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A Microscopic Inspection of the Operational Aspects of Urban Interchanges

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Paul Bennett Wellington Dorothy

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A MICROSCOPIC INSPECTION OF THE OPERATIONAL ASPECTS OF URBAN INTERCHANGES

By

Paul Bennett Wellington Dorothy

A DISSERTATION

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ABSTRACT

A Microscopic Inspection of the Operational Aspects of Urban Interchanges

By

Paul Bennett Wellington Dorothy

The Michigan Department of Transportation (MDOT) is considering the much needed rehabilitation and upgrading of many interchanges found in urban environments. Thus, Michigan State University (MSU) undertook an effort to evaluate the appropriateness of an urban interchange geometric configuration, the Single Point Urban Interchange (SPUI), as an alternative design to those presently used by MDOT. In particular, the Michigan Urban Diamond Interchange (MUDI) and the traditional diamond were investigated.

A field review was conducted to collect information about the geometric design, signal operation, pedestrian control and pavement markings of SPUIs, as none currently exist in Michigan. The field review showed that the design and operation of SPUIs vary greatly from state to state. Thus, the SPUI and MUDI designs were computer modeled to facilitate a comparison of their respective operational characteristics. A traditional diamond was also modeled to generate a frame of reference.

The results showed that the SPUI operation is adversely affected with the addition of frontage roads. MUDI operation, in most situations, is superior to that of either a SPUI and diamond interchange configuration. Also, there was less migration of delay to downstream intersections with a MUDI configuration than with either a SPUI or diamond. Finally, MUDI operation, in most situations, is

insensitive to the proximity of the closest downstream node, while the SPUI operation is sensitive.

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I am grateful to the Michigan Department of Transportation for sponsoring the research contained in this document. The advice and cooperation of the MDOT staff helped me to achieve my goal. I would like to single out Laura Aylsworth-Bonzelet and John Saller for their tremendous effort during the field review.

I would like to extend my gratitude to all my friends and colleagues at Michigan State University for their friendship and support. Further, I would like to thank my friends outside the university environment, especially "The Horsemen," for helping me keep my sanity during this endeavor.

I would close with a quote from two of the philosophers who have helped to shape what I have become.

"He who knows when he can fight and when he cannot will be victorious." –Sun Tzu

"Sweet!" –Cartman

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Chapter 1

INTRODUCTION

1.1 Introduction

As Michigan marks its 100th year of auto manufacturing, it should also be noted that freeways in the Detroit area have been in service since 1942. The first 11 kilometers (7 miles) were constructed in 1942 to reduce the travel time of workers going from Detroit to the World War II bomber plant at Willow Run. Many of the early interchanges preceded the Interstate system and, thus, Interstate design standards. The Michigan Department of Transportation (MDOT) is considering the much needed rehabilitation and upgrading of many of these interchanges located in the urban environments. MDOT and Michigan State University (MSU) undertook a joint effort to evaluate the appropriateness of an urban interchange geometric configuration, the Single Point Urban Interchange (SPUI) (Figures 1.1 and 1.2), as an alternative design to those presently used by MDOT. In particular, the Michigan Urban Diamond Interchange (MUDI) (Figure 1.3) and the traditional diamond (Figure 1.4) were investigated.

Most of the pre-interstate freeway interchanges in the city of Detroit and its environs are directional, partial cloverleaf and diamond interchanges. Directional interchanges are normally used to allow a freeway to interchange with another freeway. Conversely, partial cloverleaf interchanges are often used when a freeway interchanges traffic with a major arterial, such as a state trunkline. The loop ramps of the partial cloverleaf accommodate the left-turning movements, thus reducing conflicts on the major arterial. Finally, the simplest and

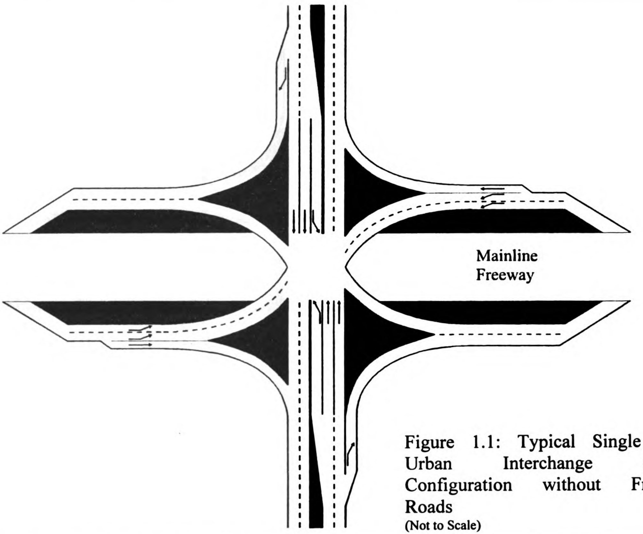


Figure 1.1: Typical Single Point Urban Interchange (SPUI) Configuration without Frontage Roads
(Not to Scale)

