green Book (*1*) states that "the length of highway open to view from the ramp terminal must be greater than the product of the speed of a vehicle on the crossroad and the time necessary for the vehicle entering the crossroad from a stopped position on the ramp to start and complete a left turn onto the crossroad." On the basis of this statement and assuming that the procedure is only concerned with vehicles approaching from the left, passenger car results should be similar to the Green Book (*1*) B-1 curve in Figure 1 (the sight distance to clear the near lane). B-1 sight distance values should be slightly less than the ramp terminal sight distances because the B-1 procedure assumes a perception-reaction time that is 0.5 sec less than the ramp terminal procedure. The B-1 curve procedure assumes a 2.0-sec perception time and time (*J*) required to actuate the clutch or automatic shift,

TABLE 2 ACCELERATION DISTANCE AND TIME VALUES ON THE BASIS OF FIGURE 4

Speed (mi/h)	Distance Traveled (ft)[a]	Distance Increment (ft)	Time Increment (sec)	Acceleration Time (sec)
Passenger Cars				
5	10	10	2.72	2.72
10	30	20	1.81	4.54
15	50	20	1.09	5.62
20	125	75	2.92	8.54
25	210	85	2.57	11.11
30	350	140	3.46	14.57
35	550	200	4.19	18.76
40	800	250	4.54	23.29
45	1,075	275	4.40	27.70
50	1,400	325	4.65	32.35
55	1,800	400	5.18	37.53
60	2,300	500	5.92	43.45
65	2,900	600	6.53	49.98
70	3,600	700	7.05	57.03
Trucks				
5	50	50	13.61	13.61
10	100	50	4.54	18.14
15	175	75	4.08	22.22
20	275	100	3.89	26.11
25	450	175	5.29	31.40
30	700	250	6.18	37.58
35	1,200	500	10.47	48.05
40	1,900	700	12.70	60.75
45	3,200	1,300	20.81	81.56
50	5,000	1,800	25.78	107.34

[a]Values read from figure 4.

whereas the ramp terminal procedure assumes a perception and preparation time t_{pp} of 2.5 sec. However, the sight distances at a ramp terminal are over 21 percent lower than the distances obtained using the B–1 curve (clear lane) procedure.

Using the Green Book (*1*) sight distance at ramp terminals values listed in Table 1 and the assumption of 2.5 sec for t_{pp}, the average acceleration times (t_t) were determined. On the basis of these times, the distance that the turning vehicle traveled (S_t) and the speed (V) reached by the turning vehicle were found. The turning radius (R) for the minor-road vehicle was determined assuming that the vehicle was 10 ft from the edge of the through pavement and that it followed the minimum turning path as assumed in the Green Book (*1*). The results are presented in Table 3.

Calculated distance traveled (S_t) and turning radius (R) values in Table 3 do not agree with the assumptions stated in the Green Book (*1*). Distance traveled (S_t) values that were stated in the section on sight distance at ramp terminals of the Green Book (*1*) are 60, 90, and 120 ft. Turning radii (R) based on these distances are given in Table 4. Radii calculated from the sight distance values in Table 1 (listed in Table 3) and radii calculated from the Green Book (*1*) assumed distance traveled by left-turning vehicles (listed in Table 4) are both less than the design turning radius for both left and right turns (see Table 5).

TABLE 3 CALCULATED VALUES ON THE BASIS OF TABLE 1 SIGHT DISTANCE VALUES

	P	SU	WB-50
Average acceleration time, t_t (sec)	4.7	7.8	11.5
S_t values from figure 2 (ft)	50	75	110
Speed, V, reached using figure 4 (mi/h)	15	7	11
Turning radius, R (ft)[a]	13	22	29

[a]Calculated assuming L = 19 ft for P, 30 ft for SU, and 55 ft for WB-50 vehicles, D = 10 ft, and S_t values from table.

TABLE 4 CALCULATED VALUES ON THE BASIS OF GREEN BOOK (*1*) ASSUMED DISTANCE TRAVELED VALUES

	P	SU	WB-50
Green Book assumed distance traveled, S_t (ft)	60	90	120
Acceleration time, t_t, using figure 2 (sec)	5.2	8.6	12.1
Turning radius, R (ft)[a]	20	32	35
Speed, V, reached using figure 4 (mi/h)	16	9	11

[a]Calculated assuming L = 19 ft for P, 30 ft for SU, and 55 ft for WB-50 vehicles, D = 10 ft, and S_t values from table.

TABLE 5 GREEN BOOK (*1*) ASSUMED VALUES FOR TURNING RADII

	P	SU	WB-50
Left turns (ft)[a]	40	50	60
Right turns (ft)[b]	24	42	45

[a]From Green Book Table IX-20.
[b]From Green Book Table II-2.

As such, the values in Table 1 cannot be reproduced on the basis of the information contained in the Green Book (*1*). Sight distances using assumed distance traveled (S_t) values listed in the Green Book (*1*) (60, 90, and 120 ft) produce sight distance values that are between 5 and 11 percent greater than the values listed in Table 1. When a perception and preparation time t_{pp} of 2.0 sec instead of 2.5 sec is used, the calculated sight distances are less than 4 percent different from the values listed in Table 1.

Sight distance at ramp terminals should be similar to the B–1 curve (clear lane) procedure for sight distance to a vehicle approaching from the left. The Green Book (*1*) does not provide the necessary equations and parameter values for the B–1 curve (clear lane) procedure to generate sight distance values for trucks. The following section contains the equations and passenger car characteristics needed to reproduce the B–1 curve. Truck characteristics can then be substituted for the *P* values to determine the sight distance for trucks.

Green Book (*1*) B–1 Curve or Clear Lane (CL) Procedure

The B–1 curve procedure is used to establish the sight distance to be provided for a passenger car turning left onto a two-lane highway when a vehicle is approaching from the left as shown in Figure 5. Sight distance is the product of the major road vehicle speed and the turning vehicle's acceleration time needed to clear the near lane. The following equations were used to reproduce the curve:

$$ISD_{B-1} \text{ or } ISD_{CL} = 1.47Vt \quad (2)$$

$$t = t_S + J \quad (3)$$

$$S_t = D + L + W_t \quad (4)$$

$$W_t = \pi \times R/2 \quad (5)$$

where

ISD_{B-1} or ISD_{CL} = sight distance along the major-roadway's near lane to the left for left turns (ft) (see Figure 5);
V = velocity of major-road vehicle (mph);
t = time for a stopped minor-road vehicle to initiate the turn and clear the near lane (sec);
J = sum of the perception time and the time required to actuate the clutch or actuate an automatic shift (sec);
S_t = distance that the turning vehicle must travel to clear the near lane (ft);
t_S = time required by the turning vehicle to accelerate and traverse the distance (S_t) to clear the near lane (sec) (data available from Figure 4 and Table 1);
D = distance from the near edge of major-road travel lanes to the front of a stopped vehicle (ft);

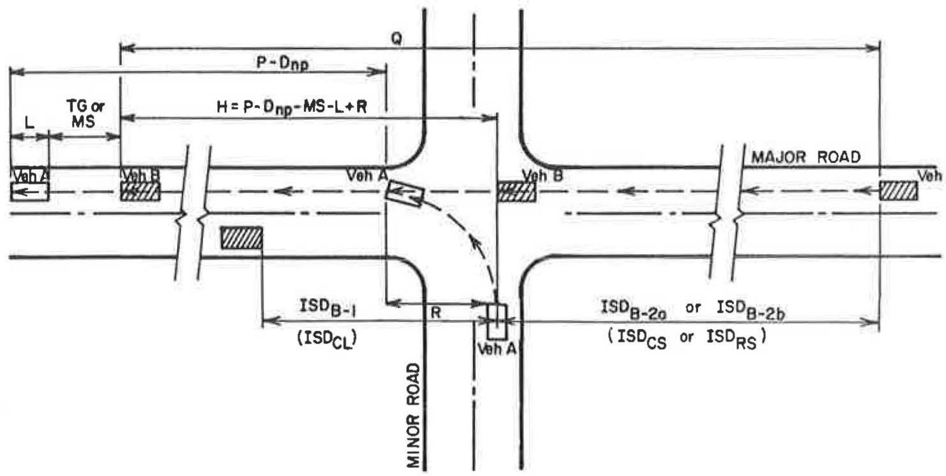

FIGURE 5 Distances considered in a left-turn maneuver [modification of Green Book (*1*), Figure IX–24].

W_t = pavement traversed along path of turning vehicle (ft);
L = length of minor road vehicle (ft); and
R = radius of left turn for minor-road vehicle (ft).

Values from the B–1 curve were used to calculate acceleration times (t_t). Assuming a perception-reaction time J of 2.0 sec, the t_t values averaged 7.4 sec. Using data derived from Figure 4, distance traveled (S_t) by a passenger car during 7.4 sec is 95 ft. Assuming a vehicle length L of 19 ft and a 10-ft distance D between the edge of the travel way and the front of the vehicle, the length of pavement W_t traversed along the turning path of the vehicle is 66 ft. The 66-ft vehicle path results in a 42-ft radius. From the Green Book (*1*), control radius R for a left-turning passenger car is 40 ft. The 42-ft radius is within an acceptable range of an assumed 40-ft radius. As such, the acceleration time (t_S) and distance (S_t) values used to generate the B–1 curve agree with Figure 4.

Table 6 presents the calculated sight distances for a passenger car, SU truck, and WB–50 truck. Except for 20 mph, the calculated passenger car sight distance values are within 5 percent of the rounded Green Book (*1*) sight distance values from Figure 1. Equations 2–5 were then used to determine sight distances for trucks. Turning radii R values selected from the Green Book (*1*) were 40, 50, and 60 ft for P, SU, and WB–50 design vehicles, respectively. The time to clear the near lane (t_l) is based on data derived from Figure 4. Sight distance for an SU design vehicle ranges from 622 ft (20 mph design speed) to 2,176 ft (70 mph design speed). Sight distances for a WB–50 design vehicle are between 687 and 2,404 ft for the same range of design speeds.

TABLE 6 CLEAR LANE (B–1 CURVE) SIGHT DISTANCE VALUES

Speed (mi/h)	B-1 Curve Figure 1 (Green Book figure IX-27) (ft)	Calculated Sight Distance		
		Passenger Car B-1-P (ft)	SU Truck B-1-SU (ft)	WB-50 Truck B-1-WB-50 (ft)
20	300	272	622	687
25	350	340	777	858
30	425	408	933	1,030
35	500	476	1,088	1,202
40	550	544	1,243	1,374
45	625	612	1,399	1,545
50	675	680	1,554	1,717
55	750	748	1,710	1,889
60	825	816	1,865	2,060
65	875	884	2,021	2,232
70	950	952	2,176	2,404

The following vehicle characteristics were used:

Characteristic	PC	SU	WB-50
Vehicle length, L (ft)	19	30	55
Turning radius, R (ft)	40	50	60
Distance to clear, S_l (ft)	92	119	159
Time to clear, t_l (sec)[a]	7.2	19.15	21.4

[a]Based on figure 4 and table 2.

Measurement of Sight Distance

The Green Book (*1*) cautions that both the horizontal sight triangle and the vertical curvature should be checked to ensure that the required sight distance is provided. Figure 3 shows the horizontal and vertical sight distance considerations. The sight line dimension along the minor road or ramp approach is 26 ft from the driver side of the major-road vehicle to what appears to be the driver of the stopped vehicle.

Distance D between the stopped vehicle and the edge of the through pavement is stated in the Green Book (*1*) section on stop control on secondary roads as being 10 ft. The section in the Green Book (*1*) on ramp terminals does not include discussions on the distance the minor-road driver is from the front of the vehicle or the distance between the edge of the travel lane and the major-road vehicle. [The section in the Green Book (*1*) on railroad grade crossings assumes a distance of 10 ft from the driver to the front of the vehicle.] If a distance of 10 ft is assumed, then the sight line dimension will be to the middle of the major-road near lane (a distance of 6 ft), rather than to the front left side of the major-road vehicle.

Available Sight Distance for Intersections

Available sight distance is the unobstructed distance along the road at which a driver can see an object. For intersections, the available sight distance is governed by the horizontal and vertical alignment and the heights of the driver's eye and the approaching vehicle. This distance can be calculated using the height of the approaching major-road vehicle and the stopped driver's eye height in the vertical curve equation with a known vertical curve length and algebraic difference in grade. Examples for a passenger car and a WB–50 truck are shown in Figures 6 and 7, respectively.

The desirable length of a vertical curve is based on stopping sight distance (SSD), algebraic difference in grade (A), driver's eye height (h_1), and object height (h_2). SSD values used for vertical curve design assume wet pavement conditions and a vehicle traveling at the design speed of the roadway. The equation for length of vertical curve is

$$L = \frac{A(\mathrm{SD})^2}{100[(2h_1)^{1/2} + (2h_2)^{1/2}]^2} \tag{6}$$

where

L = length of crest vertical curve (when $L <$ SSD) (ft);
A = algebraic difference in grade (percent);
SD = sight distance (ft) (note that wet-pavement SSD is used to determine vertical curve length);
h_1 = height of driver's eye above roadway surface (ft); and
h_2 = height of an object above roadway surface (ft).

When the driver's eye height h_1 is 3.5 ft and the object height h_2 is 0.5 ft as assumed in the Green Book (*1*), and SSD $< L$, the following formula can be used:

$$L = \frac{A(\mathrm{SSD})^2}{1{,}329} \tag{7}$$

Available sight distance can be calculated by equating Equations 6 and 7 and solving for SD. The available sight distance values listed in Table 1 should be available from the rearranged equation

$$(\mathrm{SD})^2 = \frac{100\ (\mathrm{SSD})^2\ [(2h_1)^{1/2} + (2h_2)^{1/2}]^2}{1{,}329} \tag{8}$$

Footnote (a) in Table 1 states that the driver's eye height is 3.5 ft and the object height or oncoming vehicle height is 4.25 ft. However, the values listed in Table 1 were not found using Equation 8. Because the procedure was first included in AASHO's 1965 Blue Book (*2*) and the required sight distance values did not change between editions, whereas the stopping sight distance, eye height, and object height values did change, the following equation was developed to fit the assumptions in the Blue Book (*2*):

$$(\mathrm{SD})^2 = \frac{100\ (\mathrm{SSD})^2\ [(2h_1)^{1/2} + (2h_2)^{1/2}]^2}{1{,}398} \tag{9}$$

The Blue Book (*2*) assumed that the driver's eye height was 3.75 ft for P design vehicles and 6.0 ft for SU and

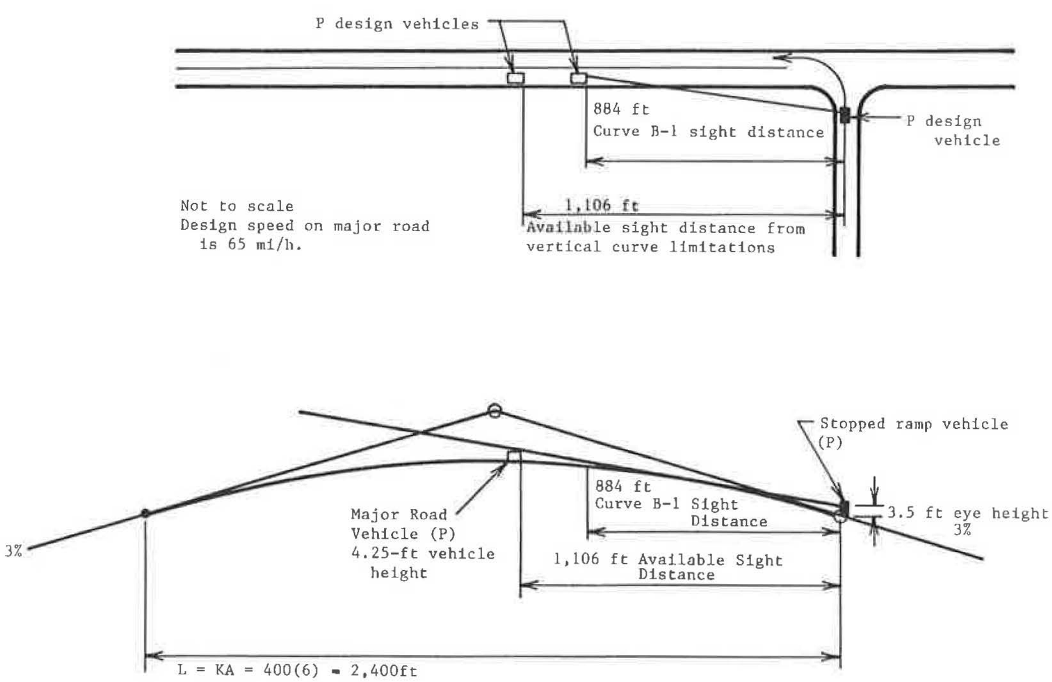

FIGURE 6 Example of sight distances at ramp terminals for a P vehicle on the minor road and a P vehicle on the major road.

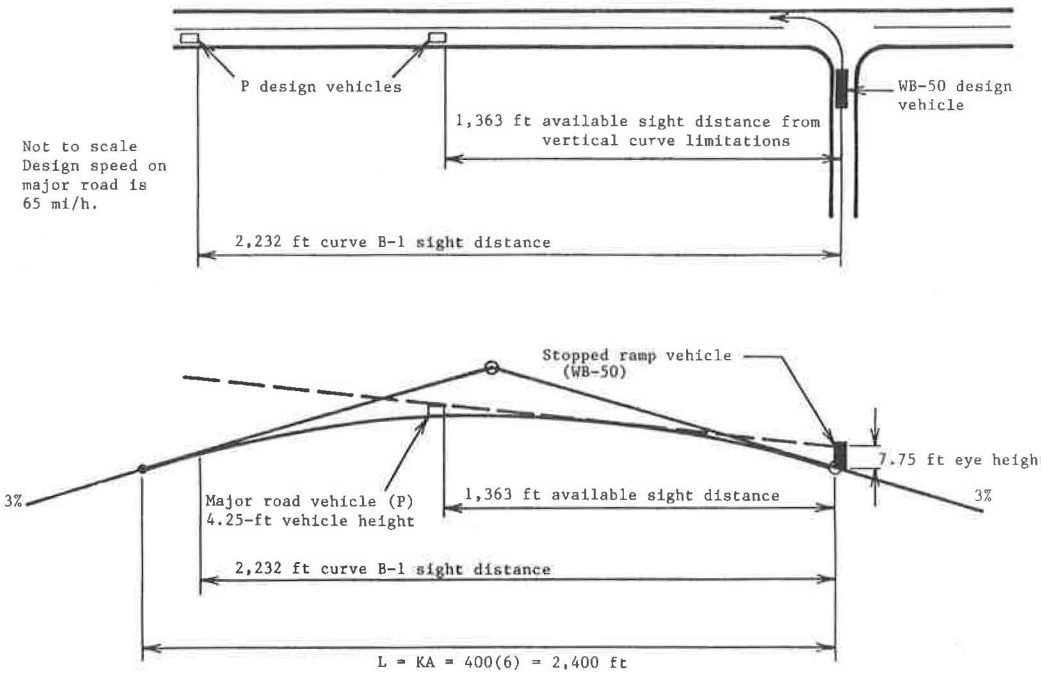

FIGURE 7 Example of sight distances at ramp terminals for a WB-50 vehicle on the minor road and a P vehicle on the major road.

WB-50 design vehicles, and that the object height was 4.5 ft. Available sight distance values in Table 1 were found using these assumptions and Equation 9. Therefore, the available sight distance values that are listed in the Green Book (*1*) failed to incorporate the revised SSD, h_1, and h_2 values.

The correct available sight distance values using the assumptions in the Green Book (*1*) (i.e., eye height h_1 of 3.5 ft for a stopped vehicle and object height h_2 of 4.25 ft) are as follows:

Design Speed (mph)	*Available Sight Distance (ft)*
30	305
40	496
50	725
60	992
70	1,297

Corrections to the available sight distance values in the Green Book (*1*) were to be made in its 1990 edition (informal communications). Nonetheless, the discussion provides a comprehensive review of Green Book (*1*), Table IX-9, which is reproduced as Table 1.

Calculated available sight distance values for P, SU, and WB-50 design vehicles are given in Table 7. Most of these values are less than the required sight distance values for the B-1 curve (clear lane) procedure listed in Table 6. Crest vertical curves would have to be designed with smaller grades to enable a ramp vehicle driver to see an oncoming major-road vehicle. The rates of vertical curvature, or K values, needed to produce the required sight distance values found in Table 6 are also presented in Table 7. A 4.25-ft object height (representing a passenger car) and 3.5, 6.0, and 7.75 ft for P, SU, and WB-50 driver's eye heights, respectively, were used in determining the K values. The 7.75-ft height represents an average driver's eye height for conventional cabs found by Middleton et al. (*3*). The 6.0- and 3.5-ft heights are from AASHTO (*1,2*).

IMPLICATION OF FINDINGS

The sight distance at ramp terminal procedure as cited in the Green Book (*1*) is intended to include the time necessary for a stopped vehicle to start and complete a left turn. The procedure does not provide additional time for the vehicle to attain any speed other than the speed achieved at the end of the turn. A P vehicle will be moving at approximately 15 mph after traveling 50 ft, an SU vehicle will be moving at 7 mph after 75 ft, and a WB-50 vehicle will be moving at 11 mph after 100 ft (see Table 3). The speed values attained indicate that sufficient sight distance for a left-turning vehicle to complete the turn and achieve running speed is not provided.

B-2a & Ca and B-2b & Cb curves (see Figure 1) represent the sight distance required for a turning vehicle to attain design speed or average running speed, respectively. The B-1 curve is the sight distance needed for a turning vehicle to clear the near lane. These values are greater than the ramp terminal sight distance values because of differences in the assumed distance traveled values. The B-1 curve procedure and vehicle characteristics available in the Green Book (*1*) resulted in values of sight distances between 933 ft (at 30 mph) and 2,176 ft (at 70 mph) for an SU vehicle and between 1,030 ft (at 30 mph) and 2,404 ft (at 70 mph) for a WB-50 vehicle.

Available sight distance values for a minor-road passenger car to a major-road passenger car traveling at 45 mph or below are less than the required sight distance values from the B-1 curve in Figure 1. On the basis of these assumptions and

TABLE 7 CALCULATED INTERSECTION SIGHT DISTANCES FOR VEHICLES STOPPED AT A RAMP TERMINAL

		Passenger Car			SU			WB-50		
Speed (mi/h)	AASHTO K	ISD Available[a] (ft)	ISD Required[b] (ft)	K[c] (ft)	ISD Available (ft)	ISD Required (ft)	K (ft)	ISD Available (ft)	ISD Required (ft)	K (ft)
30	30	305	408	54	354	933	214	376	1,030	226
35	40-50	381	476	73	442	1,088	291	470	1,202	308
40	60-80	496	544	96	575	1,243	380	611	1,374	402
45	80-120	610	612	121	708	1,399	481	752	1,545	509
50	110-160	725	680	149	841	1,554	594	893	1,717	628
55	150-220	839	748	181	973	1,710	718	1,034	1,889	760
60	190-310	992	816	215	1,150	1,865	855	1,222	2,060	904
65	230-400	1,106	884	252	1,283	2,021	1,003	1,363	2,232	1,061
70	290-540	1,297	952	293	1,504	2,176	1,163	1,598	2,404	1,231

[a]Available intersection sight distances are from the following equation (assuming SSD<L):

$$ISD^2_{available} = \frac{SSD^2\ 100\ [(2 \times h_1)^{(1/2)} + (2 \times h_2)^{(1/2)}]}{1,329}$$

where h_1 = 3.5 ft, 6.0 ft, and 7.75 ft for P, SU, and WB-50, respectively, and h_2 = 4.25 ft for P on major road.

[b]Calculated required sight distances for B-1 Curve (vehicle turns left and clears near lane).

[c]K value to provide B-1 Curve sight distance. They were derived from the equation:

$$K = \frac{ISD^2_{B\text{-}1\ Curve}}{100\ [(2 \times h_1)^{(1/2)} + (2 \times h_2)^{(1/2)}]}$$

Green Book (*1*) procedures, the minor-road vehicle does not have the intended time (as determined from Figure 4) needed to clear the near lane before the arrival at the intersection of a major-road vehicle that comes into view at the start of the turn maneuver. This situation is also present for trucks at each major-road design speed (e.g., see Figure 7). The sight distance necessary for a turning vehicle to accelerate to design speed or average running speed (B–2a & Ca and B–2b & Cb curves, see Figure 1) is significantly greater than the sight distances required from the B–1 curve procedure (for clearing the near lane). Available sight distance is much lower than the required sight distance for all vehicle types when designing for B–2a & Ca or B–2b & Cb conditions.