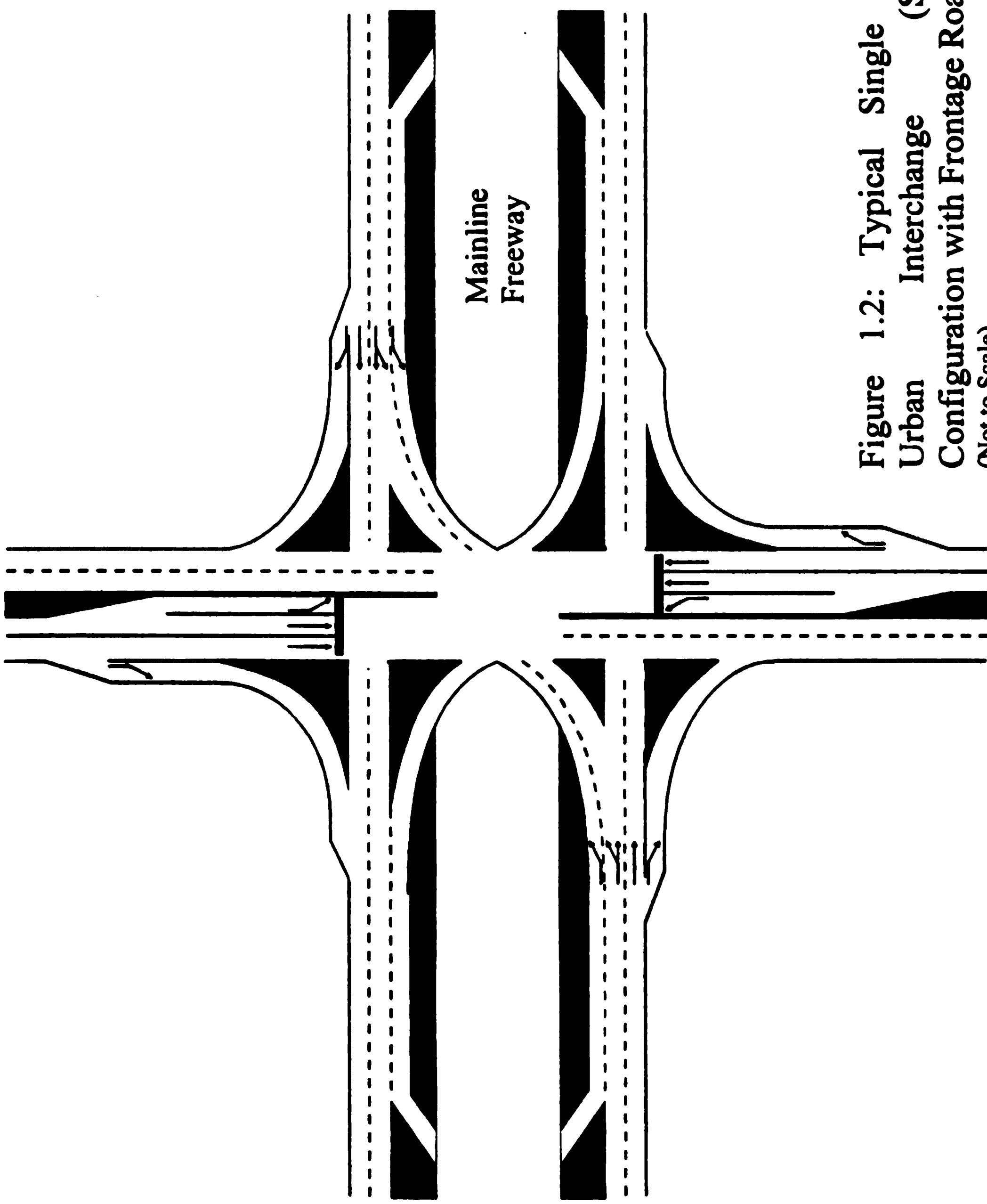


Figure 1.2: Typical Single Point Urban Interchange (SPUI) Configuration with Frontage Roads
(Not to Scale)

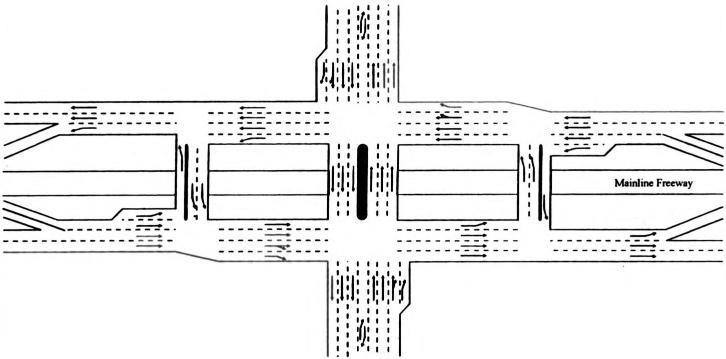


Figure 1.3: Typical Michigan Urban Diamond Interchange (MUDI) Configuration with Frontage Roads
(Not to Scale)

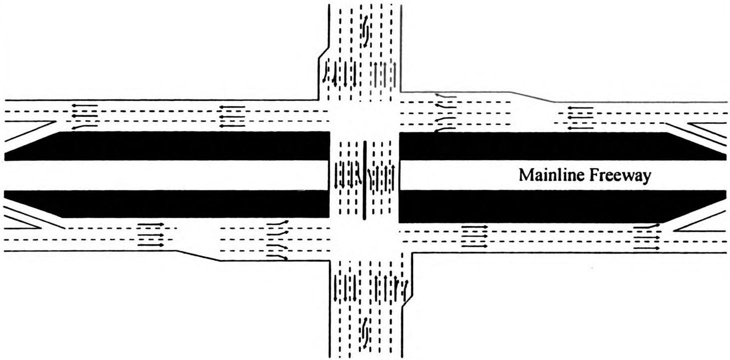


Figure 1.4: Typical Diamond Interchange Configuration with Frontage Roads
(Not to Scale)

perhaps most common interchange used is the urban diamond. Diamond interchanges are used to accommodate traffic from major city streets and for freeways with parallel frontage roads. The frontage roads usually are one-way streets and run in the same direction as the freeway.

The configuration shown in Figure 1.4 is an example of an urban diamond interchange with a city street, freeway and parallel frontage roads. The at-grade intersections of the frontage roads with the crossroad usually have stop-and-go traffic signals. If the freeway is below grade and the crossroad is at grade, then traffic exiting the freeway is going uphill and traffic entering the freeway is going downhill which is beneficial for both movements. This design of the diamond interchange allows traffic entering and exiting the freeway to do so at relatively high speeds. Moreover, if the freeway is depressed, the at-grade intersections have no sight restrictions typically created by freeway structures or differences in grades. Unfortunately, this configuration has relatively low capacity because all of the turning movements occur at the intersections and left-turning vehicles have to yield to oncoming traffic. Thus, there are several areas where traffic spillback may exceed the storage space.

The Michigan Department of Transportation (MDOT), borrowing from its indirect left-turn strategy implemented for most at-grade urban boulevards, modified the traditional urban diamond in an effort to increase the capacity. This modified diamond interchange configuration will be referred to as the Michigan Urban Diamond Interchange (MUDI) (Figure 1.3). This configuration evolved during the design and construction of freeways in the early and mid 1960s.

1.2 Statement of the Problem

There are no SPUIs in Michigan and most of the known SPUIs are located in southern states. Although this interchange design has been around for over 25 years, it has only recently become more prominent due to claims of its efficient operation. However, the benefits of the SPUI have been the subject of some debate. As the popularity of these interchanges increases in other areas of the country, they have been suggested as a logical alternative to the MUDI. Thus, the Michigan Department of Transportation (MDOT) commissioned Michigan State University to study the operational characteristics of the SPUI for application in Michigan. In addition, since both the traditional diamond and MUDI are widely used in urban areas of Michigan, the operational characteristics of these interchange configurations were also of interest.

Since no SPUIs exist in Michigan, operational experience with this interchange configuration was lacking. MDOT raised several concerns regarding the operation of urban interchanges in Michigan. These concerns affecting urban interchange design included: ability to progress the arterial cross-road, compatibility with frontage roads, sensitivity to the level (volume) of left-turning traffic, migration of delay to downstream intersections, need to provide special case signing and pavement markings for positive guidance of drivers, ability to accommodate pedestrians and operational efficiency at volume levels nearing capacity. As a result, a field review was conducted to collect information about the geometric design, signal operation, pedestrian control, and pavement markings of SPUIs.