Minimum *K* values listed in Table 7 used to design vertical curves in the preceding situations would have to be increased to also encompass the B–1, B–2a & Ca, or B–2b & Cb curve intersection sight distance values. Table 7 only lists the B–1 curve situations. *K* values for B–2a & Ca or B–2b & Cb curves would be significantly greater. However, because intersections are generally operating safely even with significant truck traffic, the B–1 curve may be overpredicting the needed intersection sight distance for the clear-lane procedure. Operational experience at intersections indicates that sight distances as long as 2,400 ft are not typically available nor required for safe operations at intersections. Because individuals cannot perceive vehicular movement much beyond 800 ft (*4*), these results indicate that the current AASHTO procedure produces impractical values for trucks. Either the minor-road vehicles are clearing the near lane in less time than determined from Figure 4, the major-road vehicle is decelerating, or the Green Book (*1*) procedures do not represent events that are actually occurring. Consideration should be given to modifying the Green Book (*1*) procedures to better represent tʰ operations of actual intersections.

A preliminary draft copy (*5*) of the revised Green Book (*1*) was reviewed and the following changes were noted:

- Perception and preparation time t_{pp} had been reduced from 2.5 to 2.0 sec.
- Sight distance values for a P vehicle had decreased by 10 ftto 30 ft (reflecting revised P vehicle acceleration data). The sight distance values for the SU and WB–50 vehicles had not changed.
- The available sight distance values had been modified to reflect driver's eye height values of 3.5 ft for P and 8.0 ft for SU and WB–50 design vehicles, and an object height of 4.25 ft.
- Figure 3 had been redrawn showing the stopped vehicle partially into a left turn (as opposed to being perpendicular to the cross road) and with 10-ft dimensions for the distances from edge of lane to front of vehicle and front of vehicle to driver.
- A formula for the sight distance to the left value for Case III (B–1 curve) had been provided. This procedure assumes that the major-road vehicle is driving at 85 percent of the major-road design speed and that the time to accelerate is based on the distance traversed, which is the sum of a 10-ft setback from the stop line, the length of the vehicle, and the distance traveled to cross the opposing lane (assumed as approximately 1.5 times the lane width). A 2-sec gap between the turning vehicle and the approaching vehicle is also stated as being assumed.

RECOMMENDATIONS

The following issues should be considered in future versions of the AASHTO policy for stop-controlled intersection sight distance:

- Because bridge railings are a common sight distance obstruction for an at-grade intersection near interchange overpasses, the discussion regarding sight distance at ramp terminals is recommended to be retained in future versions of the AASHTO policy. However, the calculation procedure should be presented as a specific example of the clear-lane (Case III–B, B–1 curve) sight distance procedure.
- Guidance should be provided on the value for distance traveled and its relationship to the turning radius for a left-turning vehicle.

Most drivers do not have the capability to accurately judge the location and speed of an oncoming vehicle at several of the sight distances produced with the truck characteristics. Generally speaking, intersections currently operate with sight distances less than those calculated. For practical reasons, intersection sight distance procedures should reflect actual field operations. For example, individual parameter values used should represent current or future vehicle and driver characteristics, or both, which can be accomplished by explicitly considering gaps in the major-road traffic that are accepted by the minor-road driver.

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The contents of this paper reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein.

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Comparison of Operating Speeds on Dry and Wet Pavements of Two-Lane Rural Highways

RUEDIGER LAMM, ELIAS M. CHOUEIRI, AND THEODOR MAILAENDER

The impact of design parameters and traffic volume on operating speeds of passenger cars is evaluated under free-flow conditions on 322 curved roadway sections of two-lane rural highways in New York state. The design parameters considered are degree of curve, length of curve, superelevation rate, gradient, sight distance, lane width, shoulder width, posted speed, and average annual daily traffic. For the evaluation of the quantitative effects of these factors on operating speeds, expressed herein by the 85th-percentile speeds, the multiple linear stepwise regression technique was used. The various stages of analyses revealed that degree of curve was the best available single-variable predictor of operating speeds on dry pavements. Other variables helped the regression model, but the equation did well even without them. Effect of wet pavements on 85th-percentile speeds of passenger cars were also examined. Analyses were performed using data from a total of 24 curved roadway sections. Ample evidence exists to indicate that wet pavement does not have a great effect on operating speed, and that drivers will not adjust their speeds sufficiently to accommodate inadequate wet pavement on curves in particular. Furthermore, results of the statistical analyses indicate that the relationship between operating speed and degree of curve, developed from speed data collected on dry pavements, is also valid for wet pavement conditions so long as visibility is not affected appreciably by heavy rain. It is obvious that the drivers do not recognize the fact that friction supply is significantly lower on wet pavements as compared with dry. For the implementation of the results for design purposes, recommendations for achieving consistency and detecting inconsistencies in horizontal alignment, as well as recommendations for harmonizing design speed and operating speed, as related to wet pavement conditions, were given for good, fair, and poor design practices. These tasks are important in modern highway design and redesign strategies for improving traffic safety.

Weather conditions have a tendency to modify vehicular speeds because of a reduction in visibility and a possible impairment of surface conditions. The general effect of wet pavement conditions is to lower friction supply between the tire and the roadway, with the amount of reduction depending on the presence of moisture, snow, and ice, or on the thickness of the water film covering the pavement. As the thickness of the water film increases, skid resistance decreases, and, in cases of heavy rain combined or not combined with geometric design deficiencies, hydroplaning conditions may occur (*1*,*2*). Geometric design deficiencies, as well as drainage and pavement design, could affect the incidence of wet-pavement accidents, especially at curved sites. For example, when consistency between successive design elements is not present, or a harmony between design speed and operating speed does not exist at a certain curved site (*3–8*), or an adequate dynamic safety of driving cannot be provided because of a reduction in friction factors caused by wet pavement conditions (*9*), critical driving conditions may occur. Other roadway sections, which may be hazardous when roadway surfaces are wet, have insufficient minimum or maximum superelevation rates, superelevation runoffs with insufficient longitudinal slopes, or sag vertical curves that do not satisfy, for example, the drainage requirements.

According to a report prepared by the National Transportation Safety Board (NTSB) (*10*), about 13.8 percent of all fatal highway accidents are fatal accidents that occurred on highway pavements that were wet. A study cited by NTSB of wet-pavement accidents that occurred on the West Virginia highway system revealed that the average rate of wet-weather accidents was 2.2 times the rate of dry-pavement accidents and that the maximum rate was 85 times the dry-pavement accident rate. Forty percent of the accidents on the West Virginia Interstate system occurred on wet pavement. The report estimated that the roads in West Virginia were not wet more than 15 percent of the time. If analyses of national and international data reflect findings similar to those cited by NTSB (*10*), then the wet-pavement problem should be of major concern worldwide and more resources should be allocated for correcting the sources of the problem.

A review of accidents in the United States and Europe (*11*) revealed that between 25 and 30 percent of all fatalities on both continents occurred on curves of two-lane rural highways. Analyses (*3*,*4*) of accidents on two-lane rural highways have indicated

1. Fatal or injury accidents accounted for more than 70 percent of the accidents on curves, whereas property damage accidents greater than $400 represented less than 30 percent; and
2. Wet pavement conditions contributed to nearly 50 percent of the accidents on curves even though vehicle mileage driven under these conditions is much lower than that on dry pavements.

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In summary, a high fatal or injury accident risk does exist on curves of two-lane rural roads, especially under wet pavement conditions.

Because of the lower coefficient of friction on wet pavement as compared with dry, wet conditions lead more frequently to critical driving maneuvers, or even accidents (*12–15*). In this connection, speed is an additional vital contributing factor in many wet weather accidents because the value of skid resistance decreases as vehicle speed increases (*9*). For this reason, geometric design guidelines (*2,16–20*) point out that for driving dynamic considerations in horizontal and vertical alignments, design speed and tangential and side friction values should be selected on the basis of wet pavement conditions.

On the basis of experiences gained in New York state, the effects of design parameters, traffic volume, and wet pavement conditions on operating free speeds of passenger cars on curved sections of two-lane rural highways are determined. The question whether drivers recognize that wet pavements offer less skid resistance than do dry pavements and that they must adjust their speeds sufficiently to accommodate wet pavement to maintain vehicle control particularly on curves is examined in detail.

DATA COLLECTION AND REDUCTION

Data collection was broken down into three categories: first, the selection of road sections appropriate for the study; second, the collection of as much field data as possible about the road sections; and third, the measurement of operating free speeds at each section.

Selection of Appropriate Road Sections

From spring 1984 to summer 1987, two-lane rural state route sections throughout northern New York, normally consisting of a sequence of tangent to curve (or curved section) to tangent, were investigated under dry pavement conditions. Site selection was limited to sections with the following features:

1. Removed from the influence of intersections;
2. No physical features adjacent or in the course of the roadway that may create abnormal hazard, like narrow bridges;
3. Delineated and with paved shoulders;
4. No changes in pavement or shoulder widths;
5. Protected by guardrails when the height of the embankment exceeded 5 ft;
6. Grades less than or equal to 5 percent; and
7. Average annual daily traffic (AADT) between 400 and 5,000 veh/day.

The selection process attempted to maintain a regional distribution and at the same time retain the longest road segments. Road sections selected provided the widest range of changes in horizontal alignment that could be found by observation or by actual information given by the New York State Department of Transportation.

Field Data Collection

This stage involved obtaining as much data in the field as possible about the road sections and specifically about the curve or curved section within the observed road section. Information recorded were the degree of curve, length of curve, superelevation rate, gradient, lane width, shoulder width, sight distance, AADT, and posted speeds. These data were collected in the field and later compared with (or were directly obtained from) the design plans of the New York State Department of Transportation regional offices.

Speed Data Collection and Reduction

In order to ensure that the speeds measured represented the free speeds desired by the driver under a set of roadway conditions and were not affected by other traffic on the road, only the speeds of isolated vehicles with a minimum time gap of about 6 sec, or those heading a platoon of vehicles, were measured in this study. Speed measurements were made during daytime hours on weekdays under dry and, in some cases, under wet pavement conditions.

The basic method used for speed data collection involved the measurement of the time required for a vehicle to traverse a measured course laid out in the center of a curve. Additional speed measurements were often taken on preceding and succeeding tangents, or both, to the curved site. Length of the course was 150 ft. The method used for measuring time over the measured distance involved use of transverse pavement markings that were placed at each end of the course and an observer who started and stopped an electronic stop watch as a vehicle passed the markings. The observer was placed at least 15 ft from the pavement edge of the road to ensure that his presence would not influence the speeds of passing vehicles, but not too far away so as to minimize the cosine effect (*21*).

By applying this procedure, satisfactory speed data, which were occasionally substantiated by the use of radar devices, were obtained for both directions of travel. About 120 to 140 passenger cars under free-flow conditions were sampled at each site for both directions of traffic. Speed data were then used to obtain the operating speed, expressed herein by the 85th-percentile speed—that speed below which 85 percent of the vehicles travel.

METHODOLOGY

Many factors affect operating speeds on two-lane rural highways, including, but not limited to, characteristics of the site, characteristics of traffic and road users, characteristics of controls, and characteristics of variable factors (*22*).

Each of these factors can act in different and varying amounts at a given location. Therefore, the task of determining the influence of each on operating speeds becomes difficult.

Only the effect of the following parameters on operating speeds will be addressed: degree of curve, length of curve, lane width, shoulder width, superelevation rate, sight distance, gradient, posted recommended speed, and AADT.

For evaluation of the quantitative effects of design and traffic parameters, the multiple linear stepwise regression technique (Max R^2 improvement technique) was used (*4*). The stepwise technique consists of adding one independent variable to the regression equation in each step. Thus, the stepwise process produces a series of multiple regression equations in which each equation has one independent variable more than its predecessor in the series (*23,24*). The following stipulations were used to terminate the stepwise process and to determine the final multiple regression equation:

1. The selected equation has to have a multiple regression coefficient R^2 that is significant at the 0.05 level;
2. Each of the independent variables included in the multiple regression equation has to have a regression coefficient that is significantly different from zero at the 0.05 level; and
3. None of the independent variables included in the multiple regression equation are highly correlated with each other. The superelevation rate and posted speed are withheld from any subsequent regression analysis because they are highly correlated with degree of curve.

The selected multiple regression equation had to fulfill all three stipulations. In addition, the following conditions were assumed to hold:

1. Degree of curve was taken as positive whether a curve turned left or right;
2. An uphill gradient was treated as positive, whereas a downhill gradient was treated as negative; and
3. When no advisory speed signs (recommended speeds) were posted in curves, the nationwide speed limit of 55 mph was taken into consideration.

OUTCOME OF THE DATA ANALYSES

Relationships Between Variables

The analysis is based on data collected for 322 curved roadway sections under dry pavement conditions in New York state (*3,4*).

Various stages of regression analyses found that the most successful equation for explaining much of the variability in 85th-percentile speeds, in terms of statistical significance and overall form, is as follows:

Overall Equation

$$V85 = 34.700 - 1.005(\mathrm{DC}) + 2.081(\mathrm{LW}) + 0.174(\mathrm{SW}) + 0.0004(\mathrm{AADT}) \quad (1)$$

$R^2 = 0.842$
$SEE = 2.814$ mph

where

$V85$ = estimate of the operating speed expressed by the 85th-percentile speed (mph),
DC = degree of curve (range 0° to 27°),
LW = lane width (ft),
SW = shoulder width (ft),
AADT = average annual daily traffic (vpd),
R^2 = coefficient of determination, and
SEE = standard error of estimate (mph).

This small value of SEE (2.814 mph) and large R^2 (0.842) suggest that the relationship represented by Equation 1 is a strong one.

Design parameters, sight distance, length of curve, and gradient were not included in the regression model because the regression coefficients associated with these parameters were not significantly different from zero at the 95 percent level of confidence.

However, in comparing Equation 1 with the following reduced Equation 2, which only includes the design parameter DC, note from the coefficients of determination (R^2) that the influence of LW, SW, and AADT in Equation 1 explains only about an additional 5.5 percent of the variation in the expected operating speeds.

Reduced Equation

$$V85 = 58.656 - 1.135(\mathrm{DC}) \quad (2)$$

$R^2 = 0.787$
SEE = 3.259 mph

This small value of SEE (3.259 mph) and moderately large R^2 value (0.787) suggest that the relationship represented by Equation 2 is also strong, and can be considered a competitor to the relationship represented by Equation 1.

Comparison of Operating Speeds on Dry and Wet Pavements

From the current data base, 24 sites were selected to provide horizontal curves of various degrees. For instance, degree of curve varied from 0° to 27°. Grades were level or nearly level on the curved sites and for a considerable distance before and beyond, which minimized the effect of grades on operating speeds of vehicles for the following comparisons.

Observations on wet pavements were taken several weeks after speeds on dry pavements were collected. On all occasions, the surfaces were wet and rain was falling from a sprinkle to moderately heavy rain. On no occasion did it rain so hard as to affect visibility appreciably. Minimum sight distances of about 450 to 550 ft, which represent the limiting values of stopping sight distances for a design speed of 55 mph (*2*), had to be provided. This limitation is based on the results of research (*25*), which indicated that when sight distance is affected appreciably because of heavy rain, drivers tend to reduce their speeds. In other words, drivers reduce their speeds not so much on account of the danger created by lower skid resistance values on wet pavements, but rather because of limited sight distances.

Available sight distances in this study were determined by a member of the study team who drove through the study section at different fixed-time intervals. When passing a specified marked mile-marker, the driver had to clearly see a mile-marker located at a distance of 0.1 mi, or about 500 ft. If this was not possible because of impaired visibility caused by heavy rain, the driver had to inform the observer taking the measurements by waving a red flag to indicate "stop wet measurements," or a green flag to indicate "continue wet measurements." Markings were done by setting up reflectors at the mile-marker positions within the section under study.

The number of vehicles recorded varied considerably from site to site as the wet pavement studies were dependent on continued rain. Again, only the speeds of passenger cars under free-flow conditions, with a minimum time gap of at least 6 sec between successive vehicles, were recorded. An example of how the data were tabulated is presented in Table 1 with the number of free-moving vehicles and corresponding 85th-percentile speeds for both wet and dry pavement conditions and with respect to degree of curve and daily precipitation in inches.

Cumulative speed distribution curves were plotted for each location studied from these data. Figure 1 shows a typical example of the distribution of speeds on dry and wet pavements. The most notable feature of the speed data used is that there was more or less a little difference in the speed distributions of free-moving passenger cars on dry and wet pavements for all curved sites under study. Similar results were shown by Stohner (*26*).

The averages of operating speeds (expressed by the 85th-percentile speeds) on dry and wet pavements for the 24 curved road sections were 47.54 and 47.41 mph, respectively, for both directions of traffic. The observed decrease in 85th-percentile speeds caused by wet road surfaces is only 0.27 percent. However, these computations do not reflect any statistical significance.

Operating speeds on dry and wet pavements were additionally examined for each of the 24 test sections. Data were plotted in Figure 2 with the dry speeds on the *y*-axis and the wet speeds on the *x*-axis. For each test section, (a) if the operating speed on wet pavement was identical to the operating speed on dry pavement, the data point would fall on the hypothetical 45-degree diagonal line shown, (b) if the operating speed on wet pavement was greater than the operating speed on dry pavement, the data point would fall below the line, and (c) if the operating speed on dry pavement was greater than the operating speed on wet pavement, the data point would be above the line. As shown in Figure 2, the data are random with points distributed both above and below the hypothetical 45-degree line.

In order to determine whether or not operating speeds on dry pavements were significantly different from operating speeds

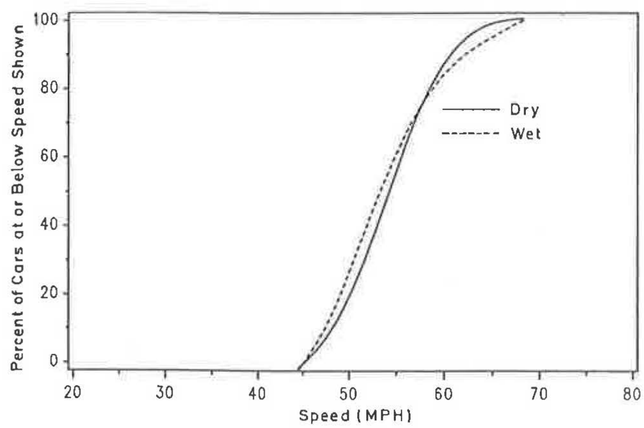


FIGURE 1 Typical example illustrating the distribution of speeds on dry and wet pavements on SR 3, county 7302, between Mile Markers 3187 and 3193.

TABLE 1 EXAMPLE FOR ALIGNMENT DATA AND OPERATING SPEEDS ON DRY AND WET PAVEMENTS (NEW YORK STATE)

	SPOT OF SPEED MEASUREMENT				OPERATING SPEEDS ON DRY PAVEMENTS		OPERATING SPEEDS ON WET PAVEMENTS		
ROUTE NUMBER	MILE-MARKER	COUNTY NUMBER	DEGREE OF CURVE	SUPER-ELEVATION	NO. OF CARS	85 % SPEED	NO. OF CARS	PRECIPI-TATION	85 % SPEED
			DEG./100 FT	PERCENT		MPH		INCHES	MPH
3	3187-	7302	2.5	2.0	57	59.3	43	0.08	59.9
	3193		2.5	2.0	54	59.4	58	0.08	58.6
3	3245-	7302	5.0	3.0	62	57.4	44	0.01	58.5
	3248		5.0	3.0	56	58.3	50	0.01	58.2
11B	1069-	7501	3.5	2.5	60	59.2	44	0.11	59.1
	1073		3.5	2.5	73	57.2	48	0.11	56.0
37	1076-	7301	0.0	--	56	59.7	54	0.04	56.5
	1082		0.0	--	65	55.5	50	0.04	57.2
37	1541-	7502	1.0	1.5	55	58.0	52	0.08	56.0
	1546		1.0	1.5	55	56.0	55	0.08	56.6
37	1152-	7203	2.3	1.5	52	52.3	52	0.04	50.6
	1156		2.3	1.5	71	48.5	62	0.04	53.3
37	1053-	7502	3.0	2.5	66	56.9	53	0.08	56.1
	1056		3.0	2.5	67	59.4	46	0.08	56.4
37	1206-	7301	3.2	2.0	48	53.1	56	0.04	57.6
	1214		3.2	2.0	57	56.7	54	0.04	54.4
37	1157-	7203	8.2	5.0	68	47.5	72	0.04	51.1
	1159		8.2	5.0	64	53.5	52	0.04	50.4
56	1295-	7501	1.5	2.0	38	56.2	70	0.05	57.2
	1299		1.5	2.0	34	57.3	75	0.05	58.3
56	1261-	7501	4.0	3.5	105	54,2	54	0.05	56.1
	1264		4.0	3.5	80	56.3	53	0.05	56.1
56	1452-	7501	5.5	3.5	73	47.6	75	0.05	47.3
	1454		5.5	3.5	81	46.1	61	0.05	47.2

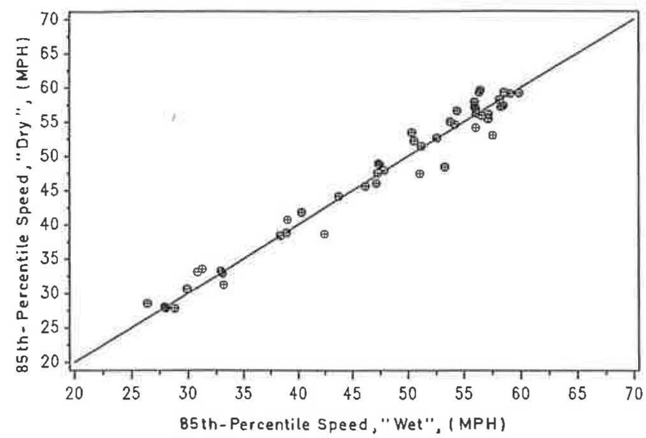


FIGURE 2 Plot relating 85th-percentile speeds on dry pavements to those on wet pavements.

TABLE 2 CRITICAL VALUES (D_c) IN THE KOLMOGOROV-SMIRNOV TWO-SAMPLE TEST (*28*)

Level of Significance	Value of D so large as to call for rejection of H_0 at the indicated level of significance, where D = maximum $\lvert S_{n_1}(X) - S_{n_2}(X)\rvert$
.10	$1.22 \cdot \sqrt{\frac{n_1 + n_2}{n_1 \cdot n_2}}$
.05	$1.36 \cdot \sqrt{\frac{n_1 + n_2}{n_1 \cdot n_2}}$
.025	$1.48 \cdot \sqrt{\frac{n_1 + n_2}{n_1 \cdot n_2}}$
.01	$1.63 \cdot \sqrt{\frac{n_1 + n_2}{n_1 \cdot n_2}}$
.005	$1.73 \cdot \sqrt{\frac{n_1 + n_2}{n_1 \cdot n_2}}$
.001	$1.95 \cdot \sqrt{\frac{n_1 + n_2}{n_1 \cdot n_2}}$

on wet pavements, the Kolmogorov-Smirnov (K–S) two-sample statistical test was used because a priori knowledge of specific population distribution characteristics is not required. This test is performed by statistically examining the cumulative frequency distributions of both the dry and wet test data. The K–S test will detect changes (or differences) in the shape of the distributions (e.g., skewed left to skewed right and bell shaped to skewed) as well as shifts in central tendency without shifts in shape (*27–29*).

The null hypothesis (H_0) tested with the speed data was as follows: "There is no significant difference between the distribution of operating speeds on dry and wet pavements." The test was conducted at the 95 percent level of confidence.

For the operating speed data, (a) if the two samples have in fact been drawn from the same population distribution, then the cumulative distributions of both samples may be expected to be fairly close to each other in as much as they both should show only random deviations from the population distribution, and (b) if the two sample cumulative distributions are too far apart at any point, the samples may come from different populations. Thus, a large enough deviation between the two sample cumulative distributions is evidence for rejecting the null hypothesis (*27–29*).