While some evaluation of the SPUI design has been done in the past, the literature review determined that nothing has been published with regard to the ability to progress the arterial cross-road, compatibility with frontage roads, sensitivity to left-turning traffic, migration of delay, or traffic levels nearing capacity. Additionally, while the operational characteristics of a boulevard intersection have been studied and the results published, the MUDI design, which is unique to Michigan, has never been formally studied and there is no literature on the subject. Thus, the SPUI and MUDI designs were computer modeled to facilitate a comparison of their respective operational characteristics. Furthermore, a traditional diamond interchange was modeled to generate a frame of reference for the results.

Chapter 2

OPERATION AND DESIGN OF THE MICHIGAN URBAN DIAMOND INTERCHANGE (MUDI)

An example of a MUDI is shown in Figure 1.3. This configuration is an urban diamond with left-turning vehicles being routed through separate left-turn structures known as directional cross-overs. Thus, left-turning movements are prohibited at the intersection. As an example, a driver traveling from bottom to top along the arterial wanting to access the left entrance ramp to the freeway would make a direct left-turning maneuver at a standard diamond interchange. For the MUDI, the driver would turn right at the first frontage road, travel to the directional cross over, make a U-turn through the cross over, travel from right to left to the arterial, cross the arterial and access the entrance ramp, thus completing the desired left turn. Similarly, a driver desiring to access a business adjacent to the service road in the opposite direction would use the cross-overs to change direction and gain access. Evident in these maneuvers is the associated increased travel distance.

The distance that the directional cross over structure is placed from the crossroad is a function of the cycle length of the traffic signals and the speed of the movement. Properly designed, if the left-turning maneuver described above began from the start of green, it should receive a green indication at both the cross over and the arterial. Thus, it does not have to stop and the total travel time for this indirect left turn would equal approximately one-half of the cycle length.

In urban areas, access to property abutting the freeway is often of such importance as to require parallel frontage roads. In addition, Intelligent Transportation System (ITS) strategies, such as ramp metering, function better with continuous frontage roads. However, the intersections of the frontage roads with the cross-road usually require the use of traffic signals. These closely spaced traffic signals may have a significant negative impact upon the operation and capacity of the cross-road. This impact may also be influenced by the cross-section (divided multilane vs. non-divided multilane) of the cross-road.

The addition of U-turn lanes to the cross over structures, as shown in Figure 1.3, is cost-effective when there is a major development or other large attractor of traffic located in the top left or bottom right quadrants of the interchange. For example, freeway traffic traveling from left to right destined for a development in the top left quadrant would exit normally at the ramp to the arterial but immediately use the U-turn structure to access the top frontage road and, thus, the abutting property. This traffic never enters the intersection with the arterial and, consequently, this strategy can significantly increase the capacity of the intersection.

Chapter 3

OPERATION AND DESIGN OF THE SINGLE POINT URBAN INTERCHANGE (SPUI)

An example of a SPUI without frontage roads is shown in Figure 1.1. The first SPUI was completed in Clearwater, Florida on February 25, 1974 and was designed by Greiner Engineering. Since that time several other states have adopted the design and have SPUI interchanges in place.

The primary feature of the SPUI is that all through and left-turn maneuvers converge at one signalized intersection area as opposed to two separate, closely spaced signals as with the traditional diamond. In addition, opposing left-turn movements operate to the left of each other, contrary to the right-hand rule. This allows for a relatively simple phasing sequence to be used to control conflicting movements. This phasing sequence typically consists of three phases accommodating: both crossroad through movements, both off-ramp left-turn movements, and both crossroad left-turn movements. The right-turn movements are usually allowed to free-flow. However, if frontage roads are present (Figure 1.2), there is a need to add a fourth phase, resulting in a reduction in capacity of the other phases. In addition, because of the physical size of many of the SPUIs, a relatively long clearance interval is required between the phases.

A limitation in the SPUI design is that the close physical relationship of the bridge abutments, roadway cross-sections, and offset left-turn paths may constrain the ability to easily upgrade the design in the future. In addition, these limitations make it difficult to utilize this design in an area where the crossroad and freeway intersect at a skew. Furthermore, the horizontal alignment of the left-turn paths can affect the amount of right-of-way needed.

Chapter 4

STATE OF THE PRACTICE

To determine the state of the practice with respect to Single Point Urban Interchanges, a literature review, AASHTO e-mail survey and telephone survey were conducted.

4.1 Literature Review

4.1.1 Single Point Urban Interchange (SPUI)

Much of the published literature on the design and operation of single point interchanges was either generated from a study by Bonneson and Messer at the Texas Transportation Institute (TTI) or referenced this effort. The TTI study was conducted over a three year period and generated six papers (3, 4, 5, 6, 7, and 14) on the subject which were published from 1989 to 1992. The objective of that study was to evaluate the appropriate design and control strategy for five diamond interchange configurations. These configurations were: conventional diamond, single point urban diamond, split diamond, three-point diamond, and three-point stacked diamond.

The first paper published by Bonneson and Messer (3) was a one year status report on the study. Of the six SPUIs identified for study, none had continuous, one-way frontage roads. Although the authors stated that it was possible to have a continuous frontage road, they had a concern that with the additional signal phase required to accommodate the frontage road traffic the capacity of the SPUI would be reduced. This concern is echoed in most of the papers produced by this study. Furthermore, the authors stated a concern that the merge capacity of the off-ramp right-turn movement into the arterial cross-road may be a potential problem. Finally, in all cases there were no crosswalks provided for pedestrians to use in crossing the arterial cross-road. The authors stated that the typical SPUI signal phasing does not provide for a protected pedestrian phase to occur across the cross-road.

The authors also identified a wide variation in the geometry and signal strategies at the studied SPUIs. The cross-section of the arterial cross-road varied from 5 to 10 lanes, while the ramp-to-ramp widths varied from 24 to 51 meters (80 to 168 feet). The number of signal heads present in the interchange area varied from 12 to 26. Furthermore, the total of all phase change intervals varied from 15 to 40 seconds, while the total of the all-red intervals varied from 3 to 28 seconds. The average daily traffic (ADT) on the arterial cross-road varied from 28,000 to 52,000.

The second paper by Bonneson (4) presented the results of a sensitivity analysis relating bridge size and clearance time to various geometric features of SPUIs. The results indicated that skew angle, arterial cross-road median width, and ramp-arterial clearance distance have a significant impact on the length of the bridge and the duration of the clearance interval.

The third paper by Bonneson (5) dealt with the headway and lost time at SPUIs. The author collected headway data on 38,000 vehicles at three SPUIs and two at-grade intersections (AGIs). The results of the analysis indicate that through movement headways at SPUIs are larger than those at AGIs. However, the larger radii of the SPUI left-turn paths were found to result in shorter headways for SPUI left-turning vehicles than for AGI left-turning vehicles. Finally, start-up lost time was found to be greater for a SPUI than for an

AGI. However, the author cautioned that these results were based on data from a small number of sites and recommended that further study be done.

The fourth paper by Bonneson (6) dealt with the operational efficiency of a SPUI. The author concludes the there may not be a significant difference in capacity and delay between small and large SPUIs without frontage roads. While the large SPUIs require a longer clearance interval resulting in greater lost time than the small SPUIs, this may be partly offset by lower minimum headways and greater use of the yellow interval associated with the large SPUI's larger turning radii. Furthermore, the author states that although the AGI has higher capacity per signal phase, the SPUI generally operates with a lower average delay. However, the author concedes that the SPUI's greater efficiency with respect to an AGI can be attributed to the elimination of the arterial cross-road through phase via a grade separation.

The fifth paper by Bonneson (7) dealt with change intervals and lost time for SPUIs. The author concludes that driver use of the yellow interval indicates that 2-3 seconds of the yellow interval can be assumed available for every signal phase. The amount of yellow interval use increased with an increase in approach speed. Furthermore, increased volume was found to cause left-turning drivers to use more of the yellow interval.

The final report from the TTI study (14) endorsed the SPUI as a safe and efficient design alternative to a Tight Urban Diamond Interchange (TUDI) in restricted urban conditions. This report was used as a basis to develop the guidelines for geometric design for use with the AASHTO "Green Book" (1). While the original paper by Bonneson and Messer (3) stated that five interchange configurations were to be studied, the final report only compared the SPUI to either an AGI or a TUDI. The capacity analyses performed by the authors show that a SPUI utilizing a 3 phase signal is slightly more efficient than a TUDI, but the advantage diminishes as the size of the SPUI becomes larger. The addition of a fourth phase to accommodate frontage roads resulted in a reduced capacity for the SPUI.

Of the 36 SPUIs studied, the dominant traffic signal control observed was isolated traffic actuated operation. Dual left turns were typically used on both approach legs of the off-ramps and arterial cross-street.

The authors noted several impacts of the large open central control area of the SPUI. These impacts included: a slight hesitancy exhibited by through drivers not promptly moving into the intersection following the onset of the green phase; driver confusion in finding the on-ramp left-turn slots; and, driver confusion resulting when drivers cannot complete the left-turn maneuver and become hung-up in the intersection. Further areas of concern with the SPUI design were noted as: downstream right-turn operations may not have time to weave across a wide arterial into the SPUI's left-turn bay unless the distance to the downstream node is sufficient; there is typically no provision for a protected arterial cross-road pedestrian phase; high-quality illumination is desired when the freeway goes over the interchange area; and, SPUIs cost more than TUDIs to construct.