In order to apply the K–S two-sample test, a cumulative frequency distribution is made for each sample of observations using the same intervals for both distributions. For each interval, then, one step function is subtracted from the other. The test focuses on the largest of these observed deviations.

Let $S_{n1}(X)$ be the observed cumulative step function for the sample corresponding to dry, that is, $S_{n1}(X) = K/n_1$, where K is the number of observations equal to or less than X. Let $S_{n2}(X)$ be the observed cumulative step function of the sample corresponding to wet, that is, $S_{n2}(X) = K/n_2$. Now the K–S two-sample test focuses in the one-tailed test (in the predicted direction) on

$$D = \text{Max}\ (S_{n1}(X) - S_{n2}(X)) \tag{3}$$

and in the two-tailed test (irrespective of direction) on

$$D = \text{Max}\ |S_{n1}(X) - S_{n2}(X)| \tag{4}$$

In the one-tailed test, the alternative hypothesis (H_1) is that the population values from which one of the samples was drawn are stochastically larger than the population values from which the other sample was drawn, whereas in the two-tailed test, H_1 is simply that the two samples are from different populations.

An example application of the results of the K–S test of operating speeds on dry versus wet pavements follows.

Set the number of pairs (n_1, n_2) of operating speed values on dry and wet pavements (for both directions of traffic) equal to 48 and the level of significance $\alpha = 0.05$. In the null hypothesis (H_0), the operating speeds on dry and wet pavements do not differ. However, for the alternative hypothesis (H_1) the operating speeds on dry and wet pavements do differ. Because H_1 does not state the direction of the predicted differences, the region of rejection is two-tailed (Equation 4). For the sampling distribution, Table 2 presents the critical values (D_c) of (D) in the K–S two-sample statistical test. If $D > D_c$, then the distribution of one group of speed data is significantly different from the distribution of the second group. Table 3 indicates that the largest discrepancy between the two (dry and wet) cumulative frequency distributions is $D = 2/48 = 0.0417$, in the operating speed range of 46 to 50 mph. Reference to Table 2 reveals that when the number of pairs (n_1, n_2) of operating speed values is 48, $D_c = 0.2776$ for level of significance $\alpha = 0.05$. Therefore, because $D = 0.0417 < D_c = 0.2776$, it can be concluded that operating speeds on dry pavements are not statistically different from operating speeds on wet pavements, at least on the basis of this research investigation.

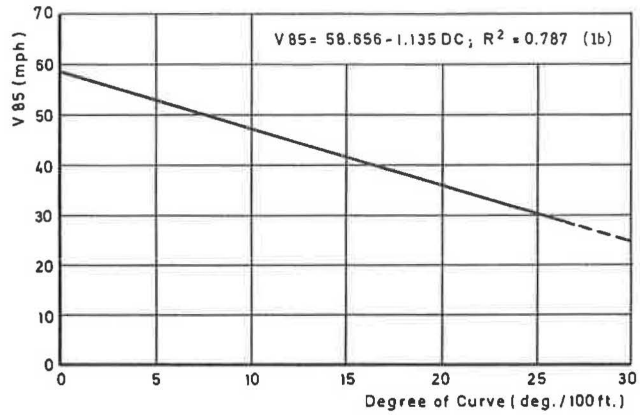
From the statistical tests, the relationship between operating speed and degree of curve (Equation 2) is valid both for dry and wet pavements so long as the visibility is not appreciably affected by heavy rain. This conclusion is substantiated by the results of a study conducted in the Federal Republic of Germany (*25*), which found that a significant decrease in operating speeds was observed only for high precipitation levels combined with highly reduced visibility conditions.

Ample evidence exists to indicate that wet pavement does not have a great effect on operating speed and that drivers

TABLE 3 RESULTS OF THE KOLMOGOROV-SMIRNOV TEST

OPERATING SPEED RANGES (mph)	$S_1(X)$	$S_2(X)$	$\|S_1(X) - S_2(X)\|$
> 26-30	4/48	5/48	1/48
> 30-34	10/48	10/48	0/48
> 34-38	10/48	10/48	0/48
> 38-42	15/48	14/48	1/48
> 42-46	17/48	16/48	1/48
> 46-50	24/48	22/48	2/48
> 50-54	29/48	29/48	0/48
> 54-58	42/48	42/48	0/48
> 58-62	48/48	48/48	0/48

will not adjust their speeds sufficiently to accommodate inadequate wet pavement on curves in particular. Drivers do not seem to recognize the fact that because of the lower coefficients of friction on wet pavements as compared with dry, wet pavements could lead to critical driving maneuvers, or even accidents. For passenger cars, the expected operating speed on wet or dry pavements can be determined by applying the nomograph for the relationship between degree of curve and 85th-percentile speed from Figure 3.



Legend:

FIGURE 3 Nomogram for evaluating operating speeds of passenger cars as related to degree of curve.

IMPLEMENTATION OF THE RESULTS FOR DESIGN PURPOSES

An international review of existing design guidelines (*2,16–20,30*) has shown that, to gain safety advantages, European countries directly or indirectly address three design issues in their guidelines much more explicitly than U.S. agencies. German, Swedish, and Swiss designers, for instance, are provided with geometric criteria, which direct them toward

1. Achieving consistency in horizontal alignment,
2. Harmonizing design and operating speeds on wet pavements, and
3. Providing adequate dynamic safety of driving.

For example, when consistency between successive design elements is not present, or a harmony between design speed and operating speed does not exist at a certain curved section, or an adequate dynamic safety of driving cannot be provided because of a reduction in friction factors because of wet pavement conditions (*15*), critical driving maneuvers may occur.

Providing an adequate dynamic safety of driving was discussed during the 69th Annual Meeting of the Transportation Research Board, January 1990 (*9*). Achieving consistency in horizontal alignment and harmonizing design and operating speeds on wet pavements were the subjects of several reports, publications, and presentations—for example, for the National Science Foundation (*3*), for the New York State Governor's Traffic Safety Committee (*4,5*), for the Transportation Research Board (*6,7,31,32*), for the Ohio Transportation Engineering Conference (*33–36*), for the International Road Federation (*8,30*), for the Swedish Road and Traffic Research Institute (*37*), for the International Road and Traffic Conference in Berlin (*24*), and for the German research community (*11*).

For achieving consistency in horizontal alignment, the following are recommended: (a) processes for evaluating horizontal design consistency and inconsistency, and (b) recommendations for achieving good and fair design practices, as well as recommendations for detecting poor designs. In harmonizing design and operating speeds on wet pavements, the review of current design practices and recommendations indicate that (a) because of the lower coefficients of friction on wet pavements as compared with dry, the wet condition should govern in determining the design speed (V_d); (b) the design speed should be constant along longer roadway sections; and (c) the design speed and the 85th-percentile speed ($V85$) on wet pavements must be well balanced to ensure a fine tuning between road characteristics, operating speed, and driving dynamics. For instance, studies (*14,38,39*) have shown that the design speed concept allows the building in of critical inconsistencies into the horizontal alignment, for example, between the flatter and sharper portions of the highway, when the controlling horizontal curves sometimes correspond to an arbitrarily selected design speed. In these cases, transition sections may exist, requiring unexpected critical speed changes from the driver, which may in turn lead to hazardous driving maneuvers. In addition, a tendency exists for some drivers to travel faster than the design speed on which the original design of the road section was based by substantial amounts, especially at lower design speed levels. This tendency points to the desirability that harmonizing design speed and operating speed is an important goal to be considered in new designs, redesigns, and rehabilitation strategies of two-lane rural highways.

In order to achieve these goals of achieving consistency in horizontal alignment and harmonizing design speed and operating speed, the following recommendations were elaborated (*6,8*):

1. Assess the road section where new designs, major reconstructions, or redesigns, for example, in case of resurfacing,

restoration, and rehabilitation (RRR) projects, may be considered.

2. Determine for this road section the degree of curve (DC) of each curve within the section, and the existing tangent lengths.

3. Determine the expected 85th-percentile speed for each curve, in relation to degree of curve, and for any tangents (DC = 0°), by applying Figure 3. The tangent must be of a certain length to be regarded in the design process, as discussed by Lamm et al. (*7*).

4. Calculate the change in degree of curve (ΔDC) and the change in operating speed ($\Delta V85$) between successive design elements, for example, between tangent and curve or curve and curve.

5. Calculate the difference between the 85th-percentile speed ($V85$) and the existing or selected design speed (V_d) for the investigated curve or tangent.

6. Determine the design category of the section according to the following recommendations, which are based on comparative analyses of accident rates for different degree of curve classes between successive design elements on the 322 investigated two-lane rural highway sections in New York (*6,8*).

Good Designs

Consistency Criterion

The change in degree of curve is $\Delta DC \leq 5°$ and the change in operating speed is $\Delta V85 \leq 6$ mph (10 km/hr) between successive design elements.

Design Speed Criterion

The difference between operating speed and design speed is $V85 - V_d \leq 6$ mph (10 km/hr) for the investigated curve or tangent. For these road sections, consistency in horizontal alignment exists and no improvements in geometric design would be necessary. No adaptations or corrections between design speed and operating speed have to be conducted. New designs should always be related to this case.

Fair Designs

- Consistency Criterion. The change in degree of curve is $5° < \Delta DC \leq 10°$ and the change in operating speed is 6 mph $< \Delta V85 \leq 12$ mph (20 km/hr) between successive design elements.
- Design Speed Criterion. The difference between operating speed and design speed is 6 mph $< V85 - V_d \leq 12$ mph for the investigated curve or tangent.

These road sections exhibit at least minor inconsistencies in geometric design. Normally, correcting the existing alignment is not necessary because low-cost projects such as traffic warning devices may, to a certain extent, be successful in correcting these defects. For instance, RRR improvements can be installed that consider appropriate recommended speeds, unless a safety problem has been documented. Despite traffic warning devices, road sections with changes in degree of curve that fall into the range of 5° to 10° have average accident rates that are about twice as high as those falling into the range of good design (*6,8*).

Superelevation rates in curves or curved sections should be related to the expected 85th-percentile speeds with respect to degree of curve, corresponding to Figure 3, and not to the design speed. Adequate driving dynamic safety is inferred as being provided under wet pavement conditions. The same holds true for calculating minimum stopping sight distances in curved and tangent sections.

Poor Designs

- Consistency Criterion. The change in degree of curve is $\Delta DC > 10°$ and the change in operating speeds on wet pavements is $\Delta V85 > 12$ mph between successive design elements.
- Design Speed Criterion. The difference between operating speed and design speed is $V85 > V_d > 12$ mph for the investigated curve or tangent.

These road sections represent strong inconsistencies in horizontal geometric design, combined with those breaks in the speed profile that may lead to critical driving maneuvers, especially under wet pavement conditions. Normally, for example, even high-cost RRR projects such as redesigns of at least hazardous road sections should be recommended, unless there is no documented safety problem. Road sections with changes in degree of curve that fall into the range of poor design normally have average accident rates that are more than four times as high as those falling into the range of good design (*6,8*).

The 85th-percentile speed should not be allowed to exceed the design speed by more than 12 mph (20 km/hr). If such a difference occurs, normally the design speed should be increased. For example, redesigns of at least hazardous road sections are recommended.

CONCLUSION

The impact of design parameters and traffic volume on operating speeds of passenger cars under free-flow conditions was evaluated. The data base consisted of 322 curved roadway sections of varying degrees of curve of two-lane rural highways in New York state. Among the parameters considered, degree of curve, length of curve, superelevation rate, gradient, sight distance, lane width, shoulder width, posted recommended speed, and AADT, only degree of curve was able to explain most of the variation in 85th-percentile speeds on dry pavements. The other parameters helped the regression model, but the equation did very well even without them.

Also examined was the effect of wet pavements on the 85th-percentile speeds of passenger cars. On all occasions, the surfaces were wet and rain was falling from a sprinkle to moderately heavy rain, but on no occasion did it rain so hard as to affect visibility appreciably. Minimum sight distances of about 450 to 550 ft, which represent the limiting values of stopping sight distances for a design speed of 55 mph, had to be provided.

Operating speeds on dry pavements were not statistically significantly different from operating speeds on wet pavements and drivers do not adjust their speeds sufficiently to accommodate adequate wet pavement on curves in particular. Drivers did not recognize the fact that friction supply is significantly lower on wet pavements as compared with dry. Furthermore, drivers reduce their speeds not so much on account of the danger created by lower skid resistance values on wet pavements, but rather because of limited sight distances because of heavy rain. By not adapting to wet roadway conditions, drivers will run a high risk of being involved in a traffic accident.

For the implementation of the results for design purposes, recommendations for achieving consistency and detecting inconsistencies in horizontal alignment as well as recommendations for harmonizing design speed and operating speed as related to wet pavement conditions were made for good, fair, and poor design practices. These recommendations are important tasks in modern highway design and redesign strategies for improving traffic safety.

ACKNOWLEDGMENT

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Methodology for Estimating Safe Operating Speeds for Heavy Trucks and Combination Vehicles on Interchange Ramps

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A number of research studies have shown the significance of considering truck and combination vehicle performance in the geometric design of interchange ramps. The greater potential these vehicles have for offtracking and loss of control as well as rollover plays an important role in determining the safe speed of a ramp. The analysis procedure described provides highway engineers with a method, first, to determine the critical speed of a ramp for such heavy vehicles. The PHASE-4 computer model is used to simulate the dynamic behavior of the vehicle for a specified ramp geometry. The complete procedure is computerized with a user friendly interface for specifying ramp parameters and built-in data sets of vehicle parameters. Then, a method to convert the critical speed to a safe operating speed for the ramp is presented. Input parameters and results for an example ramp consisting of one simple horizontal curve are included as a demonstration. The ramp is analyzed with two types of combination vehicles. The method for estimating the safe operating speed is then demonstrated by converting the critical speeds to safe operating speeds.

A number of research studies have been performed over the last 30 years that investigated vehicle performance on interchange ramps. Most of these have proceeded from a statistical base of data, and the safety significance of a given level or type of geometric feature was then determined through statistical inference, given the confidence limits of the analysis. An investigation by Ervin et al. (*1*) combined a broad base of accident data with a computer simulation of the dynamic behavior of representative tractor-semitrailer combinations. Results of this study show the significance of the consideration of truck and combination vehicle performance in the geometric design of interchange ramps.

The analysis procedure developed by the Texas Transportation Institute in conjunction with the Texas State Department of Highways and Public Transportation (SDHPT) provides the highway engineer with a method to determine speed limitations of a given ramp on the basis of the rollover potential and directional response of heavy trucks and combination vehicles. The PHASE-4 computer program (*2*) is used to simulate the dynamic behavior of the truck for the specified ramp geometry. This program was also used by Ervin et al. (*1*).

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PHASE-4 is a detailed computerized model that can accurately simulate braking and directional response of heavy trucks and combination vehicles. The program is capable of simulating trucks, tractor-semitrailers, and double and triple combinations. The program has been validated against analytical models, predecessor simulation programs, and vehicle test data acquired separately by the Texas Transportation Institute and the University of Michigan Transportation Research Institute. However, the program is primarily a research tool, requiring a large volume of input and expertise in modeling vehicle dynamics. Therefore, the PHASE-4 program was modified to determine speed limitations of heavy trucks and combination vehicles on a specified ramp geometry.

The critical speed determined by computer simulation should be converted to a safe speed limit by the application of a factor of safety. Selecting a proper factor of safety is a matter of personal preference governed by truck operating practices on ramps and any special conditions unique to a certain site. The method proposed for use by the Texas SDHPT is presented as an example.

Geometric features specifying the ramp were selected to best suit the general design practice of the Texas SDHPT and were limited to features used in most common designs. This list of ramp parameters may or may not be sufficient for other states depending on their design philosophy. Hence, additional features may be needed before the program can be used nationwide. A few additional features that can be added to improve the specification of ramp geometry are listed in the section on recommendations.

ORGANIZATION OF THE ANALYSIS PROCEDURE

The ramp analysis computer model was organized to perform the following tasks:

1. Provide the user a choice of different, easily accessible, built-in tractor-trailer vehicle data sets;
2. Read the geometric parameters specifying the ramp;
3. Determine the geometric parameters needed by the PHASE-4 program to simulate dynamic response of the chosen vehicle; and
4. Determine the critical speed of the ramp for the chosen vehicle by incrementing the speed from a user-defined lower bound and simulating the vehicle response for each speed.

BUILT-IN DATA SETS OF TRACTOR-TRAILER VEHICLE PARAMETERS

Parameters for two types of tractor-trailer vehicles are currently built into the program, for easy access to the user. When the program prompts for the type of vehicle, the user simply has to select one of the types.

The first of these two is a baseline vehicle representing a conventional five-axle tractor-semitrailer, which is loaded to the legal maximum of 80,000 lb gross combination weight, with a payload center-of-gravity (cg) height representing a medium-density freight (*1*). The normal height of the payload center of gravity in this case is 83 in. above ground. The vehicle is also defined with a common set of suspension properties representing popular levels of spring stiffness.

The second option represents a high-cg vehicle (*1*) in which the same tractor-semitrailer vehicle was chosen with

1. Payload cg height raised to 105 in. above the ground to represent a worst-case loading, which is known to occur in everyday trucking practice, and
2. Spring stiffnesses at the tandem suspensions both of the tractor and the semitrailer reduced with respect to the baseline case to represent the more compliant suspension types, which are known to be in common service.

Both of these additional features tend to increase rollover potential.

SPECIFICATION OF RAMP GEOMETRY

Design of ramps along with other roadway features of highways in the state of Texas are presently accomplished by the use of the roadway design system (RDS) (*3*). Hence, the program input structure for ramp geometry was developed to relate to RDS output as much as possible in order to ensure speedy and effortless transformation of ramp parameters.

First, the program prompts for the following parameters to fix the general layout of the ramp:

- Beginning station,
- Ending station,
- Elevation of ending station,
- Width of road,
- Number of horizontal curves,
- Number of vertical curves, and
- Number of superelevation transition sections.

Then, the horizontal alignment is to be specified for each horizontal curve by entering

- *P.C.* station,
- *P.T.* station,
- Radius (ft),
- *P.I.* angle (degrees), and
- Curve direction (to left or right).

These parameters are listed in RDS horizontal alignment output (*3*). Next, the following parameters are read in for each vertical curve to define the vertical alignment.

- Vertical *P.I.* station,
- Vertical *P.I.* elevation,
- First, vertical curve length (if asymmetrical), and
- Second vertical curve length (if asymmetrical).

Finally, superelevation information is specified with the following parameters for each superelevation transition section

- Transition station,
- Transition length,
- Super rate,
- Beginning or ending transition,
- Curving to left or right,
- Transition type (1, 2, or 3), and
- First and second vertical curve lengths, if type is 2.

Both the vertical alignment and the superelevation parameters are identical to the respective RDS input form parameters and arranged in the same order.

Figures 1–4 show the horizontal alignment, vertical alignment, and beginning and ending superelevation transition curves, respectively, for a ramp consisting of one simple horizontal curve and two vertical curves. Parameters for this ramp that are required for the analysis are listed in Tables 1–4 as an example of the program input.

RAMP ELEVATIONS AND GRADIENTS

For the simulation of dynamic response of the tractor-trailer, the PHASE-4 computer program requires ramp elevations Z and gradients dZ/dX and dZ/dY (denoted by $Z_{,X}$ and $Z_{,Y}$), at given values of X and Y inertial coordinates (see Figures 5 and 6). The term "elevation" is used for Z herein for convenience, although the positive direction of Z in PHASE-4 is downwards into the ground. Elevation Z is needed at each wheel and gradients are needed at wheels as well as axle centers (*2*). The ramp surface can be of any shape subjected to the following restrictions:

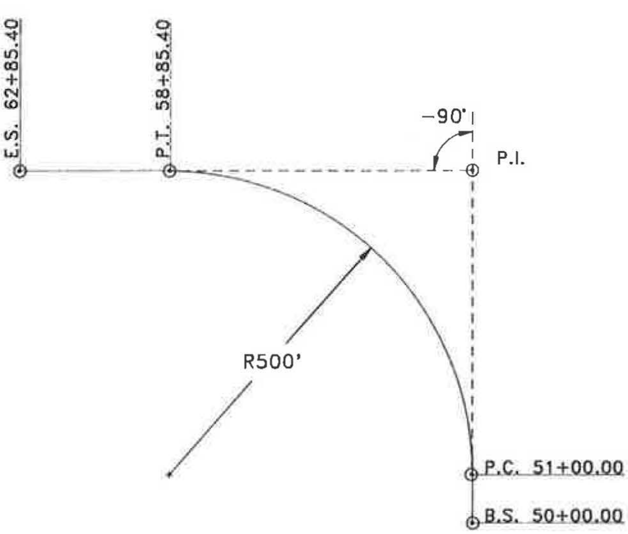

FIGURE 1 Horizontal alignment of example ramp.

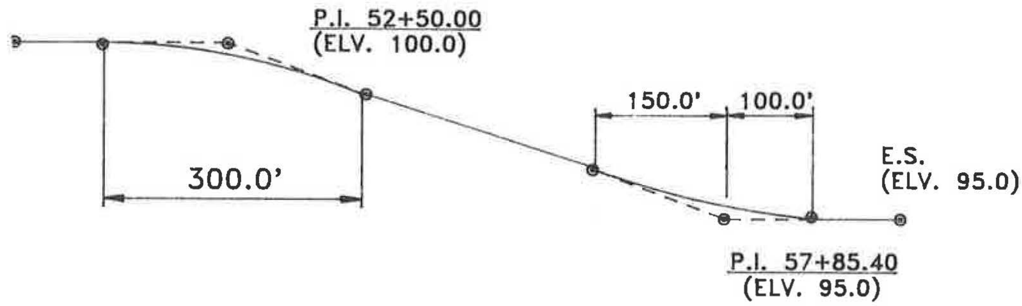

FIGURE 2 Vertical alignment of example ramp.

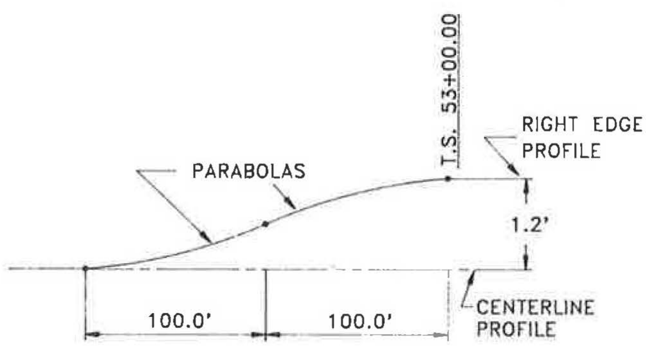

FIGURE 3 Beginning superelevation transition (Type 3) of example ramp.

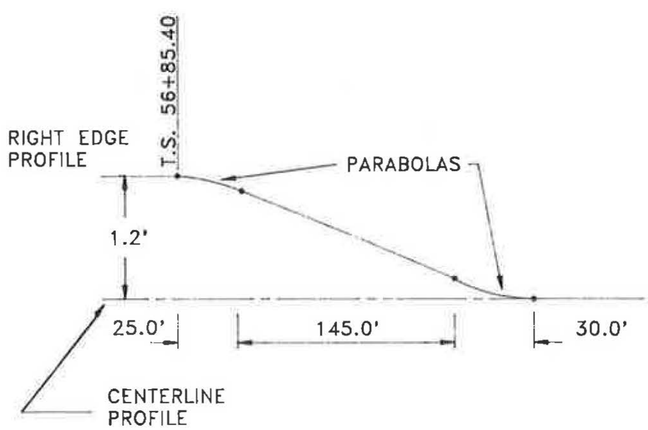

FIGURE 4 Ending superelevation transition (Type 2) of example ramp.

1. The complete vehicle train starts from a flat horizontal surface, and
2. All gradients (cross-slopes and grades) encountered by the vehicle train during the simulation remain less than about 0.1 (rise/run).

In order to facilitate the computation of ramp elevations and gradients, a curvilinear coordinate system $X'Y'$ was defined in which the X'-axis follows the ramp centerline and the Y' coordinates are measured perpendicular to the X' axis. This system is referred to herein as the "ramp coordinate system" (see Figure 5). Furthermore, the ramp is divided into (a) a set of horizontal alignment (HA) segments consisting of circular horizontal curves as well as straightline tangential portions connecting them, and (b) a set of vertical alignment (VA) segments consisting of parabolic vertical curves (both symmetric and asymmetric) as well as constant-grade sections connecting them.