A separate comparison of the SPUI and TUDI was conducted by Fowler (10). The author used TRANSYT-7F to model both a SPUI and TUDI using twelve different volume scenarios. The author concluded that the relative performance of both interchange configurations are highly dependent on the characteristics of the intersecting movements, such as: directional split, unbalanced turns, volume, and V/C ratio. However, the author states that the SPUI's performance is less susceptible to unbalanced off-ramp turns and directional split and, in smaller configurations, provides greater capacity.

Hawkes and Falini (11) also contended that the SPUI accommodates higher traffic volumes than conventional diamond interchanges. In addition, the authors claimed that the SPUI provides greater safety and requires less right-of-way. However, the authors presented no empirical data to reinforce their conclusions.

Leisch (12) also contends that the SPUI is an effective design--if applied appropriately. The author goes further to state that unfortunately the design has been considered by some as an interchange type to solve all problems, which it does not. The author suggests that the conditions for which the SPUI might be appropriate are where the turning volumes are light to moderate and the right-of-way is restrictive. He also contends that the efficiencies gained by fully utilizing the 3-phase signal are lost if more than one left-turning volume requires dual turning lanes and if the cross-road has more than two through lanes in each direction.

In a further study, Leisch, et. al. (13), state that many articles and analyses presented up to May 1989 appear to be based upon limited data, limited analyses and less than equitable comparisons. The authors tested the concept of two designs, the SPUI and TUDI, by modeling five real world locations using TRANSYT-7F. With the exception of one location, the TUDI was determined to be between 7 and 36 percent more efficient than the SPUI. In all cases where the TUDI was more efficient, the volumes consisted of heavy through traffic and heavy, unbalanced left-turns. The site where the SPUI was more efficient had light cross-road through traffic and all left-turn movements were heavy.

The authors also state several concerns with the SPUI design. First, they state that the SPUI design is impractical with the addition of frontage roads due to the need for a fourth phase. Second, they could find no conclusive observation of safety differences between the SPUI and TUDI. Next, their experience showed that the SPUI generally costs between $1 and $2 million more than a TUDI. Further, the authors state that the maximum additional right-of-way for the TUDI would rarely exceed more than half an acre. Finally, they state that the SPUI has little potential for expansion or modification without reconstruction, while the TUDI can readily be expanded to accommodate more lanes.

Merritt (15) raised additional concerns with the operation of the SPUI. The author stated that drivers who are unfamiliar with the SPUI design may encounter some initial difficulty. Thus, the SPUI design needs to rely heavily on guide signing, pavement markings, and lane use signing for the necessary positive guidance of drivers. The author further stated that the proximity of nearby intersections is a concern.

Abbey and Thurgood (2), based on field observations, consider the SPUI most appropriate when: an AGI becomes congested to the extent it can no longer function and there is too much turning demand to provide just an overpass; an existing TUDI is not performing well because the turning traffic makes it impossible to get optimum functional use from the coordinated traffic signals at the two TUDI intersections; right-of-way is too tight to accommodate a traditional diamond or other type of interchange; and, the skew angle of the intersecting routes is not excessive.

4.1.2 Michigan Urban Diamond Interchange (MUDI)

No literature exists specifically regarding the MUDI. However, the MUDI configuration is similar to a conventional boulevard intersection with respect to geometry, signal operation and pavement markings. In a study by Dorothy, et. al. (8), the operational aspects of the Michigan design for divided highways were analyzed. The results of this study showed that the boulevard arterial design utilizing indirect left-turn strategy and signalized cross-overs was superior to both the five-lane and seven-lane center left-turn lane arterial designs. In addition, the boulevard indirect left-turn strategy was superior to a boulevard strategy which allowed direct left-turns at the major intersection. Thus, accommodating left-turns at a major intersection has a greater negative effect on operational efficiency than the adverse travel distance which results from the use of an indirect left-turn strategy.

4.2 AASHTO E-Mail Survey

A survey was submitted by e-mail to each of the other 49 state departments of transportation. The survey requested information on the design and operation of Single Point Urban Interchanges (SPUI). 14 state DOTs responded: Arkansas, California, Indiana, Iowa, Missouri, New Mexico, New York, North Dakota, Oklahoma, Pennsylvania, Texas, Vermont, West Virginia and Wyoming. Of these, only California, Indiana, Missouri, and New Mexico have operating SPUIs. In addition, New York is presently designing their first SPUI. None of the responding states with existing SPUIs reported having frontage roads as part of the design.

Generally, the respondents reported that the major advantages of a SPUI configuration with respect to other geometric configurations are: that it requires the same or less Right-of-Way, has less delay and user costs, is adaptable to frontage roads, requires fewer signals, is easier to coordinate the traffic signals with the surrounding system, costs less, has fewer conflict points, allows for U-turn movements, and, has superior aesthetics. The responding states also stated that the major disadvantages of a SPUI configuration with respect to other interchange designs are: it is not an optimal solution if adequate Right-of-Way is available, it costs more, it has long or special bridge structures, signals are difficult to mount, it has long clearance intervals, it has unbalanced traffic flows from the off ramps, it is tough on pedestrians, it should not be considered where the Right-of-Way allows for the construction of a Partial-Cloverleaf interchange, it has less capacity than a Partial-Cloverleaf, the downstream intersections may control the flow, left-turn storage capacity on the cross-road is critical, and, sight distance may be limited.

The responses received from different states varied widely. With respect to delay, one state reported that delay decreased and another reported no noticeable increase in delay. Accident rates were reported to be similar to diamond interchanges or having no noticeable increase in accidents. One state reported that signing was more difficult and two other states reported that they used conventional signing. One state reported that they used conventional pavement markings, another state reported that pavement markings may be a problem, and a third state reported that there is a need for extensive pavement markings. A SPUI was reported to cost $2 to 4 million more than a conventional diamond, $8 to 12 million for converting an existing diamond, and, the same as a conventional diamond. Finally, the Right-of-Way requirements were reported to be similar to a tight diamond, to depend upon the use of retaining walls, and, to be less than a conventional diamond.

The limited number of responses to the survey restricted its usefulness for comparison to the conditions found in Michigan. While maintenance of a SPUI was not a problem for one state and was "little" problem for another state, snow plowing was not considered, as none of the responding states with SPUIs are in a climate where snow plowing would be anticipated to be a problem. In addition, Michigan tries to progress traffic on most of its major arterials. However, only one state responded that they had a cross-road with good progression, while the other states responding did not make an effort to progress the cross-road traffic.

4.3 Telephone Survey

The review of the literature and the response to the e-mail survey, while helpful, had significant inconsistencies and lacked information in key areas. A telephone survey was subsequently conducted with some of the e-mail states and with several additional states' Departments of Transportation. The states called in the telephone survey were: Indiana, Illinois, Minnesota, Florida, Arizona, Missouri, and, Texas. The objective of the phone survey, in addition to collecting more information, was to locate the most appropriate sites for a field review. Specifically, it was desired to observe the operation of SPUIs with frontage roads, the progression of the cross-road traffic, and, the operation of SPUIs under winter-time conditions.

The individuals having the greatest knowledge of the operations of the SPUIs were sought out. Thus, most of the phone conversations were with the district traffic engineers. Of the seven state DOTs telephoned, four gave strong favorable recommendations on the positive aspects of a SPUI. One state DOT could not recall its operation and had ambivalent feelings. The remaining two state DOTs had unfavorable opinions.

One engineer responded that the SPUI was his preferred design. Conversely, another engineer responded that the SPUI did not have a single advantage with respect to the design and operation of a conventional tight diamond. Also on a negative note, another state traffic engineer responded that when their first SPUI was open to traffic there was a great deal of driver confusion which negatively impacted the operation of the interchange.

When attempting to narrow the search for appropriate field review sites, it was discovered that only two of the states had any experience operating a SPUI with frontage roads. Additionally, only two engineers reported that they progressed the traffic on the cross-road arterial. Most of the states reported that they rely solely on traffic actuated signalization along the arterial.

The comments of the Minnesota DOT were of special interest since they have a similar climate. The district traffic engineer in Duluth believed that a SPUI was easier to operate than a conventional diamond interchange. In addition, he reported that pedestrians did not have a problem and he knew of no winter time difficulties.

4.4 Conclusions from the State of the Practice

The literature review, e-mail survey and telephone survey pointed out several aspects of SPUI design that would need to be addressed in the field review and the simulation modeling. These aspects can best be presented by grouping them into several topic areas: geometric design, signal operation, pedestrian control, pavement markings, and simulation modeling.