For each HA segment (see Figure 5), the following are computed using the input parameters read for horizontal alignment:

1. X' coordinate at the end of the segment, X'_{hi};
2. Inertial coordinates of the end of the segment (point on the ramp centerline), X_i and Y_i;

TABLE 1 PARAMETERS SPECIFYING THE GENERAL LAYOUT OF EXAMPLE RAMP

	Description	Numerical Value
1	Beginning Station	50+00.00
2	Ending Station	62+85.40
3	Elevation of Ending Station	95.00
4	Width of the Road (ft.)	30.00
5	Number of horizontal curves in the roadway	1
6	Number of vertical curves in the roadway	2
7	Number of transition sections for superelevation	2

TABLE 2 PARAMETERS SPECIFYING HORIZONTAL ALIGNMENT OF EXAMPLE RAMP

Curve no.	P.C. Station	P.T. Station	Radius (ft.)	P.I. Angle (deg.)	Curve Direction
1	51+00.00	58+85.40	500.00	90.00	L

TABLE 3 PARAMETERS SPECIFYING VERTICAL ALIGNMENT OF EXAMPLE RAMP

Vertical Curve No.	Vertical P.I. Station	Vertical P.I. Elevation (ft.)	First Vertical Curve Length (ft.)	Second Vertical Curve Length (ft.)
1	52+50.00	100.00	300.00	0.00
2	57+85.40	95.00	150.00	100.00

TABLE 4 PARAMETERS SPECIFYING SUPERELEVATION FOR EXAMPLE RAMP

Transtn. Section No.	Transition Station	Transtn. Length (ft.)	Super Rate	B or E Curve	Curve to L or R	Type	First VCL (ft.)	Second VCL (ft.)
1	53+ .00	200.00	0.080	B	L	3		
2	56+85.40	200.00	0.080	E	L	2	25.00	30.00

Note: B or E = Beginning or Ending
L or R = Left or Right
VCL = Vertical Curve Length

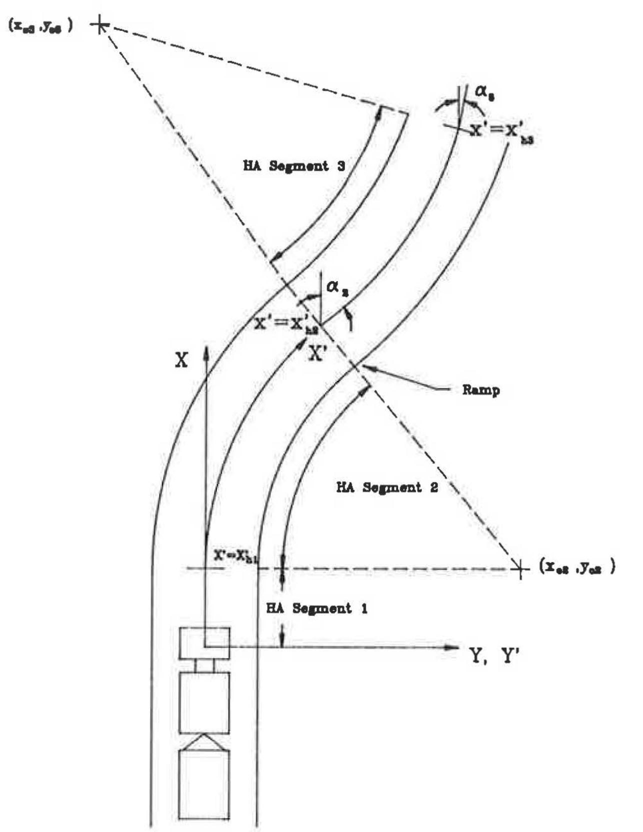

FIGURE 5 Parameters of horizontal alignment (HA) segments.

3. Inertial coordinates of the center (X_{oi}, Y_{oi}), (only for circular segments); and
4. Angle α_i between the inertial X axis and the tangent to the ramp centerline at the end of the segment (measured clockwise from X axis).

For each VA segment (see Figure 6), the following are determined using the input parameters read for vertical alignment:

1. X' coordinate at the end of the segment, X'_{vj};
2. Inertial Z coordinate at the end of the segment (on the ramp centerline), Z_j; and
3. Longitudinal gradient g_j at the end of the segment $(= -dZ/dX'$ at $X'_{vj})$.

In determining which HA and VA segment contains a certain wheel position, first the coordinate of the intersection between each HA segment centerline and the normal to it through the wheel position X'_{ci} is determined. The HA segment containing the wheel is the one satisfying the condition

$$X'_{h(i-1)} < X'_{Ci} \leq X'_{hi} \tag{1}$$

where $X'_{h(i-1)}$ is the X' coordinate at the end of the $(i-1)$th HA segment. Then, the VA segment containing the wheel is found by the condition

$$X'_{v(j-1)} < X'_C \leq X'_{vj} \tag{2}$$

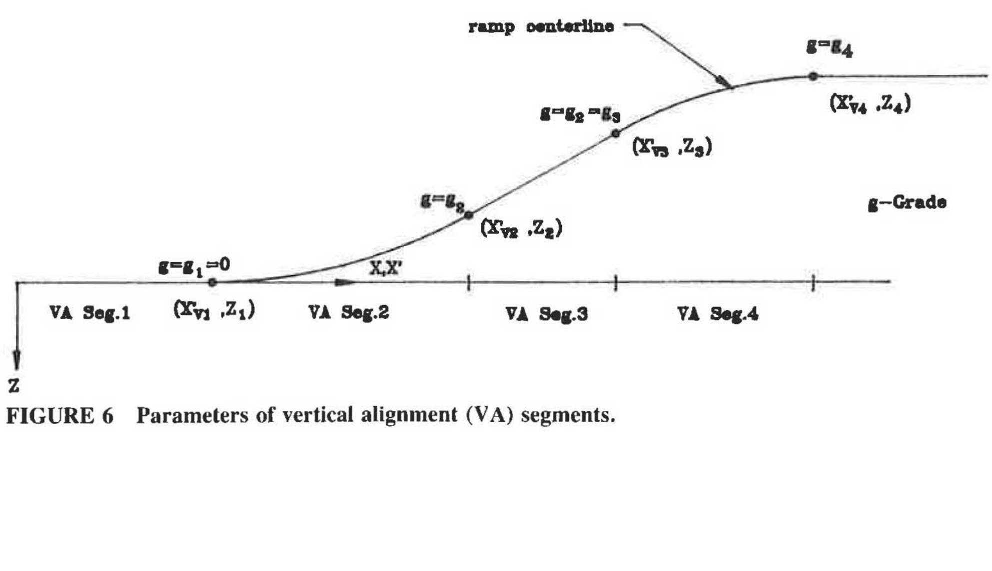

FIGURE 6 Parameters of vertical alignment (VA) segments.

where $X'_{v(j-1)}$ is the X' coordinate at the end of the $(j-1)$th VA segment, and X'_C is equal to X'_{Ci} satisfying Condition 1.

On determining the VA segment the wheel is on, elevation Z_C at the point with the ramp coordinates $(X'_C,0)$ is found as follows, depending on whether the segment is at constant grade or parabolically shaped. If $(X'_C - X'_{vj})$ and $[X'_C - X'_{v(j-1)}]$ are denoted X'_{Cvj} and $X'_{Cv(j-1)}$, respectively, for (a) a constant-grade segment,

$$Z_C = Z_{j-1} - X'_{Cv(j-1)}g_j \tag{3}$$

(b) a symmetric parabolic curve (*4*),

$$Z_c = Z_{j-1} - X'_{Cv(j-1)}\left[g_{j-1} + \frac{(g_j - g_{j-1})X'_{Cv(j-1)}}{2L_{vj}}\right] \tag{4}$$

and (c) an asymmetric parabolic curve (*4*),

$$Z_c = Z_{j-1} - X'_{Cv(j-1)}\left[g_{j-1} + \frac{(g_j - g_{j-1})X'_{Cv(j-1)}L_{2vj}}{2L_{vj}L_{1vj}}\right] \tag{5}$$

for $X'_{Cv(j-1)} \le L_{1vj}$, and

$$Z_c = Z_j - X'_{Cvj}\left[g + \frac{(g_j - g_{j-1})X'_{Cvj}L_{1vj}}{2L_{vj}L_{2vj}}\right] \tag{6}$$

for $X'_{Cv(j-1)} > L_{1vj}$,

where

L_{vj} = vertical curve length,
L_{1vj} = first vertical curve length for an asymmetric curve, and
L_{2vj} = second vertical curve length for an asymmetric curve.

The gradient dZ/dX' at $(X'_C,0)$ is found by taking the derivative of the Equations 3, 4, 5, or 6 with respect to X'_C.

Next, it is determined whether the wheel is on a (a) superelevation transition, (b) superelevated section, or (c) nonsuperelevated section. If X'_c corresponding to the wheel position satisfies the condition

$$X'_{sk} - L_{sk} < X'_C \le X'_{sk} \tag{7}$$

where L_{sk} and X'_{sk} are the length and X' coordinate at the end of the kth superelevation transition, respectively, then the wheel is on the kth superelevation transition. If X'_C satisfies the condition

$$X'_{sk} < X'_C \le X'_{s(k+1)} - L_{s(k+1)} \tag{8}$$

the wheel is on a superelevated section or a nonsuperelevated section depending on whether the kth transition is a beginning or an ending transition, respectively.

If the wheel is on a superelevation transition, cross slope dZ/dY' and gradient dZ/dX' (denoted by $Z_{,Y'}$ and $Z_{,X'}$, respectively) at the wheel are determined by the use of expressions presented in Table 5, depending on the type of transition (see Figure 7) and whether it is a beginning or an ending transition. Expressions for transition Types 2 and 3 were derived with the use of the formulas for reverse parabolic curves (*4*). For a superelevated section, the cross slope is equal to the superelevation, and the gradient is the same as that at the ramp centerline. If the section is not superelevated, the cross slope is zero and the gradient is equal to that at the ramp centerline.

Elevation Z at the wheel is then determined from the expression

$$Z = Z_C + Y'Z_{,Y'} \tag{9}$$

and the gradients $Z_{,X}$ and $Z_{,Y}$ are

$$Z_{,X} = Z_{,X'} \cos \alpha_C - Z_{,Y'} \sin \alpha_C$$
$$Z_{,Y} = Z_{,X'} \sin \alpha_C - Z_{,Y'} \cos \alpha_C \tag{10}$$

TABLE 5 EXPRESSIONS USED TO DETERMINE THE GRADES AND CROSS SLOPES OF SUPERELEVATION TRANSITION SECTIONS

transition type	beginning transition	ending transition
1	$Z_{,X'} = Z_{C,X'} + eY'/L_s$ $Z_{,Y'} = eX'_s/L_s$	$Z_{,X'} = Z_{C,X'} - eY'/L_s$ $Z_{,Y'} = e(1 - X'_s/L_s)$
2	$g_{,Y'} = -e/[L_s - (L_1+L_2)/2]$	$g_{,Y'} = e/[L_s - (L_1+L_2)/2]$
	(a) $X'_s \le L_1$ $Z_{,X'} = Z_{C,X'} - X'_sY'g_{,Y'}/L_1$ $Z_{,Y'} = -g_{,Y'}X'^2_s/(2L_1)$	(a) $X'_s \le L_1$ $Z_{,X'} = Z_{C,X'} - X'_sY'g_{,Y'}/L_1$ $Z_{,Y'} = e - g_{,Y'}X'^2_s/(2L_1)$
	(b) $L_1 < X'_s \le L_s - L_2$ $Z_{,X'} = Z_{C,X'} - Y'g_{,Y'}$ $Z_{,Y'} = -g_{,Y'}(X'_s - L_1/2)$	(b) $L_1 < X'_s \le L_s - L_2$ $Z_{,X'} = Z_{C,X'} - Y'g_{,Y'}$ $Z_{,Y'} = e - g_{,Y'}(X'_s - L_1/2)$
	(c) $X'_s > L_s - L_2$ $Z_{,X'} = Z_{C,X'} - Y'g_{,Y'}X'_{sL}/L_2$ $Z_{,Y'} = e - g_{,Y'}[L_2 - X'_{sL} - (L_2 - X'_{sL})^2/(2L_2)]$ $g_{,Y'} = 2e/L_s$	(c) $X'_s > L_s - L_2$ $Z_{,X'} = Z_{C,X'} - Y'g_{,Y'}X'_{sL}/L_2$ $Z_{,Y'} = -g_{,Y'}[L_2 - X'_{sL} - (L_2 - X'_{sL})^2/(2L_2)]$ $g_{,Y'} = -2e/L_s$
3	(a) $X'_s \le L_s/2$ same as for type 2(a) with $L_1 = L_s/2$	(a) $X'_s \le L_s/2$ same as for type 2(a) with $L_1 = L_s/2$
	(b) $X'_s > L_s/2$ same as for type 2(c) with $L_2 = L_s/2$	(b) $X'_s > L_s/2$ same as for type 2(c) with $L_2 = L_s/2$

Note:
L_s = transition length
e = rate of superelevation
$X'_s = X'_C - X'_{s(k-1)}$
$X'_{sL} = X'_{sk} - X'_C$
L_1 = first vertical curve length
L_2 = second vertical curve length

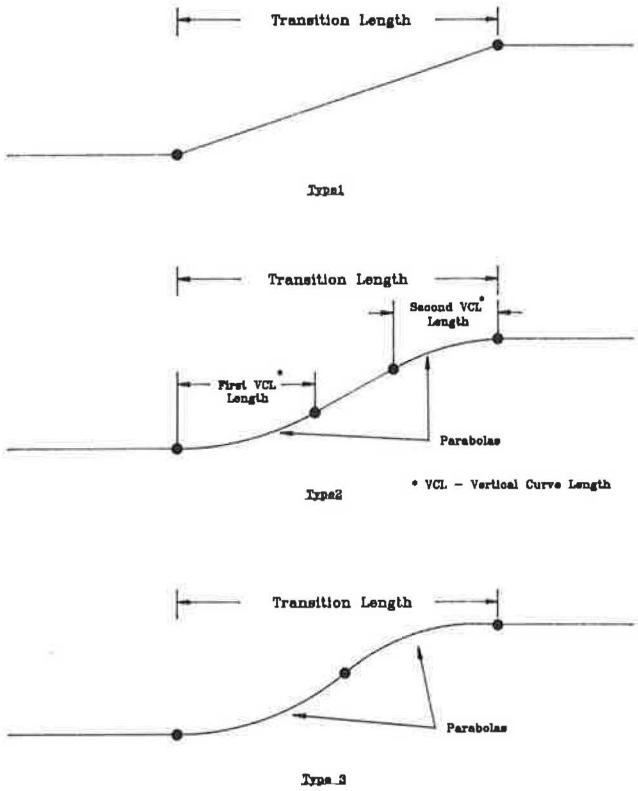

FIGURE 7 Types of superelevation transitions.

where α_C is the angle between the tangent to the ramp centerline at X'_C and the inertial X axis. If only the gradients are required (i.e., at axle centers), the step of computing Z by Equation 9 is bypassed.

PARAMETERS NEEDED FOR TRAJECTORY CONTROL

In order to control the trajectory of the tractor-trailer, the PHASE-4 program uses a path-follower, closed-loop driver model (*2*). For these computations, the program requires the deviation Y_p (see Figure 8) of the ramp centerline from a given point on the line through the origin of inertial coordinate system X, Y and parallel to the vehicle heading direction. If this point is X_p distance away from the origin of the X, Y coordinate system, X_p and Y_p can be expressed in terms of X, Y, and the yaw angle ψ as

$$X_p = X \cos \psi + Y \sin \psi$$

$$Y_p = -X \sin \psi + Y \cos \psi \qquad (11)$$

In order to find Y_p for a given X_p, first the deviation Y_{pi} to each segment is determined. Initially, a straight line segment is considered unbounded and the full circle is considered for a circular segment. Hence, for a straight line segment,

$$Y_{pi} = \frac{(Y_{hi} - m_i X_{hi}) - X_p (\sin \psi - m_i \cos \psi)}{(\cos \psi + m_i \sin \psi)} \qquad (12)$$

and for a circular segment,

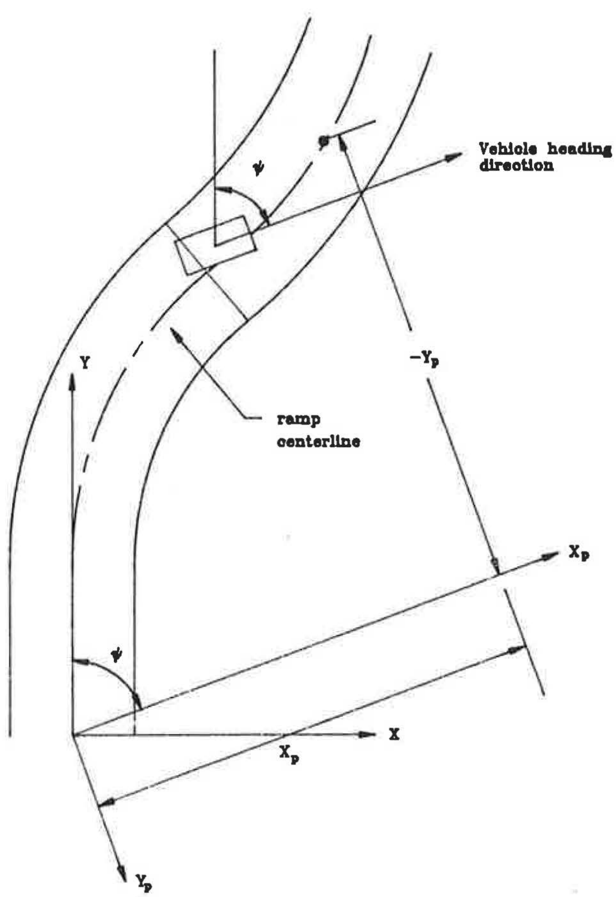

FIGURE 8 Parameters needed for trajectory control.

$$Y_{pi} = \pm\sqrt{R_i^2 - [X_p - (X_{oi} \cos \psi + Y_{oi} \sin \psi)]^2} - X_{oi} \sin \psi + Y_{oi} \cos \psi \qquad (13)$$

where R_i is the radius of curvature of the segment. Then it is checked whether the point on the segment centerline represented by the values X_p and Y_{pi} is within the bounds of that segment. In case of a circular segment, there are two such points given by the two values of Equation 13. If it is within bounds, the required value of Y_p is given by Y_{pi}. All segments are checked until a value for Y_p is found.

CRITICAL SPEED

Two types of hazardous situations are considered in computing the critical speed of a given ramp for a given tractor-trailer type: rollover and wheels running off the ramp. The critical speed for rollover is defined as the lowest speed at which a wheel would lift off the ramp surface. In the latter case, the critical speed is the lowest speed at which a wheel runs off the ramp. Overall critical speed is the minimum of these two speeds. In order to determine critical speed, first the tractor-trailer dynamic response is simulated starting with a given lower-bound speed. If the simulation is completed without any wheels lifting off the ground or running off the ramp, the simulation is repeated for 5-mph speed increments until a wheel is predicted to do so. On encountering a speed at which a wheel lifts off or runs off the ramp, the simulation is repeated with 1-mph increments starting from the previous 5-mph increment. The critical speed found in this manner is accurate to 1 mph. The vehicle is assumed to be in a coast mode during the simulation. Hence, the actual speed at which a wheel is predicted to lift off or run off the ramp is slightly different from the starting speed. This actual speed rather than the starting value is considered as the critical speed.

In order to give an indication of how close the vehicle came to each critical condition at each speed increment, a rollover factor and an offtrack factor are calculated.

Rollover Factor

At each time step during the simulation, the wheel with the minimum normal force is found. Then, the ratio of this force to the steady-state tire normal force is determined. The rollover factor is the minimum value of this ratio over the duration of the simulation for each speed. If offtracking is predicted, the duration is from the beginning to the point where the wheel runs off the road. When this factor equals zero, the corresponding wheel lifts off the ground.

Offtrack Factor

At each time step during the simulation, the wheel farthest from the ramp centerline is found. Then the ratio of the offset of this wheel from the centerline to the half-width of the ramp is determined. The offtrack factor is the maximum value of this ratio over the duration of the simulation for each speed. If a wheel lift-off is predicted, the duration is from the beginning to the point where the wheel lifts off the ground. When this factor equals 1, the corresponding wheel runs off the ramp.

The rollover factor and the offtrack factor for each speed, the wheel that lifted off the ground or ran off the ramp, the inertial coordinates of the wheel position, and the time at which it occurs are listed by the ramp analysis computer program as the table of stability factors. Tables 6 and 7 present stability factors for the baseline and high-cg vehicles, respectively, for the example ramp shown in Figures 1–4.

TABLE 6 STABILITY FACTORS ON EXAMPLE RAMP FOR BASELINE VEHICLE

Speed (mph)	Roll-over Factor	Off-track Factor	Vehicle Unit	Suspension	Axle	Side	X coord. (ft.)	Y coord. (ft.)	Time (sec.)
45.0	.400		tractor	Rear	Second	Left	213.98	-17.61	3.4451
		.401	tractor	Front		Right	606.01	-553.82	14.0841
50.0	.280		tractor	Rear	Second	Left	231.01	-21.70	3.3426
		.439	tractor	Front		Right	606.59	-554.83	12.8194
55.0	.053		tractor	Rear	Second	Left	249.45	-26.68	3.2801
		.449	trailer	Rear	Second	Right	606.74	-533.45	12.1370
56.0	.008		tractor	Rear	Second	Left	320.11	-53.93	4.1626
		.466	trailer	Rear	Second	Right	606.99	-528.01	11.8820
57.0	0.0*		tractor	Rear	Second	Left	232.90	-22.09	2.9575
		.420	tractor	Front		Left	153.66	-9.23	1.8175

* Rear suspension, Second axle, Left tire of the tractor comes off the road at X = 232.90 ft.; Y = -22.09 ft.; U = 56.48 mph & T = 2.9575 s.

TABLE 7 STABILITY FACTORS ON EXAMPLE RAMP FOR HIGH-cg VEHICLE

Speed (mph)	Roll-over Factor	Off-track Factor	Vehicle Unit	Suspension	Axle	Side	X coord. (ft.)	Y coord. (ft.)	Time (sec.)
45.0	.065		tractor	Rear	Second	Left	237.22	-23.26	3.8126
		.370	tractor	Front		Left	149.72	-8.06	2.2400
46.0	.013		tractor	Rear	Second	Left	230.44	-21.43	3.6251
		.375	tractor	Front		Left	150.29	-8.18	2.2000
47.0	0.0*		tractor	Rear	Second	Left	219.29	-18.72	3.3801
		.379	tractor	Front		Left	150.57	-8.28	2.1575

* Rear suspension, Second axle, Left tire of the tractor comes off the road at X = 219.29 ft.; Y = -18.72 ft.; U = 46.76 mph & T = 2.1575 s.

ESTIMATION OF SAFE OPERATING SPEED

Safe operating speed V_S for a heavy truck or a combination vehicle on a certain interchange ramp is the lesser of (a) the critical speed found by the PHASE-4 simulation divided by a factor of safety F, or (b) the design speed V_D of the ramp determined in accordance with the AASHTO Green Book (*5*).

First, factor of safety F is assumed to calculate a preliminary safe speed V_{S1} as

$$V_{S1} = V_{CR}/F \tag{14}$$

where V_{CR} is the critical speed computed by the simulation. Side friction factor f given in Figure III-5 of the Green Book (*5*) corresponding to V_{S1} is then found. A second speed V_{S2} is calculated as in the Green Book from the expression

$$V_{S2} = \sqrt{15R(e + f)} \tag{15}$$

where e is the maximum superelevation on the ramp and R is the minimum radius of curvature of the ramp (ft). If $V_{S1} \leq V_{S2}$, it can be shown that V_{S1} is also less than the design speed V_D of the ramp determined according to the Green Book (*5*). Hence, safe operating speed V_S of the ramp is V_{S1}. However, if $V_{S1} > V_{S2}$, the safe speed is equal to the AASHTO design speed V_D of the ramp because it can be shown that $V_{S1} > V_D$.

Because of limited information, selection of F in Equation 14 must be based for the most part on intuition and engineering judgment. Little is known about the manner in which drivers of large trucks negotiate ramps, e.g., how well they track the curvature and how well they adhere to advisory speeds. Further research is needed in this area. For the present problem, a value of 2 was selected for F.

Table 8 presents the summary of safe operating speed calculations for the example ramp shown in Figures 1–4, considering both baseline and high-cg vehicles. In this example,

TABLE 8 SUMMARY OF THE CONVERSION OF CRITICAL SPEED TO SAFE OPERATING SPEED FOR EXAMPLE RAMP

Parameter	"Baseline" Vehicle	"High C.G." Vehicle
V_{CR}	56	46
$V_{S1}{}^{a}$	28	23
f^{b}	0.176	0.192
e	0.08	0.08
R	500	500
$V_{S2}{}^{c}$	43.8	45.2
safe operating speed	28	23
AASHTO design speed	41.5	41.5

[a] From equation 14 with F = 2.0.

[b] From Figure III-5 of reference 5 with speed = V_{S1}.

[c] From equation 15.

V_{S1} is less than V_{S2} for both vehicles and therefore becomes the safe operating speed. The design speed according to the Green Book (*5*) is also presented in Table 8, and it is considerably larger than the estimated safe speeds. The safe operating speed calculated for the high-cg vehicle is about 5 mph lower than that for the baseline vehicle because of the greater rollover potential of the high-cg vehicle. This example underscores the need to consider large trucks in ramp design and in determining safe operating speeds.

Friction coefficients used in the simulations (i.e., in the built-in data sets) represent dry pavement conditions. As indicated in the example (see Tables 6, 7), the critical speed on dry pavements is governed by rollover as opposed to offtracking. Although wet conditions were not studied, the critical speed for trucks will generally not be governed by offtracking on a wet pavement. Design trucks overturned at a lateral acceleration between 0.25 and 0.45. Most wet pavements have an available side friction coefficient in excess of 0.3.

CONCLUSIONS AND RECOMMENDATIONS

The importance of considering truck and combination vehicle performance in the geometric design of interchange ramps has been shown by recent studies. The simplified, user friendly procedure developed using the PHASE-4 computer model (*2*) provides the highway engineer with a valuable method for estimating critical ramp speeds for heavy vehicles. A safe operating speed can then be computed by applying an appropriate factor of safety.

Further research should include improving the ramp specification of the ramp analysis computer model with additional geometric features, increasing the number of built-in tractor-trailer parameter sets to suit the nationwide trucking practice, incorporating an option to analyze the ramp under wet conditions, better quantifying the factor of safety applied to the critical speed to find the safe operating speed, and selection of speed limits for ramps with a mixture of vehicles.

Some additional geometric features to improve ramp specification are (a) transition spirals in horizontal alignment, (b) capability of specifying ramp geometry with stations numbered opposite to the traffic flow, and (c) specifying special ramp features with templates at selected stations [similar to RDS template form (*3*)].

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Review of AASHTO Case III Procedures for Intersection Sight Distance

KAY FITZPATRICK AND JOHN M. MASON, JR.

Equations to reproduce the 1984 AASHTO Green Book Case III–B and III–C passenger car sight distance values are developed. These equations produced values within 8 percent of the Green Book graphical values. Truck performance characteristics were selected on the basis of the information provided in the Green Book. Passenger car characteristics were then replaced with the truck characteristics to determine sight distance values for trucks. Values up to 3,200 ft were calculated. Field observations at intersections indicate that sight distances of such magnitude are not required. On the basis of a review of the Green Book procedures, specific recommendations are identified for consideration in future versions of AASHTO geometric design policy.

AASHTO's 1984 *A Policy on Geometric Design of Highways and Streets* (Green Book) (*1*) states "where traffic on the minor road of an intersection is controlled by stop signs, the driver of the vehicle on the minor road must have sufficient sight distance for a safe departure from the stopped position, even though the approaching vehicle comes in view as the stopped vehicle begins its departure movements." Three basic maneuvers can occur at a stop-controlled intersection:

- Travel across the intersecting roadway by clearing traffic on both the left and the right of the crossing vehicle,
- Turning left onto the crossing roadway by first clearing traffic on the left and then entering the traffic stream with vehicles from the right, and
- Turning right onto the intersecting roadway by entering the traffic stream with vehicles from the left.

These maneuvers, shown in Figure 1 [copied from Green Book (*1*), Figure IX–23], are referred to as Cases III–A, III–B, and III–C.

The Green Book (*1*) provides sight distance values for passenger cars (P) in Cases III–A, III–B, and III–C and for single-unit (SU) trucks and tractor-semitrailer trucks (WB–50) for Case III–A (crossing maneuver). However, AASHTO states that the "required sight distances for trucks making left (or right) turns into a cross road will be substantially longer than for passenger cars." The Green Book (*1*) also states that the "relationships for trucks can be derived using appropriate assumptions for vehicle acceleration rates and turning paths." The Green Book (*1*) lacks specific guidance on original assumptions on which vehicle characteristic values should be assumed.

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Because details regarding the development of AASHTO Figure IX–27 (Figure 2) are not available, equations are presented that reasonably reproduce the AASHTO Case III–B and III–C passenger car intersection sight distance (ISD)

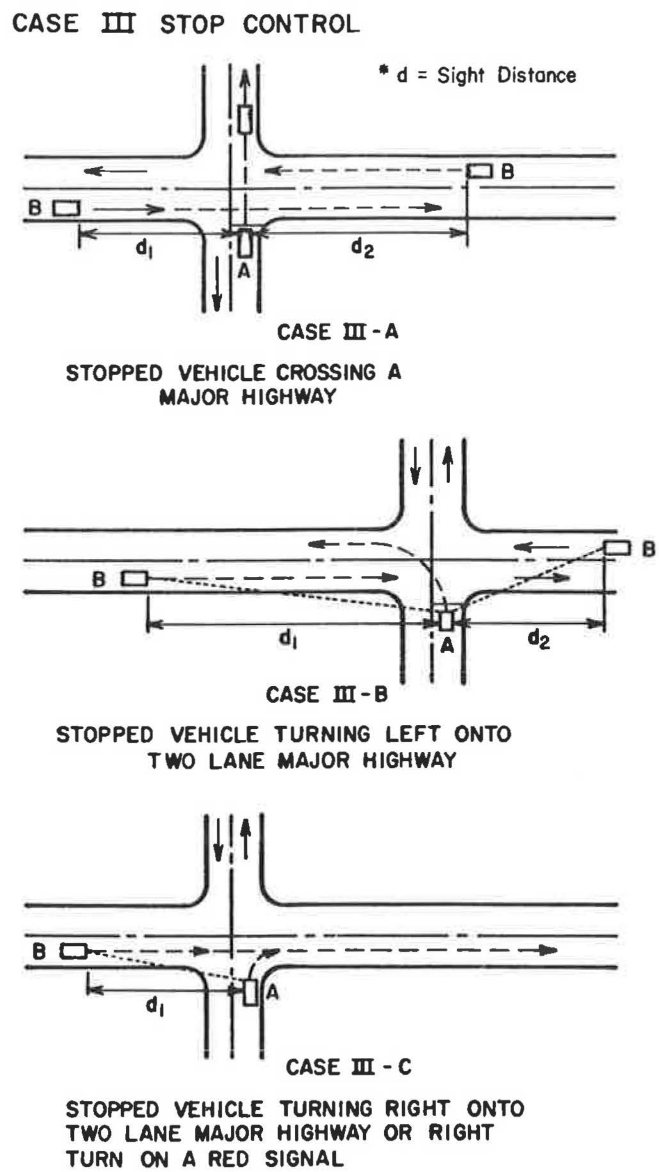

FIGURE 1 Intersection sight distance at at-grade intersections [Green Book (*1*), Figure IX–23].

curves. Once developed, associated truck characteristics were selected on the basis of information provided in the Green Book (*1*). Passenger car characteristics were then replaced with these truck characteristics to calculate Case III–B and III–C sight distance values for trucks. The resulting equations and values provide a baseline from which future ISD comparisons and modifications can be clearly established.

CURRENT GREEN BOOK CASE III POLICIES

Case III–A—Crossing Maneuver

As stated in the AASHTO Green Book (*1*), "the sight distance for a crossing maneuver is based on the time it takes for the stopped vehicle to clear the intersection and the distance that a vehicle will travel along the major road at its design speed in that amount of time." Case III–A shown in Figure 1 illustrates this condition. The required sight distance may be found in Green Book (*1*), Figure IX–26, or calculated from the following equations:

$$ISD = 1.47 \times V \times (J + t_a) \quad (1)$$

$$S = D + W + L \quad (2)$$

where

- ISD = d_1 or d_2, sight distance from intersection along major highway (ft);
- V = design speed on major highway (mph),
- J = sum of perception time and time required to actuate clutch or automatic shift (assumed 2.0 sec),
- t_a = time required to accelerate and traverse distance (S) to clear major highway pavement (sec),
- S = distance that crossing vehicle must travel to clear major highway (ft),
- D = distance from near edge of the major-road travel lanes to front of stopped vehicle (assumed 10 ft),
- W = pavement width along path of crossing vehicle (ft), and
- L = overall length of minor-road vehicle (ft).

Values of t_a can be read directly from Figure 3 [copied from Green Book (*1*), Figure IX–21] for nearly level conditions for a given distance S. AASHTO Green Book (*1*) values of L are 19, 30, 50, 55, and 65 ft for P, SU, WB–40, WB–50, and WB–60 vehicles, respectively.

Sight distance values for trucks can be found by using the appropriate t_a and L values. Figure 3 shows time versus distance curves for a WB–50 and SU truck and passenger car under normal and assumed acceleration.

Case III–B—Turning Left onto a Crossroad

A vehicle turning left onto a crossroad from a stopped position clears vehicles approaching from the left and then turns left and enters the traffic stream from the right. The turning vehicle should be able to accelerate to the average running speed by the time the approaching vehicle is within a certain tailgate distance after reducing its speed to the average running speed, or the turning vehicle should be able to accelerate to the design speed by the time the approaching vehicle, maintaining the design speed, is within a certain tailgate distance. Figure IX–24 in the Green Book (*1*) contains the details of this case.

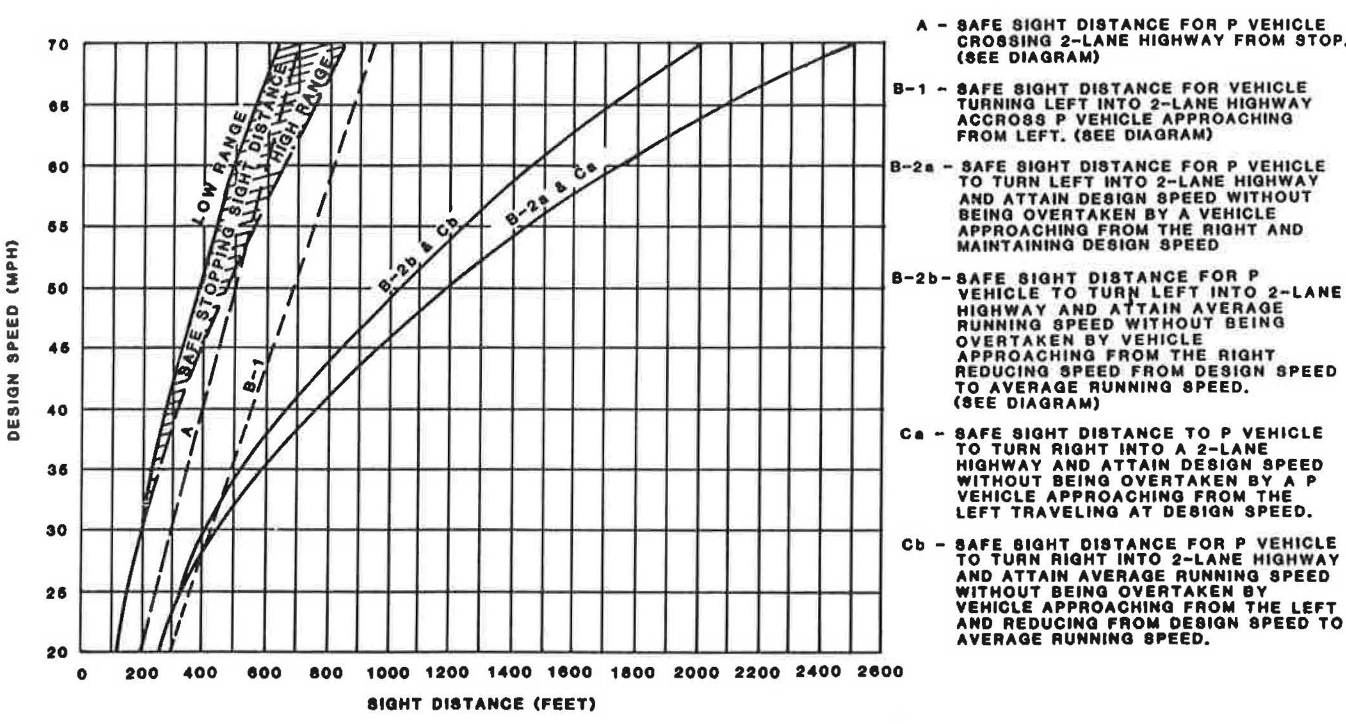

FIGURE 2 Intersection sight distance at at-grade intersections [Green Book (*1*), Figure IX–27, Case III–B and C].

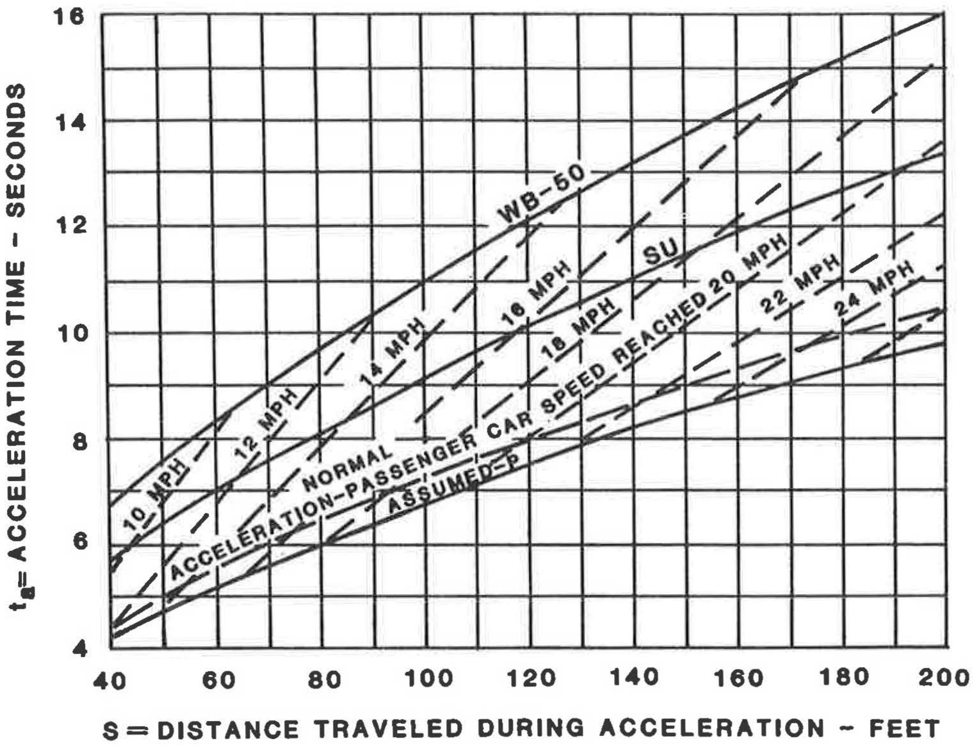

FIGURE 3 Time versus distance to accelerate from a stop curve [Green Book (*1*), Figure IX–21].

ISD values are determined from the design curves shown in Figure 2.

AASHTO states that the required sight distances for trucks turning left onto a crossroad will be substantially longer than for passenger cars. AASHTO further indicates that the sight distance for trucks can be determined using appropriate assumptions for vehicle acceleration rates and turning paths. However, the specific assumptions are not identified in the Green Book (*1*). As AASHTO presents this condition, the case lacks sufficient information to derive the design curves for determining required sight distance dimensions. Specifically, the following elements are unknown:

- Acceleration of the minor-road vehicle,
- Deceleration of the major-road vehicle, and
- Tailgate distance.

Case III–C—Turning Right onto a Crossroad

A vehicle turning right must have sufficient sight distance of vehicles approaching from the left in order to complete its right turn and accelerate to the running speed or design speed before being overtaken by traffic approaching the intersection from the left and traveling at the same running or design speed. Case III–C policy is described in Figure IX–25 in the Green Book (*1*). Sight distance for a right-turn maneuver is only a few feet less than that required for a left-turn maneuver. For Case III–C, as in Case III–B, AASHTO indicates that sight distances for trucks need to be considerably longer than for passenger vehicles and sufficient information is lacking to derive the design curves for determining the sight distance dimensions. Figure 2 is also used to determine the ISD for right-turning vehicles.

REPRODUCTION OF AASHTO SIGHT DISTANCE VALUES

Several ISD curves or values are presented in the Green Book (*1*) without supporting derivations or references. Its Figure 2, sight distance for Case III–B and III–C, contains six curves. Two curves are actually the upper and lower limits for stopping sight distance (SSD). Chapter 3 (Elements of Design) of the Green Book (*1*) contains the procedure for producing the SSD values. The curve labeled A, which represents Case III–A methodology, is the sight distance for a passenger car crossing a two-lane highway from a stopped position.

The remaining three curves represent the following sight distance procedures:

- Left-turning vehicle to clear the near lane [Green Book (*1*) Case III–B, B–1 curve, or clear-lane (CL) procedure];
- Turning vehicle to accelerate to design speed while major-road vehicle maintains a constant speed [Green Book (*1*) Cases III–B and III–C, B–2a & Ca curve, or constant-speed (CS), procedure]; and
- Turning vehicle to accelerate to running speed while major-road vehicle reduces speed from design speed to running speed

[Green Book (*1*) Case III–B and III–C, B–2b & Cb curve, or reduced-speed (RS) procedure].

These three curves lack sufficient information to easily identify some of the assumed parameter values. Using the information that is provided in the Green Book (*1*), and making some reasonable assumptions for the missing information, the identified curves were reproduced as presented.

Vehicle Acceleration From a Stopped Position

Two figures in the Green Book (*1*) contain vehicle acceleration information. Figure 3 shows time versus distance curves depicting passenger car under normal acceleration, the recommended assumed passenger car acceleration, SU trucks, and WB–50 trucks. These curves represent 40 to 200 ft of distance traveled during acceleration. Figure 4 [copied from Green Book (*1*), Figure IX–22] shows speed-distance curves for normal rate passenger car acceleration for level conditions from 0 to 2,400 ft and "normal rate for level conditions, representative weight-power ratio" for truck acceleration from 0 to 5,000 ft. Truck acceleration curves in Figure 4 represent SU and WB–50 design vehicles and were determined from truck operation studies.

Acceleration times can be estimated from Figure 4. Table 1 presents the distance values in 5-mph increments read from Figure 4. The distance increment column is the distance traveled during the previous 5-mph increase. The time increment column was calculated using the following equation:

$$\text{Time Increment} = \frac{2 \times \text{Distance Increment}}{V_{\text{initial}} + V_{\text{final}}} \quad (3)$$

Acceleration time is the sum of the previous time increments. An inherent assumption in using Figure 4 both for right- and

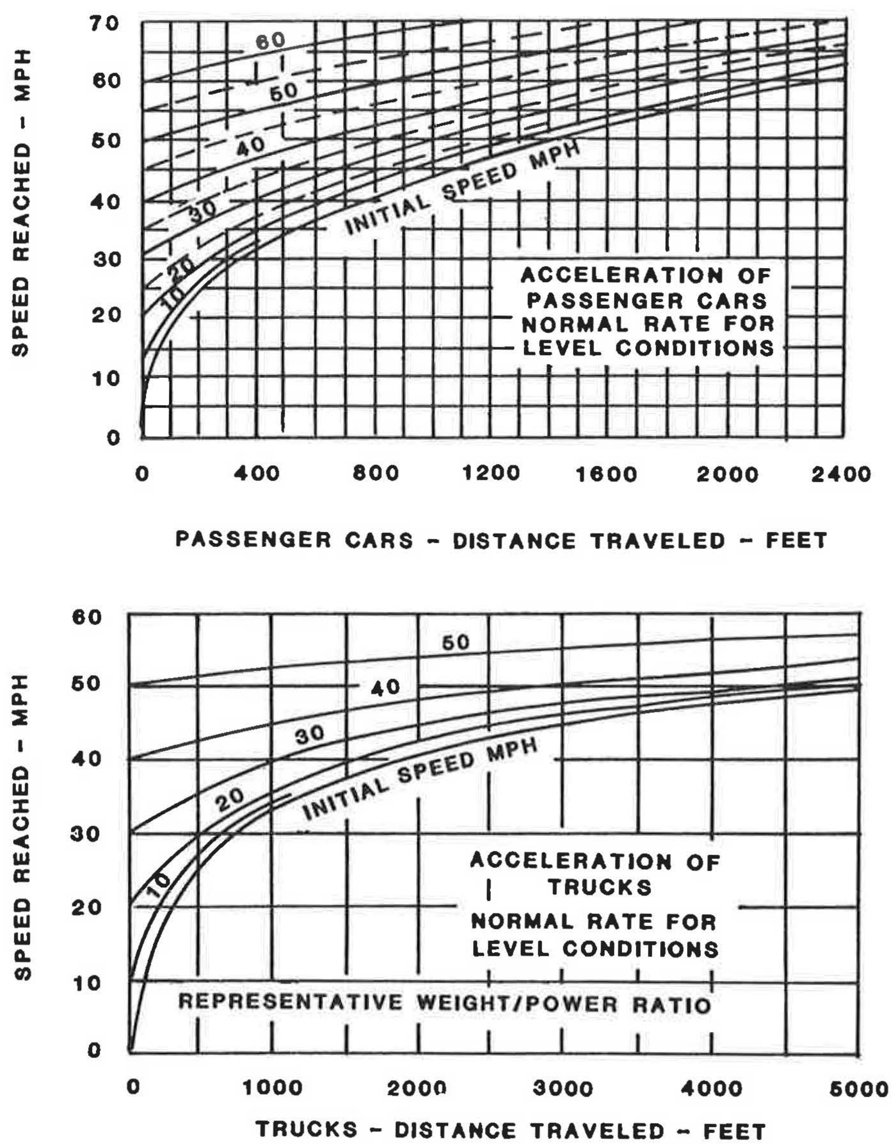

FIGURE 4 Speed reached versus distance to accelerate curves [Green Book (*1*), Figure IX–22].

TABLE 1 DISTANCE AND TIME VALUES ON THE BASIS OF FIGURE 4

Speed (mi/h)	Distance Traveled* (ft)	Distance Increment (ft)	Time Increment (sec)	Acceleration Time (sec)
Passenger Cars				
5	10	10	2.72	2.72
10	30	20	1.81	4.54
15	50	20	1.09	5.62
20	125	75	2.92	8.54
25	210	85	2.57	11.11
30	350	140	3.46	14.57
35	550	200	4.19	18.76
40	800	250	4.54	23.29
45	1,075	275	4.40	27.70
50	1,400	325	4.65	32.35
55	1,800	400	5.18	37.53
60	2,300	500	5.92	43.45
65	2,900	600	6.53	49.98
70	3,600	700	7.05	57.03
Trucks				
5	50	50	13.61	13.61
10	100	50	4.54	18.14
15	175	75	4.08	22.22
20	275	100	3.89	26.11
25	450	175	5.29	31.40
30	700	250	6.18	37.58
35	1,200	500	10.47	48.05
40	1,900	700	12.70	60.75
45	3,200	1,300	20.81	81.56
50	5,000	1,800	25.78	107.34

*Values read from figure 4.

left-turning vehicles is that the acceleration rates for the turning maneuvers are not significantly different.

AASHTO B-1 Curve or Clear Lane (CL)

The B-1 curve, as described by AASHTO, is used to establish the sight distance to be provided for a passenger car turning left onto a two-lane highway when an automobile is approaching from the left as shown in Figure 5 [modified from Green Book (*1*), Figure IX-24]. Sight distance is the product of the major-road vehicle's speed and the turning vehicle's acceleration time needed to clear the near lane. The following equations were used to reproduce the curve:

$$ISD_{B-1} \text{ or } ISD_{CL} = 1.47Vt \quad (4)$$

$$t = t_S + J \quad (5)$$

$$S_t = D + L + W_t \quad (6)$$

$$W_t = \pi \times R/2 \quad (7)$$

where

ISD_{B-1} or ISD_{CL} = sight distance along major roadway's near lane to the left for left turns (ft), (see Figure 5);

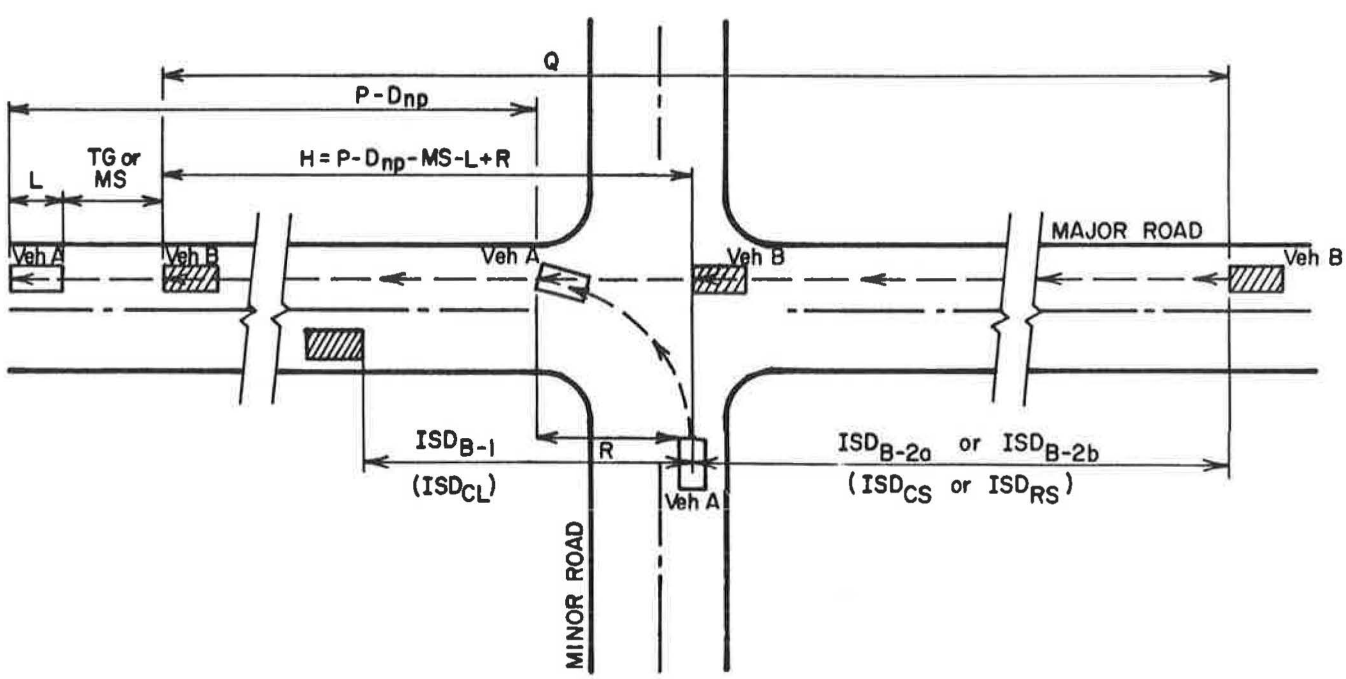

FIGURE 5 Distances considered in a left-turn maneuver [modification of Green Book (*1*), Figure IX-24].

t = time for stopped minor-road vehicle to initiate turn and clear the near lane (sec);
t_S = time required by turning vehicle to accelerate and traverse distance (S_t) to clear near lane (sec);
S_t = distance that turning vehicle must travel to clear near lane (ft);
W_t = distance traversed along path of turning vehicle (ft);
L = length of minor-road vehicle (ft); and
R = radius of turn for minor-road vehicle (ft).

Data for t_S are available from Figure 4 and Table 1. AASHTO Green Book (*1*) values of R are 40, 50, and 60 ft for the P, SU, and WB-50 vehicles, respectively, for left turns, and 24, 42, and 45 ft, respectively, for right turns.

Values from the Green Book B-1 curve were used to calculate acceleration times (t_S). Assuming perception-reaction time J of 2.0 sec, the t_S value averaged 7.4 sec. Using data derived from Figure 4 or Table 1, the distance traveled S_t by a passenger car during 7.4 sec is 95 ft. Assuming a vehicle length L of 19 ft and a 10-ft distance D between edge of travel way and front of the vehicle, the pavement traversed W_t along the turning path of the vehicle is 66 ft, which results in a 42-ft radius. Control radius R for a left-turning passenger car from the Green Book (*1*) is 40 ft. The 42-ft radius is within an acceptable range of an assumed 40-ft radius. As such, the acceleration time (t_S) and distance (S_t) values used to generate the B-1 curve agree with Figure 4.

Table 2 presents the calculated sight distances for a passenger car, SU truck, and WB-50 truck. Figure 6 shows the

TABLE 2 GREEN BOOK (*1*) B-1 CURVE (CLEAR LANE) SIGHT DISTANCE VALUES

Speed (mi/h)	Green Book B-1 Curve (Figure 2) (ft)	Calculated Sight Distance: Passenger Car B-1-P (ft)	SU Truck B-1-SU (ft)	WB-50 Truck B-1-WB50 (ft)
20	300	272	622	687
25	350	340	777	858
30	425	408	933	1,030
35	500	476	1,088	1,202
40	550	544	1,243	1,374
45	625	612	1,399	1,545
50	675	680	1,554	1,717
55	750	748	1,710	1,889
60	825	816	1,865	2,060
65	875	884	2,021	2,232
70	950	952	2,176	2,404

The following vehicle characteristics were used:

Characteristic	PC	SU	WB-50
Vehicle length, L (ft)	19	30	55
Turning Radius, R (ft)	40	50	60
Distance, S_t (ft)	92	119	159
Time to clear, t_t (sec)	7.25	19.15	21.4

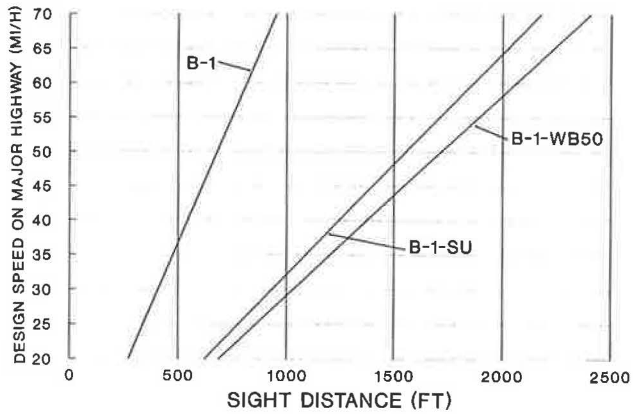

FIGURE 6 Sight distance curves for B-1 clear-lane procedure.

data values presented in Table 2. Turning radii R values selected from the Green Book (*1*) were 40, 50, and 60 ft. The time (t_S) to clear the near lane is based on data derived from Figure 4 or Table 1.

Green Book (*1*) B-2a & Ca Curve for Constant Speed (CS)

Using information presented in the Green Book (*1*), the following equations were developed to reproduce the Green Book (*1*) B-2a & Ca curve [see Figure 5 for an illustration of the dimensions for left turns and Figure 7, modified from Green Book (*1*), Figure IX-25, for right turns]:

$$\text{ISD}_{\text{B-2a\&Ca}} \text{ or } \text{ISD}_{\text{CS}} = Q - H \quad (8)$$

$$Q = 1.47 \times V \times t_{as} \quad (9)$$

$$t_{as} = t_S + J \quad (10)$$

$$H = P - D_{np} + \text{R} - \text{TG} - \text{L} \quad (11)$$

$$D_{np} = \pi \times R/2 \quad (12)$$

$$\text{TG} = 1.47 \times V \times t_{\text{TG}} \quad (13)$$

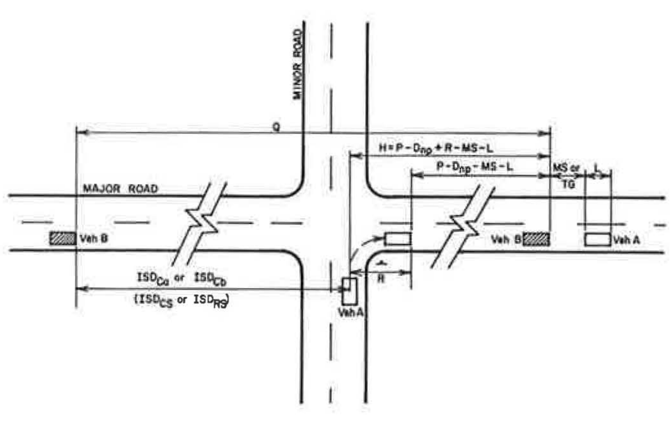

FIGURE 7 Distances considered in a right-turn maneuver [modification of Green Book (*1*), Figure IX-25].

where

$ISD_{B-2a\&Ca}$ or ISD_{CS} = sight distance along major roadway's far lane to the right for left turns and along near lane to the left for right turns assuming the major-road vehicle maintains constant speed during minor-road vehicle's turning maneuver (ft) (see Figures 5 or 7),

Q = distance traveled by major-road vehicle during minor-road vehicle's turning maneuver (ft),

H = major-road vehicle's distance from intersection when at assumed tailgate distance to minor-road vehicle (ft),

t_{as} = time for stopped minor-road vehicle to move into traffic stream and accelerate to design speed (sec),

P = total distance traveled by minor-road vehicle from stopped position to location when design speed is achieved (ft) (data derived from Figure 4 or Table 1),

D_{np} = distance minor-road vehicle traveled during the turning maneuver that is not parallel to major highway (ft),

TG = tailgate distance (ft), and

t_{TG} = tailgate time (sec).

If the major-road vehicle maintains a constant speed during the turn maneuver, the distance Q is constant speed V multiplied by the time for the minor-road vehicle to complete the turn maneuver. Time t_{as} would equal the minor-road driver's perception-reaction time J plus the time t_S from when the vehicle began moving to the time when the turning vehicle reached the same speed as the major-road vehicle. A perception-reaction time value for the turning vehicle driver is not mentioned in the Green Book (*1*). The 2.0 sec used for the crossing maneuver (Case III–A) was assumed for the turning maneuvers.

The Green Book (*1*) does not provide information on how to derive tailgate distance TG. Experimenting with different values for TG to provide the closest estimate of the B–2a & Ca curve resulted in a vehicle separation time (t_{TG}) of 1.0 sec. Tailgate distance, measured from the rear of the turning vehicle to the front of the oncoming vehicle, is the product of the speed of the major-road vehicle (V) and the 1.0-sec interval.

Values used for distance P and time t_S to accelerate were derived from Figure 4 in 5-mph increments and are presented in Table 1. ISD values calculated for passenger cars using these assumptions are presented in Table 3 and shown in Figure 8. Differences between these values and the Green Book (*1*) values range from 0 to 6 percent increases.

The Green Book (*1*) implies that the sight distances would be greater for trucks but does not provide specific values. Truck ISD values presented in Table 3 were calculated using these assumptions and equations. Acceleration distance P and time t_S values were obtained from the acceleration of truck curves in Figure 4 (presented in Table 1). Truck intersection sight distance values are between 158 and 169 percent greater than those obtained from the B–2a & Ca curve.

Sight distance lengths between Case III–B (left-turn) and Case III–C (right-turn) situations only differ because of the different turning radii R between a left turn and a right turn. Typically, left-turn sight distance is 2 percent greater than the sight distance calculated using minimum turning radius for right turns.

TABLE 3 GREEN BOOK (*1*) B–2a & Ca CURVE (CONSTANT SPEED) SIGHT DISTANCE VALUES

Speed (mi/h)	Green Book B-2a & Ca Curve (figure 2) (ft)	Calculated Sight Distance Passenger Car (ft)	Calculated Sight Distance Truck BT-2a & Ca (ft)
20	250	249	670
25	340	343	903
30	450	460	1,179
35	580	604	1,516
40	750	781	1,938
45	950	990	2,483
50	1,190	1,233	3,199
55	1,440	1,512	*
60	1,730	1,832	*
65	2,100	2,197	*
70	2,500	2,612	*

*Acceleration time and distance information is not available.

The following vehicle characteristics were used:

Characteristic	PC	AASHTO Truck
Vehicle length, L (ft)	19	55
Turning Radius, R (ft)	28*	60
Time to accelerate, t_t, from figure 4 and table 1		

*Selected based on dimension shown in Green Book figure IX-24. Minimum turning radius is 24 ft for a right-turning passenger car and 40 ft for a left-turning passenger car.

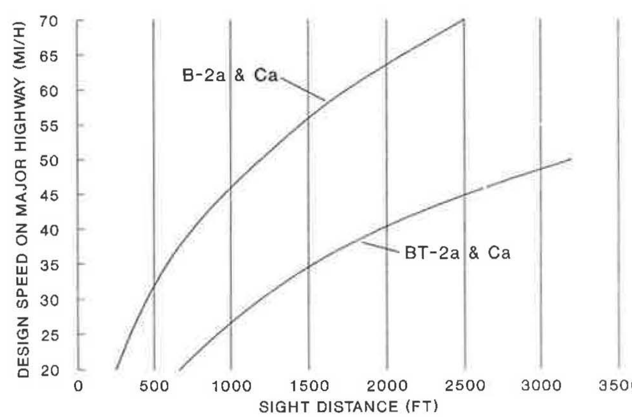

FIGURE 8 Sight distance curves for B–2a & Ca constant-speed procedure.

Green Book (*1*) B–2b & Cb, or Reduced-Speed (RS), Curve

The B–2b & Cb curve represents the condition in which the major-road vehicle decelerates during the turning maneuver of a minor-road vehicle (see Figures 5 and 7), which appears to more closely represent actual operations at an intersection. The following equations were developed to reproduce the curve:

$$ISD_{B-2b\&Cb} \text{ or } ISD_{RS} = Q - H \quad (14)$$

$$Q = 1.47V_{ds}t_{ds} + D_{dec} + 1.47V_{rs}t_{rs} \quad (15)$$

$$t_{rs} = t_{as} - t_{ds} - t_{dec} \quad (16)$$

$$t_{as} = t_{ar} + J \quad (17)$$

$$t_{ds} = J + t_{pr} \quad (18)$$

$$t_{dec} = (2 \times D_{dec})/(1.47)(V_{ds} + V_{rs}) \quad (19)$$

$$H = P - D_{np} + R - TG - L \quad (20)$$

$$D_{np} = \pi \times R/2 \quad (21)$$

$$TG = 1.47 \times V_{rs} \times t_{TG} \quad (22)$$

where

$ISD_{B-2b\&Cb}$ or ISD_{RS} = sight distance along major roadway's far lane to the right for left turns and along the near lane to the left for right turns, assuming that major-road vehicle reduces speed from design speed to running speed during minor-road vehicle's turning maneuver (ft) (see Figures 5, 7);
V_{ds} = design speed of major-road vehicle (mph);
t_{ds} = time major-road vehicle is at design speed during turning maneuver (sec);
D_{dec} = distance major-road vehicle traversed during deceleration (ft), data derived from Green Book (*1*), Figure II–13;
t_{dec} = time major-road vehicle is decelerating (sec);
V_{rs} = running speed of major-road vehicle (mph);
t_{rs} = time major-road vehicle is at running speed during turning maneuver (sec);
t_{ar} = acceleration time for the minor-road vehicle to achieve running speed from a stopped position (sec), data derived from Figure 4 or Table 1; and
t_{pr} = perception-reaction time for the major-road driver (assumed to be 2.0 sec).

Tailgate time t_{TG} is assumed to be 1.0 sec.

The Green Book (*1*) does not include a discussion on how to calculate distance Q traversed by the major-road vehicle during the turning vehicle's maneuver. Calculation for this distance is more complex than for the previous situation. Distance Q comprises three segments: distance traveled at design speed, distance traveled while decelerating from design speed to running speed D_{dec}, and distance traveled at running speed. Time at design speed was assumed to equal the minor-road driver's perception time J plus the major-road driver's perception-reaction time t_{pr}, assuming that the major-road driver begins to decelerate when initiation of the minor-road vehicle's turn maneuver is perceived.

Distance to decelerate D_{dec} is available from Green Book (*1*), Figure II–13. Final speed curves of 50, 40, 30, 20, and 0 mph are also available. Reductions to 55 mph or greater can be determined using a comfortable deceleration rate of 3.3 mph/sec as discussed by the ITE (*2*). Time to decelerate t_{dec} can be calculated from distance to decelerate D_{dec} using Equation 19. Time spent at running speed t_{rs} can then be calculated by subtracting the time at design speed t_{ds} and time to decelerate t_{dec} from the turning maneuver time t.

Distance P covered by the minor-road vehicle for this situation is similar to the constant-speed procedure except the vehicle is accelerating to running speed instead of design speed. Tailgate distance TG is based on running speed.

Trial and error was used to estimate the speed reduction of the major-road vehicle used in the Green Book (*1*). Findings based on these assumptions agreed with the B–2b & Cb curve to within 8 percent. Speed reductions in 5-mph increments that provided the best agreement with the Green Book (*1*) curves were

- No speed reduction for design speeds less than 30 mph,
- Speed reduction of 5 mph for design speeds between 30 and 65 mph, and
- Speed reduction of 10 mph for design speed of 70 mph.

Table 4 presents the results using these assumptions for passenger cars. Sight distance for trucks can now be calculated using these equations and the truck acceleration data derived from Figure 4 (see Table 1). These sight distance values (BT–2b & Cb) are also presented in Table 4. The truck sight distances are between 131 and 178 percent higher than the B–2b & Cb curve. Green Book (*1*) passenger car and the calculated truck sight distances are shown in Figure 9.

HISTORY AND CRITIQUE OF AASHTO PROCEDURES

Case III–A—Crossing Maneuver

AASHTO's crossing maneuver policy has remained the same in concept, formulation, and assumed J, t_a, D, and L values since it appeared in the 1954 Blue Book (*3*). AASHTO curves for t_a and time to accelerate (Figure 3) were established from tests conducted by the Bureau of Public Roads in 1937 (*4*). The parameter that most influences ISD is a vehicle's acceleration characteristic. One of the most comprehensive discussions of vehicle acceleration rates appears in the appendix

TABLE 4 GREEN BOOK (*1*) B–2b & Cb CURVE (REDUCE SPEED) SIGHT DISTANCE VALUES

Speed (mi/h)	Green Book B-2b & Cb Curve (figure 2) (ft)	Calculated Sight Distance: Passenger Car (ft)	Calculated Sight Distance: Truck BT-2b & Cb (ft)
20	250	249	670
25	325	343	903
30	425	460	1,179
35	525	494	1,213
40	660	638	1,549
45	825	814	1,971
50	1,025	1,023	2,516
55	1,225	1,266	3,232
60	1,475	1,545	*
65	1,725	1,865	*
70	2,000	1,906	*

*Acceleration time and distance information is not available.

The following vehicle characteristics were used:

Characteristic	PC	AASHTO Truck
Vehicle length, L (ft)	19	55
Turning Radius, R (ft)	28*	60
Time to accelerate, t_t, from figure 4 and table 1		

*Selected based on dimension shown in Green Book figure IX-24. Minimum turning radius is 24 ft for a right-turning passenger car and 40 ft for a left-turning passenger car.

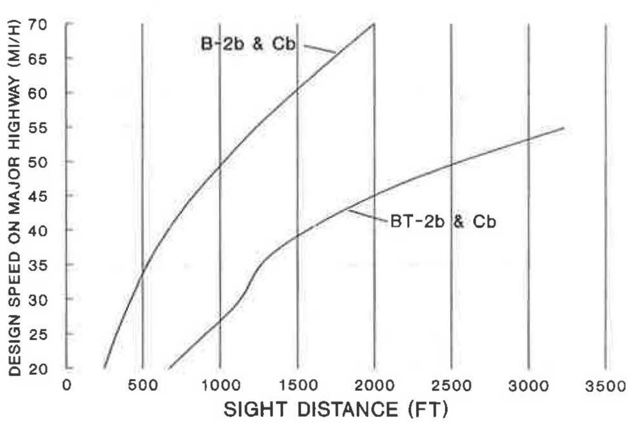

FIGURE 9 Sight distance curves for B–2b & Cb reduced-speed procedure.

of NCHRP Report 270 (*4*) in which findings from several studies are discussed. Olson et al. (*4*) concluded that the AASHTO curves represent a conservative design policy in relation to current and projected vehicle fleets.

Cases III–B and III–C—Turning Left or Right onto a Crossroad

McGee et al. (*5*) stated that Case III–B and III–C is a new policy that had not been considered in editions of the AASHTO design policies before 1984. In 1988, Poe (*6*) generated equations to reproduce the B–2a & Ca curve. A 2.0-sec tailgate time was assumed between the turning vehicle and the oncoming vehicle. This larger time gap resulted in a greater percent difference with the Green Book (*1*) B–2a & Ca curve than those discussed. Mason et al. (*7*) used truck characteristics from the literature to estimate sight distance values for the constant-speed procedure. Mason et al. (*7*) sight distance values were less than the present values because the values for time to accelerate t_{as} cited in the literature were less than those determined from Figure 4.

Equations were developed to reproduce the design curves in Figure 2. These curves were reproduced to within 8 percent accuracy for passenger cars. When truck characteristics were substituted for passenger car characteristics, the resulting sight distances were between 690 and 1,700 ft for the clear-lane procedure (B–1 curve), between 670 and 3,200 ft for the constant-speed procedure (B–2a & Ca curve), and between 670 and 2,500 ft for the reduced-speed procedure (B–2b & Cb curve). These sight distances are for speeds from 20 to 50 mph. However, individuals cannot perceive movement much beyond 800 ft, and a car at 2,000 ft appears the size of a pinhead held at 18 in. (*8*).

The Green Book (*1*) depicts tailgate distance as the minimum distance between the rear bumper of the turning vehicle and the front bumper of the major-road vehicle. The term tailgate implies an inappropriate driving behavior, but the minimum distance is not necessarily an improper action by the major-road driver. Therefore, the term minimum separation may be a better description of that particular dimension.

Design speeds selected for the original design of the roads may not be in agreement with the current functional classification of the roads or the intersection and may not agree with current operations. Initial speed used in the ISD procedure should reflect the functional classification and the operations of the intersection. The term "design speed" could be replaced with "initial speed," which would represent the higher of the intersection design speed or the 85th-percentile speed along the major road. Because major-road drivers are reducing to speeds less than average running speed, the term reduced speed may better describe the events occurring at the intersection.

SUMMARY OF FINDINGS

Crossing Maneuver (Case III–A)

AASHTO's crossing maneuver policy has remained the same in concept, formulation, and in assumed J, t_a, D, and L values since it appeared in the 1954 Blue Book (*3*). AASHTO curves for t_a were established from tests conducted by the Bureau of Public Roads in 1937 (*4*).

Turning Left or Right onto a Crossroad (Cases III–B and III–C)

Turning left or right onto a crossroad is a new design consideration that was not mentioned in earlier editions of AASHTO's design policies (*5*). As currently included in the Green Book (*1*), the case lacks specific information to derive the design curves for determining required passenger car ISD values. Using the information provided and making assumptions for the missing information, the curves or values were reasonably reproduced. Passenger car characteristics were then replaced with truck characteristics to determine the ISD values for trucks. Specific sight distance values are listed in Tables 2–4 for the B–1, B–2a & Ca, and B–2b & Cb curves, respectively.

IMPLICATION OF FINDINGS

ISD values were produced for trucks using equations and driver and vehicle characteristics implied in the Green Book (*1*). Passenger car curves were reproduced to within 8 percent accuracy. Using truck characteristics in the modified equations frequently produced sight distance values greater than those at which drivers can normally detect motion. The constant-speed procedure generated a sight distance of 3,200 ft for a 50-mph major-road design speed, whereas the reduced-speed procedure resulted in a sight distance of 2,500 ft for the same design speed. Operation experience at intersections indicates that sight distances of such magnitude are rarely necessary for safe and efficient operations. Procedures should be based on actual intersection operations through field observations of the intersection characteristics (i.e., major-road vehicles tend to reduce speed more than 5 mph during a minor-road vehicle's turning maneuver).

RECOMMENDATIONS

Several issues were identified for consideration in future versions of the AASHTO geometric design policy for stop-controlled ISD. A preliminary draft copy of the revised Green Book (*9*) was received and reviewed. Differences between the Green Book (*1*) and the draft of the revised Green Book (*9*) are noted after each of the following issues:

- Case III–B and III–C procedures warrant additional discussion regarding the equations used to calculate ISD values. A comprehensive discussion using numeric values for the major-road vehicle's speed reduction would be beneficial. The draft of the revised Green Book (*9*) included the equation for calculating sight distance to the left for a left turn (B–1 procedure). The draft also informally presented, in the written portion of the chapter and on a revised Green Book (*9*), Figure IX–24, equations to produce B–2b & Cb (reduced-speed) sight distance values. Discussion supporting stated assumptions and a comprehensive example of a sight distance calculation would further improve the section.
- The two sight distances used in Case III–B, sight distance to the left—clear-lane condition (B–1 curve)—and sight distance to the right—constant-speed condition (B–2a curve) or reduced-speed condition (B–2b curve)—should be clearly defined. The revised Green Book (*9*) includes a discussion on the B–1 curve that assists in emphasizing the differences between the procedures.
- Consideration should be given to substituting the terms minimum separation, initial speed, and reduced speed in place of tailgate distance, design speed, and average running speed, respectively, in Case III–B and III–C procedures. The revised Green Book (*9*) includes the terms vehicle gap, design speed, and 85 percent of design speed.
- Guidance should be provided on the sight distance dimension along the minor road from the edge of the travel lane to ensure consistent application, i.e., is the assumed 10-ft dimension to the front of the vehicle or to the driver? If it is to the vehicle, the assumed distance between the driver and the front of the vehicle should be stated. (The driver's eye is established as 20 ft behind the edge of pavement or curb line.)
- Case III–B on Figure 1 should reflect the sight distance values for near and far lanes. Sight distance to the right should be longer than the sight distance to the left. Also, consideration should be given to replacing d_1 with ISD_{CL} or ISD_{B-1} (clear-lane intersection sight distance) and d_2 with ISD_{CS} or $ISD_{B-2a-Ca}$ or ISD_{RS} or $ISD_{B-2b\&Cb}$ (constant-speed or reduced-speed intersection sight distance). This change will assist in clearly showing which sight distance procedures are being used. Figure 1 was not modified in the draft of the revised Green Book (*9*).
- Other issues noted in the draft Green Book (*9*) include the following:
  - —The curve for acceleration time versus distance traveled (Figure 3) was updated with information from NCHRP 270 (*4*). The curves for the WB–50 and SU design vehicles were not modified.
  - —The curves for distance traveled versus speed reached (Figure 4) were updated.
  - —Speed of the major-road vehicle is assumed as 85 percent of the design speed on the major roadway. However, the corresponding formulas for the B–2b & Cb procedures do not include any time for reducing speed.
  - —A formula for the sight distance to the left value for Case III is provided. The procedure assumes that the major-road vehicle is driving at 85 percent of the major highway's design speed. Time to accelerate is based on the distance traversed, which is the sum of a 10-ft setback from the stop line, the length of the vehicle, and the distance traveled to cross the opposing lane (assumed as approximately 1.5 times the lane width). A 2-sec gap between the turning vehicle and the approaching vehicle is also stated as being assumed.
  - —Discussion on the sight distance for a P vehicle to turn and attain design speed (B–2a & Ca curve) was not present. Also, the B–2a & Ca curve was not included on the ISD for the at-grade intersections figure (see Figure 1).
  - —The draft chapter (*9*) contains the same comment on required sight distance for trucks that was in the Green Book (*1*). "Required sight distances for trucks making left turns onto a major highway will be substantially longer than for passenger cars. These relationships for trucks can be derived using appropriate assumptions for vehicle acceleration rates and turning paths." The procedure outlined in this discussion can be used to determine the sight distance requirements and implications of trucks at stop-controlled intersections. Pertinent substitutions and minor modifications will be necessary to incorporate AASHTO's proposed revisions to this section of the new geometric design policy.

Most drivers do not have the capability to accurately judge the location and speed of an oncoming vehicle at several of the sight distances produced using the truck characteristics.

Generally speaking, intersections currently operate with sight distances less than those calculated. For practical reasons, ISD procedures should reflect actual field operations. For example, individual parameter values used in the Green Book (*1*) ISD procedures should represent current and future vehicle and driver characteristics, or both, which can be accomplished by explicitly considering gaps in the major-road traffic that are accepted by minor-road drivers.

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The contents of this paper reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of FHWA or the U.S. Department of Transportation.

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Sight Distance Requirements for Symmetrical- and Unsymmetrical-Crest Vertical Curves

SNEHAMAY KHASNABIS AND RAMAKRISHNA R. TADI

Major changes in vehicular design have occurred in the United States during the last 25 years that have affected both the computation and measurement of stopping sight distance (SSD). First, a brief review of the changes in the design parameters of symmetrical-crest vertical curves, as reflected in the 1984 AASHTO manual, is presented. Second, an analytical approach computing the length of unsymmetrical curves to provide SSD requirements, for which no design guidelines are currently available, is presented. Unsymmetrical curves may be warranted in special situations with constrained geometrics, e.g., freeway ramps, grade separation structures, multiple control points, etc. A parameter (γ) is introduced as an indicator of nonsymmetry in the computation of length of unsymmetrical-crest curves. For values of γ less than unity, the procedure presented results in a longer curve length than that used for a symmetrical curve, with the maximum length occurring at $\gamma = 0.38$ for driver's eye and object heights of 3.5 and 0.5 ft, respectively. Overall, unsymmetrical curves required longer lengths than those currently used for symmetrical curves. A complete procedure for setting up unsymmetrical curves to meet SSD requirements is presented along with a technique to locate the highest point. Finally, recommendations are made for further research for formalizing additional design guidelines for unsymmetrical curves.

Sight distance considerations constitute a key element of highway design. The ability of the motorist to see a sufficient distance ahead to perceive potential hazards and to make proper decisions is a major factor in the safe and efficient operation of highways (*1,2*). Motorists must not be trapped in situations where they have neither sufficient time nor distance to take evasive actions. Further, traffic engineers must recognize the importance of interface between vehicular and human factors in the design of highways.

Vehicular design has undergone significant changes in dimensions and operating characteristics during the last 25 years. Also, substantial changes in the mix of vehicles between passenger cars and heavy vehicles have occurred during the last two decades. Lastly, highway users differ in their physical stature and in their psychological attributes. Today's driving population in most countries has somewhat of a different distribution of age groups and sex compared with the early 1960s. Highway design practices should recognize these changes and should attempt to incorporate their effect into the design parameters.

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THE IMPORTANCE OF SIGHT DISTANCE

AASHTO discusses three types of sight distance for consideration in highway design (*2*). There are stopping sight distance (SSD), passing sight distance (PSD), and decision sight distance (DSD). In addition, the importance of sight distance at intersections is mentioned in the AASHTO manual. Of the three types, SSD constitutes the single most important design criterion for highways.

Current highway design standards dictate that at any point on a given roadway, a minimum SSD must be provided. Failure on the part of the roadway designer to provide the minimum SSD may expose the motorist to undue hazards and increase the likelihood of accidents. The provision of minimum PSD although considered desirable will result in inordinately long vertical curves and in high construction costs. DSD is important only in special situations in which there is a likelihood of error in information reception, decision making, or control actions. Clearly, consideration of SSD (rather than PSD or DSD) is of utmost importance in the design of crest curves (*3*). SSD is incorporated in the design of unsymmetrical crest vertical curves. In order to provide continuity, a brief synopsis of symmetrical vertical curves is also presented.

GEOMETRY OF CREST VERTICAL CURVES

The purpose of vertical curves that join two intersecting grade lines of railroads or highways is to smooth out the changes in vertical motion. Vertical curves are designed to contribute to safety, comfort, and appearance of the roadway. These curves are generally parabolic in nature and can be either symmetrical or unsymmetrical. Symmetrical curves are those with equal tangents at the point where the curve is divided equally at the vertical point of intersection (VPI) of the two tangents (Figure 1). The point on the left tangent line where the curve starts is termed the vertical point of curvature (VPC) and the corresponding point where the curve ends on the right tangent is called the vertical point of tangency (VPT).

The majority of vertical curves constructed in the United States are symmetrical in nature. Standards for incorporating sight distance requirements for symmetrical crest curves were originally developed by AASHO in 1965 (*1*) and updated in 1984 (*2*). The rate of change r of slope of a symmetrical vertical curve remains unchanged throughout the length of the curve. The constant value of r is a characteristic feature of the parabolic nature of the curve.

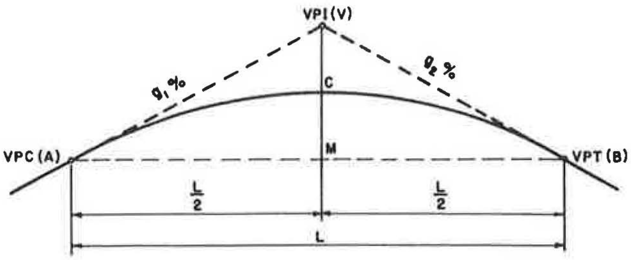

FIGURE 1 Symmetrical-crest vertical curve.

An unsymmetrical curve is characterized by two unequal tangents, resulting in an unequal division of the curve at the VPI (Figure 2). The rates of change of the slope of the two sections of the curve under the two tangents are different and the point under the VPI forms the transition between the two rates. Although unsymmetrical curves are far less common than symmetrical ones, AASHTO states "it is possible that an unsymmetrical curve will fit more closely certain imposed requirements than the usual symmetrical equal-tangent curve" (*2*). Unsymmetrical curves may be warranted in situations with constrained geometrics—roadways with multiple control points, freeway ramps, and grade-separated structures where a minimum vertical clearance between two roadbeds must be provided. Few guidelines are currently available on how to incorporate sight distance requirements in unsymmetrical-crest vertical curves. However, unsymmetrical curves should be used more frequently than before, but because AASHTO does not provide guidelines this analysis can be used to design an unsymmetrical curve should one be needed.

Two aspects of SSD are important from the roadway designer's viewpoint: (a) computation of SSD at various speeds, and (b) measurement of roadway length at crest curves to ensure the provision of SSD. Both computation and measurement represent the use of analytic techniques requiring basic assumptions on vehicular dimensions and geometric features of the roadway. Major changes in vehicular design have occurred in the United States during the last 25 years that have affected both the computation and measurement of SSD. Historically, the critical dimensions (i.e., length, breadth, and height) of passenger cars have decreased because of safety standards, energy consumption, and driver preferences. Also, advances in automotive technology have brought about major changes in vehicular dynamics, including acceleration and deceleration characteristics, speed attainability over gradient sections, and maneuverability around sharp curves (*4*).

First, the changes in the design parameters of symmetrical vertical curves are reviewed to incorporate changes in vehicular design as reflected in the 1984 AASHTO manual (*2*).

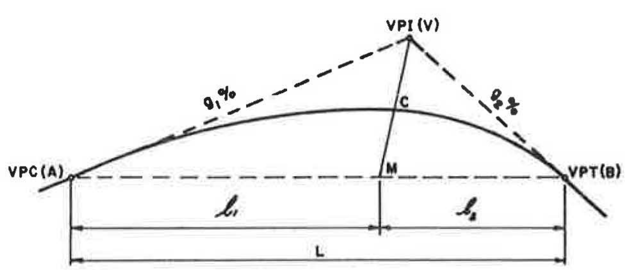

FIGURE 2 Unsymmetrical-crest vertical curve.

Second, the lengths of unsymmetrical curves to provide SSD requirements are computed.

COMPUTATION OF SSD

Equation 1 is used to calculate SSD values as the sum of reaction distance and braking distance. Included are two parameters, reaction time t_r and pavement friction f, and one major variable, speed V.

$$S = 1.47Vt_r + \frac{V^2}{30(f \pm G)} \tag{1}$$

where

S = SSD,
V = speed (mph),
t_r = perception-reaction time (sec),
f = coefficient of friction between tires and pavement, and
G = percent of grade divided by 100.

Perception-Reaction Time

The 1965 AASHO manual (*1*) recommended a design value of 2.5 sec for perception-reaction time (including 1.5 sec for perception and 1.0 sec for reaction) under emergency braking conditions for a typical driver (*1,5,6*). The value of 2.5 sec is based on limited experimental data collected many years ago. Recent studies (*2*) indicate that "a reaction time of 2.5 sec is considered adequate for more complex conditions than those of the various studies, but it is not adequate for the most complex conditions encountered by the driver." Woods (*7*) reported that "perception reaction time is substantially less than the 2.5 second value currently used. . . . The distribution of perception reaction time has an extreme value of 3 seconds." However, the 1984 AASHTO manual (*2*) has retained the earlier assumed value of 2.5 sec for the purpose of computing SSD.

Pavement Friction

Values of coefficient of friction (f) in Equation 1 for wet pavement as used in the 1965 AASHO manual (*1*) were developed almost 30 years ago (*8*). The 1984 AASHTO manual (*2*) recommends f values that are slightly lower than those used in the past. These revised values recognize recent studies that indicate that there is a wide variation in pavement friction that reflects the effect of surface texture on stopping distances (*4*).

Speed

The 1965 AASHO manual (*1*) assumed that top speeds were somewhat lower on wet pavements than on the same surface under dry conditions. On the basis of this assumption, the average running speed rather than design speed was used in computing SSD values. This assumption was questioned in later years as many motorists were found to drive as fast on

TABLE 1 COMPARISON OF MINIMUM SSD VALUES FOR WET PAVEMENT BETWEEN 1965 (*1*) AND 1984 (*2*) AASHTO GUIDELINES (t_r = 2.5 sec)

	1965 AASHO			1984 AASHTO			1983 AUTHORS' RECOMMENDATION		
Design Speed	Assumed Speed	Coef. of Friction	SSD	Assumed Speed for Condition	Coef. of Friction	SSD	Assumed Speed for Condition	Coef. of Friction	SSD
(MPH)	(MPH)	(f)	(feet)	(MPH)	(f)	(feet)	(MPH)	(f)	(feet)
30	28	0.36	200	28-30	0.35	200-225	29	0.35	180
40	36	0.33	275	36-40	0.32	275-325	38	0.31	300
50	44	0.31	350	44-50	0.30	400-475	47	0.28	440
60	52	0.30	475	52-60	0.29	525-650	56	0.27	600
70	58	0.29	600	58-70	0.28	625-850	64	0.26	760

wet pavements (*9*). The 1984 manual suggests the use of design speed rather than average running speed. Table 1 presents a comparison of SSD values computed using the two versions of the AASHTO manual (*1,2*). Also included in Table 1 is a set of SSD values suggested in 1983, by Khasnabis and Reddy (*8*), before publication of the 1984 AASHTO manual (*2*).

The higher values indicate the effect of design speed and revised friction (f) values.

Symmetrical-Crest Curves

The expression given by Hickerson (*10*) for computing the length of a symmetrical-crest vertical curve is as follows:

$$L = \frac{AS^2}{100(\sqrt{2h_1} + \sqrt{2h_2})^2} \quad \text{if } S < L \tag{2}$$

where

- L = required length of vertical curve (ft),
- S = required sight distance (SSD in this case) (ft),
- A = algebraic difference in grades (percent),
- h_1 = driver's eye height (ft), and
- h_2 = object height (ft).

Equation 2 can be rewritten as

$$L = KA \tag{3}$$

where

$$K = \frac{S^2}{100(\sqrt{2h_1} + \sqrt{2h_2})^2} \tag{4}$$

In Equation 3, L is the length of crest curve needed for each percent algebraic change in grade A and is a convenient expression of the design control. For each design speed V (which determines the value of S) and a given combination of driver's eye height (h_1) and object height (h_2), K is thus the rate of vertical curvature (or the length per unit value of A) needed to provide the required SSD. Situations in which $S > L$ are somewhat uncommon and are not considered critical from the point of view of design.

Eye Height (h_1)

The 1965 AASHO guidelines (*1*) recommended an eye height value of 3.75 ft (45 in.) on the basis of the actual observation of vehicular dimensions up to the mid-1960s. Passenger vehicle design trends have resulted in lower vehicular heights and thus in lower driver's eye height values (*9,11,12*). Eye height has finally stabilized and little change, if any, is likely to occur during the next decade.

The 1984 AASHTO manual (*2*) recognizes these changes and calls for using a driver eye height of 3.5 ft for the design of crest vertical curves. In their 1983 study, Khasnabis and Reddy (*8*) suggested the use of 3.5 ft: "It appears that height of 3.5 feet (as opposed to the currently used 3.75 ft) would better represent the eye height of a majority of passenger cars in the United States." Even this reduction of eye height is considered by some experts to be insufficient to reflect a majority of passenger cars in the United States. For example, Woods (*7*) reported a 95 percentile value of 3.23 ft and has shown that the assumed value of 3.5 ft at a design speed of 70 mph used to compute the length of crest vertical curves would result in an α level of 25 percent (corresponding to Type 1 error).

Object Height (h_2)

In 1940, AASHO selected a design object length of 4 in. on the basis of optimizing the trade-off between object height and the required length of vertical curve (*11*). This 4-in. object height represents a compromise solution between construction cost and the need to maintain a clear sight line to the pavement, which is evident from the fact that this requirement was changed to 6 in. in 1965 and has been retained at 6 in. in 1984. To quote the 1984 manual (*2*), a 6-in. height is representative of the "lowest object that can be perceived as a hazard by a driver in time to stop before reaching it." Khasnabis and Reddy (*8*) discussed the implication of object height and demonstrated that even a small reduction in eye height will result in a substantially longer crest curve.

Required Length of Symmetrical Curve

For h_1 = 3.5 ft and h_2 = 0.5 ft, Equation 4 can be rewritten as

$$K = \frac{S^2}{1{,}329} \tag{5}$$

TABLE 2 COMPARISON OF CREST VERTICAL CURVE LENGTHS ON THE BASIS OF SSD REQUIREMENTS

Design Speed (MPH)	K = Rate of Vertical Curvature Length (ft) per Percent of A		
	1965 AASHO	1984 AASHTO	1983 AUTHORS' RECOMMENDATION
30	28	30-38	24
40	55	60-80	68
50	85	110-160	146
60	160	190-310	271
70	255	290-540	435

In Table 2, a set of K values is presented for use in comparing crest curve lengths between the 1965 and 1984 AASHTO guidelines (*1,2*) and the 1983 recommendation of Khasnabis and Reddy (*8*). Higher values of K for 1984 in Table 2 reflect the consequence of increased SSD values as shown in Table 1 plus a reduction in h_1 from 3.75 to 3.5 ft.

UNSYMMETRICAL-CREST CURVES

Unsymmetrical vertical curves are not common, but as AASHTO mentions, "on certain occasions, because of critical clearance or other controls, the use of unsymmetrical vertical curves may be required" (*2*,p.305). Figure 2 shows that the points of intersection of the vertical line through the VPI with the curve at C and with the long chord at M are not the midpoints of the curve or the chord, respectively. An unsymmetrical curve is not divided into two equal halves around the VPI. Thus, unlike a symmetrical curve with a constant rate of change in the slope, an unsymmetrical curve is characterized by two rates of change in slope—one for the left portion of the curve from the VPC to the VPI and another for the right section of the curve from the VPT to the VPI. This difference is explained in the derivations of Appendix A.

Length Requirement of Unsymmetrical Curves

In Figure 2, let

l_1 = length of curve AC (left section) (ft), and
l_2 = length of curve BC (right section) (ft)

so that

$$L = l_1 + l_2$$

where L = length of entire curve (ft). Setting

g_1 = percent grade of left tangent AV, and
g_2 = percent grade of right tangent BV,

the algebraic difference in percent grade of the tangents is

$$A = |g_2| + |g_1|$$

and $\gamma = l_1/l_2$. Deviation of the value of γ from unity is a measure of the degree of nonsymmetry.

Thus, it can be shown (Appendix A) that

$$L = \frac{AS^2}{200(\sqrt{h_1\gamma} + \sqrt{h_2/\gamma})^2} \tag{6}$$

Alternatively,

$$l_2 = \frac{AS^2\gamma}{200(\gamma\sqrt{h_1} + \sqrt{h_2})^2(1 + \gamma)} \tag{7}$$

$$l_1 = \gamma l_2, \tag{8}$$

so that L can be computed from

$$L = l_1 + l_2. \tag{9}$$

Note that $\gamma > 1$ when $l_1 > l_2$ or $0 < \gamma \leq 1$, when $l_1 \leq l_2$.

In practice, γ is likely to be within the range of 0.25 to 2 and when γ equals unity, the curve becomes symmetrical. Further, deviation of the γ value from unity (in either direction) is an index of the degree of nonsymmetry. The length of the unsymmetrical curve L needed to provide a required sight distance S can be computed from Equation 6. Alternatively, Equation 6 can be written as

$$L = K^1A \tag{10}$$

where

$$K^1 = \frac{S^2}{200(\sqrt{h_1\gamma} + \sqrt{h_2/\gamma})^2} \tag{11}$$

K^1 is the length of crest curve needed for each percent of algebraic change in grade A. For each design speed V (which determines the value of S), a combination of h_1, h_2 (eye and object heights), and known γ value (degree of nonsymmetry), K^1 is the rate of unsymmetrical vertical curvature needed to provide the required SSD.

Alternatively, Equations 7–9 can be rewritten as

$$k_2^1 = \frac{S^2\gamma}{200(\gamma\sqrt{h_1} + \sqrt{h_2})^2\,(1 + \gamma)} \tag{12}$$

$$k_1^1 = \gamma k_2^1 \tag{13}$$

$$K^1 = k_2^1 + k_1^1 \tag{14}$$

where

k_2^1 = rate of vertical curvature of the right section,
k_1^1 = rate of vertical curvature of the left section, and

K^1 = rate of unsymmetrical vertical curvature as defined before.

If r_1 and r_2 are the rates of change of slope of the left portion (from VPC to VPI) and the right portion (from VPT to VPI), respectively,

$$r_1 = \frac{A}{L}\frac{l_2}{l_1} \quad (15)$$

and

$$r_2 = \frac{A}{L}\frac{l_1}{l_2} \quad (16)$$

By contrast, a symmetrical curve has a constant rate of change of slope r from the VPC to the VPT equal to A/L.

If the object of the analysis is to derive estimates of k_1^1 and k_2^1 separately, Equations 12 and 13 can be used for such purposes. Table 3 presents a set of K^1 values, using Equation 11 and the same data used in Table 2, plus the additional parameter γ. At $\gamma = 1$, the K^1 values in Table 3 are similar to those in Table 2. Tables 4 and 5 present the values of k_1^1, k_2^1, and K^1 as computed by Equations 12–14 for the same set of γ values. Note that at $\gamma = 1$, k_1^1 and k_2^1 are the same, representing a special case of a symmetrical curve.

Design Guidelines

The K^1 values have also been plotted in Figures 3 and 4 for the low and high values of SSD for specific design speeds, for assumed values of γ ranging from 0.25 to 2.0. Because guidelines for length requirements for unsymmetrical curves are currently unavailable, Figures 3 and 4 can be used to compute the length of unsymmetrical-crest curve for a specific value of γ.

From data presented in Tables 3–5, as well as Figures 3 and 4, length requirements of unsymmetrical curves (for the assumed value of h_1 and h_2) exceed those of symmetrical curves when the numerical value of γ is less than unity. However, the change in the K^1 value as a result of a reduction in the value of γ is not monotonic. For example, the reduction in γ from 1.0 to 0.5 brings about a significant increase in the value of K^1. However, a reduction in γ from 0.5 to 0.25 causes a small reduction in K^1 because of the nature of the mathematical function (Equation 11) used for computing K^1.

Calculus was used to identify the value of γ at which K^1 is maximized. By taking the first derivative of Equation 11 with

TABLE 3 K^1 VALUES FOR DIFFERENT SPEEDS AND γ VALUES EXPRESSED AS RATE OF CURVATURE (IN FEET PER UNIT VALUE OF A) FOR UNSYMMETRICAL CURVES

Design Speed (MPH)	SSD (Ft)	γ Value									
		0.25		0.50		1.00		1.50		2.00	
		Low	High	Low	High	Low	High	Low	High	Low	High
30	200-225	36	46	37	47	30	38	24	31	20	26
40	275-325	68	96	70	98	57	79	46	64	38	53
50	400-475	145	204	148	209	120	170	97	137	81	114
60	524-650	249	382	255	392	207	318	167	257	139	213
70	625-850	354	654	362	670	294	544	237	439	197	365

TABLE 4 k_1^1, k_2^1, K^1 VALUES FOR DIFFERENT SPEED AND γ VALUES (IN FEET PER UNIT VALUE OF A) FOR UNSYMMETRICAL CURVES—LOW END

Design Speed (MPH)	SSD (Ft)	γ Value														
		0.25			0.50			1.00			1.50			2.00		
		k_1^1	k_2^1	K^1	k_1^1	k_2^1	K^1	k_1^1	k_2^1	K^1	k_1^1	k_2^1	K^1	k_1^1	k_2^1	K^1
30	200	7	29	36	12	25	37	15	15	30	14.6	9.7	24	13	7	20
40	275	14	55	69	23	47	70	28.5	28.5	57	27.6	18.4	46	25	13	38
50	400	29	116	145	49	99	148	60	60	120	58	39	97	54	27	81
60	525	50	200	250	85	170	255	103.5	103.5	207	100	67	167	93	46	139
70	625	71	283	354	121	241	362	147	147	294	142	95	237	132	66	198

TABLE 5 k_1^1, k_2^1, K^1 VALUES FOR DIFFERENT SPEED AND γ VALUES (IN FEET PER UNIT VALUE OF A) FOR UNSYMMETRICAL CURVES—HIGH END

Design Speed (MPH)	SSD (Ft)	γ Value														
		0.25			0.50			1.00			1.50			2.00		
		k_1^1	k_2^1	K^1	k_1^1	k_2^1	K^1	k_1^1	k_2^1	K^1	k_1^1	k_2^1	K^1	k_1^1	k_2^1	K^1
30	225	9	37	46	16	31	47	19	19	38	19	12	31	17	9	26
40	325	19	77	96	33	65	98	39.5	39.5	79	38	26	64	35	18	53
50	475	41	163	204	70	139	209	85	85	170	82	55	137	76	38	114
60	650	77	306	383	131	261	392	159	159	318	154	103	257	142	71	213
70	850	131	524	655	224	446	670	272	272	544	263	176	439	243	122	365

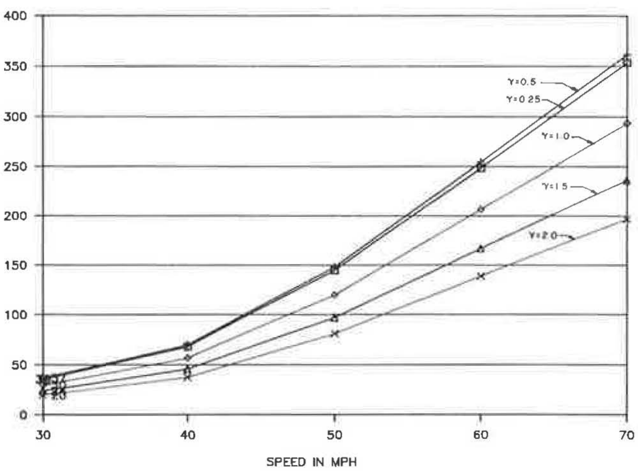

FIGURE 3 K^1 values for different speeds for lower SSD values.

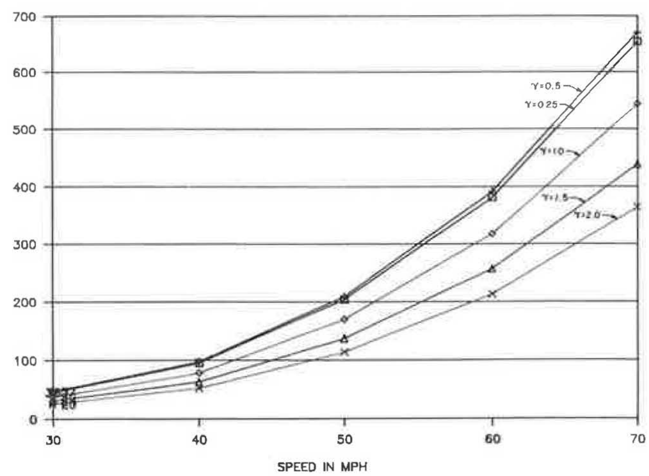

FIGURE 4 K^1 values for different speeds for higher SSD values.

respect to γ and setting it equal to zero, K^1 is maximized at $\gamma_{critical} = \sqrt{h_2}/\sqrt{h_1}$ and for the assumed values of $h_1 = 3.5$ ft and $h_2 = 0.50$ ft, this $\gamma_{critical}$ value is 0.38.

In an effort to further demonstrate the relationship between K^1 and γ, K^1 values for different speed data for γ values ranging from 0.20 to 0.60 are presented in Table 6. From this table, the curve length is indeed maximized when γ approaches 0.40, which closely approximates the γ value of 0.38 derived by calculus.

Figures 3 and 4 indicate that unsymmetrical curves, if designed as symmetrical curves according to current AASHTO guidelines, may not provide safe stopping distances for γ parameters less than unity. Additionally, the required length is maximized at $\gamma = 0.40$. For example, for a design speed of 50 mph, the lower range of the K value of a symmetrical curve is 110 ft (Figure 3 and Table 2). For an unsymmetrical curve for $\gamma = 0.50$, the corresponding value is 148 ft (Table 3 and Figure 2). Further, for the unsymmetrical curve for $\gamma = 0.50$, k_1^1 and k_2^1 are estimated as 49 and 99 ft, respectively, making a total length (K^1) of 148 ft (Table 4). Clearly, more research is needed before specific guidelines could be formalized in this respect.

For γ parameters exceeding unity, the reverse might appear to be true, but K^1 values for γ exceeding unity should not be used for design purposes because the derivation of K^1 values is based on assumed direction of travel from the left to the right, with eye height h_1 and object height h_2 located at left and right, respectively. Because a majority of two-lane rural highways are for two-way travel, clearly the directions of h_1 and h_2 could reverse themselves from those assumed in the derivation. Thus, the curves in Figures 3 and 4 to the right of the line representing $\gamma = 1$ (symmetrical curves) ought to be used with extreme caution, only when the roadway is for one-way travel. For all other cases, the γ value should be computed as

$$\gamma = (l_1/l_2) \text{ or } (l_2/l_1)$$

whichever is smaller, and the curves to the left of the line representing $\gamma = 1$ should be used. Thus, the curves in Figures 3 and 4, to the right of the line representing $\gamma = 1$ (or K^1 values for γ values exceeding unity) are purely for academic interest with little practical value. This situation is somewhat analogous to SSD values computed for dry pavements, although

TABLE 6 K^1 VALUES (IN FEET PER UNIT VALUE OF A) FOR VARIOUS VALUES OF γ AND DESIGN SPEEDS FOR UNSYMMETRICAL VERTICAL CURVES

γ Value	Design Speed (MPH)									
	30		40		50		60		70	
	Low	High	Low	High	Low	High	Low	High	Low	High
0.20	34	34	65	90	137	193	236	361	334	618
0.30	37	37	70.5	99	149	210	257	394	364	674
0.40	38	38	71.4	100	151	213	260	399	369	682
0.50	37	37	70	99	148	209	255	392	362	670
0.60	36	36	68	95	143	202	247	379	350	647

for all practical considerations wet-pavement conditions prevail in highway design.

Highest Point on a Crest Curve

Both for symmetrical and unsymmetrical curves, the turning point can be under the vertical point of intersection (VPI) of the two tangents, which is likely to happen only in specific situations; more often than not, the turning point is likely to be located either to the right or the left of the VPI.

The procedure for locating the turning point of a vertical curve includes taking the first derivative of the expression for computing the elevation of the curve (expressed in terms of x, the distance from the VPC), setting it equal to zero, and solving for x. The procedure results in the following equation for symmetrical curves:

$$X_{TP} = \frac{g_1 L}{|g_1| + |g_2|} = \frac{g_1 L}{A} \qquad (17)$$

where $A = |g_1| + |g_2|$.

The highest point of a symmetrical curve is more likely to be located either to the left or right of the VPI, depending on whether $|g_1|$ is less or more than $|g_2|$, respectively. Only when $|g_1|$ equals $|g_2|$ is the highest point exactly under the VPI, i.e., X_{TP} equals $L/2$ in Equation 17.

Following the same procedure and approaching both from the left tangent (VPC) and the right tangent (VPT) for an unsymmetrical curve, the turning point(s) can be located using the following expressions, as derived in Appendix B:

$$X_{TPL} = \frac{g_1 L}{A} \cdot \frac{l_1}{l_2} \qquad X_{TPL} \leq l_1 \qquad (18)$$

or

$$X_{TPR} = \frac{g_2 L}{A} \cdot \frac{l_2}{l_1} \qquad X_{TPR} \leq l_2 \qquad (19)$$

where X_{TPL} equals the location of the turning point measured from the left (VPC), and X_{TPR} equals the location of the turning point measured from the right (VPT). The purpose of the inequalities in expressions 18 and 19 is to ensure that the turning point is contained within the prescribed length of the l_1 or l_2 value. Only one of these two equations will prevail in most cases. Only under a specific set of geometric combination of lengths and grades will the turning point for unsymmetrical curves be under the VPI. This condition will happen only when the points X_{TPL} and X_{TPR} converge under the VPI as derived as follows:

$$X_{TPL} = g_1 \frac{L}{A} \cdot \frac{l_1}{l_2} = l_1 \qquad (20)$$

or

$$\frac{g_1 L}{A l_2} = 1 \qquad (21)$$

and

$$X_{TPR} = \frac{g_2 L}{A} \cdot \frac{l_2}{l_1} = l_2 \qquad (22)$$

or

$$\frac{g_2 L}{A l_1} = 1 \qquad (23)$$

Equations 21 and 23 are complements of each other, representing the rare case when the turning point, whether approached from the left or the right, will be under the VPI. Last, when $|g_1|$ equals $|g_2|$, and l_1 equals l_2, both X_{TPL} and X_{TPR} are equal to $L/2$. In this case, the vertical curve is a symmetrical curve (see Equation 15).

APPLICATION

For example, given $g_1 = +3$ percent, $g_2 = -4$ percent, $\gamma = 0.5$; design speed $V = 50$ mph; and elevation of the VPC $= E_{VPC} = 100$ ft at Station 50 + 00; it may be required to

1. Compute the length of the unsymmetrical curve for SSD condition,
2. Construct the complete vertical curve, and
3. Locate the highest point and its elevation.

From Table 1, at a design speed of 50 mph, the range of SSD is between 400 and 475 ft. The lower value is used for this solution. From Table 4, at a design speed of 50 mph, $k_1^1 = 49$ ft and $k_2^1 = 99$ ft; approximately, $k_1^1 = 50$ ft and $k_2^1 = 100$ ft. $A = |g_1| + |g_1| = 7$, so that $l_1 = 7 \times 50$ ft $= 350$ ft, and $l_2 = 7 \times 100$ ft $= 700$ ft, so that $L = l_1 + l_2 = 1{,}050$ ft.
[Horizontal distances are measured in stations (100 ft) and vertical distances are feet.]

The results are as follows:

$E_{VPC} = E_A = 100$ ft (given)

$$E_{VPI} = E_{VPC} + g_1x_1 = 100 + 3 \times 3.5 = 110.5 \text{ ft}$$

$$E_{VPT} = E_{VPI} - g_2x_2 = 110.5 - 4 \times 7 = 82.5 \text{ ft}$$

$$r_1 = \frac{A}{L} \cdot \frac{l_2}{l_1} = \frac{-7}{10.5} \times \frac{7}{3.5} = -1.333$$

$$r_2 = \frac{A}{L} \cdot \frac{l_1}{l_2} = \frac{-7}{10.5} \times \frac{3.5}{7} = -0.3333$$

In order to construct the curve from the left,

$$E_{x1} = E_{VPC} + g_1x_1 + {}^1\!/_2 r_1x_1^2 \qquad (0 \le x_2 < l_1)$$

$$= 100 + 3x_1 - {}^1\!/_2 \times 1.333 \times x_1^2 \quad (0 \le x_1 \le 3.5) \qquad (24)$$

In order to construct the curve from the right,

$$E_{x2} = E_{VPT} + g_2x_2 - {}^1\!/_2 r_2x_2^2 \quad (0 \le x_2 < l_2)$$

$$= 82.5 + 4x_2 - {}^1\!/_2 \times 0.33 \times x_2^2 \quad (0 \le x_2 \le 7) \qquad (25)$$

The complete construction of the curve is presented in Table 7 using Equations 24 and 25. In Column 5 of Table 7, the changes in change in elevation for every station are presented. The values approximate -0.33 (r_1) and -1.33 (r_2) for the left and the right portions of the curve, respectively, because the rates of change of the slope for the left section and right section are constant, being equal to r_1 and r_2, respectively. Further, the change in change in elevation per station length is equal to the rate of change of slope.

In order to locate the turning point,

$$X_{TPL} = \frac{g_1L}{A} \cdot \frac{l_1}{l_2} = \frac{3 \times 10.5}{7} \times \frac{3.5}{7} = 2.25$$

(which is alright, because it is less than 3.5). Also,

$$X_{TPR} = \frac{g_1L}{A} \cdot \frac{l_2}{l_1} = \frac{4 \times 10.5}{7} \times \frac{7}{3.5} = 12$$

(which is too large, because it exceeds 7).

Thus, the turning point is located within the l_1 regime (left portion of the curve) at $x_1 = 2.25$, as its elevation is

$$Ex_1 = 100 + 3x_1 - {}^1\!/_2 \times 1.333x_1^2$$

$$= 100 + 3 \times 2.25 - {}^1\!/_2 \times 1.333 \times (2.25)^2 = 103.37 \text{ ft}$$

CONCLUSION

Changes in design parameters of symmetrical-crest vertical curves have been reviewed in an effort to incorporate changes in vehicular design, and a procedure for computing length requirements of unsymmetrical-crest curves was presented for which no design guidelines are currently available. The following conclusions were obtained.

Symmetrical Curves

The 1984 AASHTO manual (*2*) recommends the use of slightly longer SSD values than those in the 1965 AASHO manual (*1*). These longer lengths are the results of changes in the assumed value of pavement friction and the use of a range of speed. During the last 30 years, there has been a gradual reduction in the height of passenger cars with a smaller reduction in vehicular eye heights. The new AASHTO procedure incorporating these changes results in longer vertical curves. The required length of vertical curves is much more sensitive to object height than to eye height. However, the original object height of 6 in. has been retained in the 1984 AASHTO manual (*2*).

Unsymmetrical Curves

Parameter γ introduced as an indicator of nonsymmetry can be incorporated into the computation of lengths of unsymmetrical curves. For values of γ less than unity, the procedure presented results in longer curve lengths than those used for symmetrical curves, with the maximum length occurring at $\gamma_{critical} = 0.38$. For values of γ exceeding unity, the procedure results in shorter curve lengths. However, caution is recommended to the highway engineer in the use of the parameter exceeding unity. For two-way travel, because the direction of eye and object are interchangeable, the use of the smaller of the two values of γ (l_1/l_2 and l_2/l_1) is recommended for computing the length of unsymmetrical curves.

TABLE 7 CONSTRUCTION OF UNSYMMETRICAL VERTICAL CURVE GIVEN $g_1 = 3$ PERCENT, $g_2 = -4$ PERCENT, $l_1 = 350$ ft, $l_2 = 700$ ft, AND $\gamma = 0.5$

Station (1)	X1 or X2 (Station) (2)	Elevation on Curve Ex1 or Ex2 ft (3)	Change in Elevation ft/Station (4)	Change in Change in Elevation ft/Station^2 (5)
50+00	0 (VPC)	100		
50+50	0.50	101.33		
51+50	1.50	102.99	1.66	
52+50	2.50	103.33	-0.34	-1.32 (r1)
53+50	3.50 (VPI)	102.33	-1.00	-1.34 (r1)
53+50	7(VPI)	102.33		
54+50	6	100.49	-1.84	
55+50	5	98.33	-2.16	-0.32 (r2)
56+50	4	95.83	-2.50	-0.34 (r2)
57+50	3	92.99	-2.84	-0.35 (r2)
58+50	2	89.83	-3.16	-0.32 (r2)
59+50	1	86.33	-3.50	-0.34 (r2)
60+50	0 (VPT)	82.50	-3.83	-0.33 (r2)

Thus, the procedure suggested will always result in longer curve lengths for unsymmetrical curves than those that are currently used for symmetrical curves. Further research is needed before specific design guidelines for unsymmetrical curves can be formalized.

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APPENDIX A

LENGTH OF AN UNSYMMETRICAL VERTICAL CURVE

Setting r_1 equal to rate of change of slope of the left portion of the curve from A to V and r_2 equal to rate of change of slope of the right portion of the curve from B to V, Hickerson (*9*) has shown that

$$r_1 = \frac{A}{L} \cdot \frac{l_2}{l_1} \text{ and } r_2 = \frac{A}{L} \cdot \frac{l_1}{l_2}$$

Because $l_1/l_2 = \gamma$, $r_1 = A/L\gamma$ and $r_2 = A\gamma/L$. [Horizontal distances are measured in units of stations (100 ft) and vertical distances in feet.]

By the parabolic law, offsets vary as the square of the distance. Hence, from Figure A1,

$$h_1 = {}^1/_2 r_1 d_1^2 \text{ and } h_2 = {}^1/_2 r_2 d_2^2$$

hence,

$$d_1^2 = \frac{2h_1}{r_1} \text{ and } d_2^2 = \frac{2h_2}{r_2}$$

Substituting the values of r_1 and r_2,

$$d_1^2 = \frac{2h_1 L\gamma}{A} \text{ and } \frac{2h_2 L}{A\gamma}$$

Therefore,

$$S = d_1 + d_2$$
$$= \sqrt{\frac{2h_1 L\gamma}{A}} + \sqrt{\frac{2h_2 L}{A\gamma}}$$
$$= \sqrt{\frac{2L}{A}} \times (\sqrt{h_1\gamma} + \sqrt{h_2/\gamma})$$

Squaring,

$$S^2 = \frac{2L}{A} \times (\sqrt{h_1\gamma} + \sqrt{h_2/\gamma})^2$$

Solving for L,

$$L = \frac{AS^2}{2(\sqrt{h_1\gamma} + \sqrt{h_2/\gamma})^2}$$

When both horizontal and vertical distances are measured in feet, this expression can be rewritten as

$$L = \frac{AS^2}{200(\sqrt{h_1\gamma} + \sqrt{h_2/\gamma})^2}$$

To compute the length l_1 and l_2 separately, refer again to Figure A1, from which it can be shown (*9*) that

$$r_1 = \frac{A}{L} \cdot \frac{l_2}{l_1} \text{ and } r_2 = \frac{A}{L} \cdot \frac{l_1}{l_2}$$

$$e = {}^1/_2 r_1 l_1^2 = {}^1/_2 r_2 l_2^2 = \frac{A}{2L} \cdot l_1 l_2$$

[Horizontal distances are measured in units of stations (100 ft) and vertical distances are measured in feet.] Because offsets vary as the square of the distance,

$$h_1 = {}^1/_2 r_1 d_1^2 \text{ and } h_2 = {}^1/_2 r_2 d_2^2$$

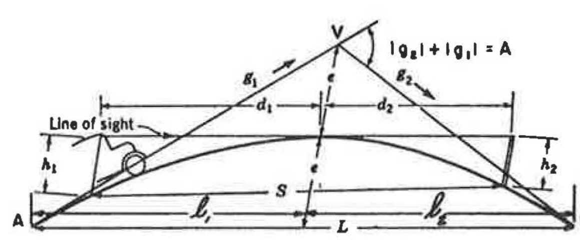

FIGURE A1 Sight distance over vertical curve when S < L (*10*).

or

$$\left(\frac{h_1}{e}\right) = \left(\frac{d_1}{l_1}\right)^2 \text{ and } \left(\frac{h_2}{e}\right) = \left(\frac{d_2}{l_2}\right)^2.$$

or

$$d_1 = l_1\sqrt{\frac{h_1}{e}} \text{ and } d_2 = l_2\sqrt{\frac{h_2}{e}}$$

Now,

$$S = d_1 + d_2 = \frac{1}{\sqrt{e}}(l_1\sqrt{h_1} + l_2\sqrt{h_2})$$

Because

$$L = l_1 + l_2 = l_2(1 + \gamma)$$

it follows that

$$e = \frac{A}{2L}\cdot l_1 l_2 = \frac{A\gamma l_2^2}{2l_2(1+\gamma)} = \frac{A\gamma l_2}{2(1+\gamma)}$$

Thus,

$$S = \sqrt{\frac{2(1+\gamma)}{A\gamma l_2}}\cdot(\gamma l_2\sqrt{h_1} + l_2\sqrt{h_2})$$

$$= \sqrt{\frac{2l_2(1+\gamma)}{A\gamma}}\cdot(\gamma\sqrt{h_1} + \sqrt{h_2})$$

or

$$S^2 = \frac{2l_2(1+\gamma)}{A\gamma}\cdot(\gamma\sqrt{h_1} + \sqrt{h_2})^2$$

$$l_2 = \frac{AS^2\gamma}{2(1+\gamma)(\gamma\sqrt{h_1} + \sqrt{h_2})^2}$$

When both horizontal and vertical distances are measured in feet, this expression can be rewritten as

$$l_2 = \frac{AS^2\gamma}{200(1+\gamma)(\gamma\sqrt{h_1} + \sqrt{h_2})^2}$$

and by definition,

$$l_1 = \gamma l_2$$

because

$$L = l_1 + l_2$$

APPENDIX B

TURNING POINT OF AN UNSYMMETRICAL CURVE

It can be shown from Figures B1 and B2 that

$$Ex_1 = E_A + g_1x_1 + {}^1\!/_2 r_1x_1^2$$

when measured from the left

$$Ex_2 = E_B + g_2x_2 + {}^1\!/_2 r_2x_2^1$$

when measured from the right where Ex_i = elevation at point x_i on the curve, with $0 < x_1 \le l_1$ (from the left) and $0 < x_2 \le l_2$ (from the right).

To locate the turning point, set

$$\frac{dEx_i}{dx_i} = 0$$

to yield $g_1 + \gamma_1x_1 = 0$ and $g_2 + \gamma_2x_2 = 0$. At the turning point, $x_1 = X_{TPL}$ and $x_2 = X_{TPR}$, where X_{TPL} = location of turning point from the left and X_{TPR} = location of turning point from the right.

Thus,

$$X_{TPL} = \frac{g_1}{r_1}$$

$$= \frac{g_1L}{A}\cdot\frac{l_1}{l_2}$$

and

$$X_{TPR} = \frac{g_2}{r_2}$$

$$= \frac{g_2L}{A}\cdot\frac{l_2}{l_1}$$

(because $r_1 = Al_2/Ll_1$ and $r_2 = Al_1/Ll_2$).

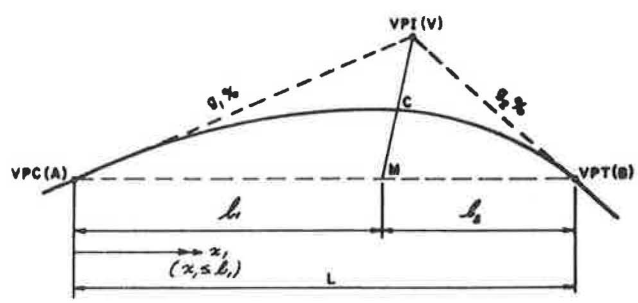

FIGURE B1 Unsymmetrical vertical curve with distance measured from the left.

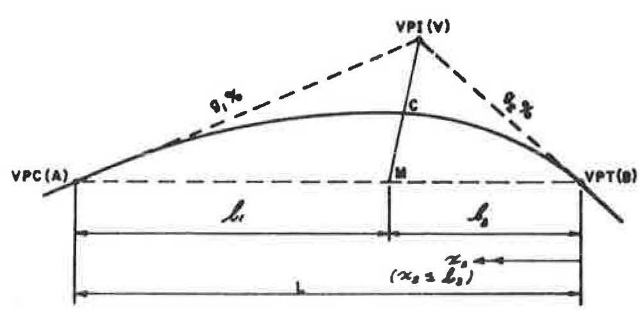

FIGURE B2 Unsymmetrical vertical curve with distance measured from the right.