Several inconsistencies in the geometric design of SPUIs were discovered. The studies by Bonneson and Messer (3) and Leisch, et. al. (13) raised the concern that the operation of a SPUI may be adversely affected by the addition of continuous frontage roads due to the need for a fourth signal phase. However, the responses from the e-mail survey listed the adaptability to frontage roads as one of the major advantages of the SPUI design. The study by Messer and Bonneson (14) stated that dual left-turns were typically used on both approach legs of the off-ramps and arterial cross-street. However, Leisch (12) contends that the efficiencies gained by fully utilizing the 3-phase signal are lost if more than one left-turning volume requires dual turning lanes.

The signal operation of SPUIs varied by location. Messer and Bonneson (14) studied the operation of 36 SPUIs and observed the dominant traffic signal control to be isolated traffic actuated operation. This was reinforced by the results of the telephone survey in which most of the states reported that they rely solely on traffic actuated signalization along the arterial. However, most arterials in Michigan are operated in a progressed-coordinated system. Only one state from the e-mail survey and two states from the telephone survey stated that their agencies progressed the traffic on the arterial.

The reported ability to accommodate pedestrians varied. Bonneson and Messer (3) reported that the typical SPUI signal phasing does not provide for a protected pedestrian phase to occur across the cross-road. However, the district engineer for Duluth reported that pedestrians did not have a problem.

The reported need for pavement markings also varied. As part of the e-mail survey, one state reported that they used conventional pavement markings, another state reported that pavement markings may be a problem, and a third state reported that there is a need for extensive pavement markings. Merritt (15) stated that the SPUI design needs to rely heavily on guide signing, pavement markings, and lane use signing for the necessary positive guidance of drivers.

The studies by Fowler (10) and Leisch, et. al. (13) used computer modeling to compare the operation of a SPUI and a TUDI. However, both studies used TRANSYT-7F which is a macroscopic model and is best suited to modeling large networks, not individual intersections. In addition, the study by Fowler (10) only modeled 24 scenarios and the study by Leisch, et. al. (13) only modeled ten scenarios.

Chapter 5

FIELD REVIEW OF THE SPUI

While the MUDI configuration can be compared to a boulevard intersection, the SPUI configuration has no direct comparison. Based on information gathered through the e-mail and telephone surveys, sites were selected in several states for inclusion in the field review. These sites were located in Indiana, Illinois, Minnesota, Florida, Missouri and Arizona.

During a typical field review, the engineers and technicians responsible for the operation of the SPUI interchange being studied were interviewed. These interviews included a visit to the site where the operation of the SPUI was discussed. If possible, plan view drawings, signing plans, aerial photographs, signal timings, traffic volumes, in-house studies, and, economic data pertaining to the SPUI in question were collected. In the field, extensive photographs and video of the interchange were taken.

Based on the field review conducted between January 1996 and May 1996, subjective observations can be made about the design and operation of a SPUI. These observations are based upon the consensus of the team which conducted the field review. The members were, in addition to the author, Dr. Thomas Maleck (Michigan State University), Laura Aylsworth-Bonzelet (Michigan Department of Transportation), and John Saller (Michigan Department of Transportation). These observations can best be presented by grouping them into several topic areas: geometric design, signal operation, pedestrian control, and pavement markings.

5.1 Geometric Design

The geometric features of the SPUIs varied greatly from state to state. The difference in designs was much greater than anticipated and this difference may explain some of the inconsistencies in the responses to the e-mail and phone surveys.

The most significant difference in design is between a SPUI with the cross-road going over the freeway and a SPUI with the freeway going over the cross-road. The SPUIs with the cross-road going over the freeway (Figure 5.1) were found to look and operate more like a conventional signalized intersection. Because of this, driver confusion is reduced. Conversely, significant driver confusion was observed at interchanges utilizing the cross-road under the freeway design. At times, vehicles became trapped in the intersection due to driver confusion, creating a dangerous situation (Figure 5.2). In addition, routing the freeway over the cross-road exposes the freeway and major traffic volume to preferential icing in cold weather climates.

Another significant difference in design is related to the physical size of the interchange. Some of the newer SPUI designs include the provision of a dedicated lane to permit a U-turn maneuver from the exit ramp back onto the entrance ramp (Figure 5.3). These dedicated structures were located under the tailspans requiring the tailspans to be much longer than normal. While the smaller designs can provide for most U-turns, this dedicated lane is necessary to accommodate large trucks and to increase the speed of the maneuver. Even at interchanges where this maneuver was prohibited, it was still observed to occur regularly. The smaller designs were observed to function with reduced clearance times. In addition, the Right-of-Way requirements are obviously much less with the smaller design.

Figure 5.1: SPUI with cross-road going over the freeway with all signal heads located on a single overhead tubular beam.

Figure 5.2: Confused Driver (car with lights on) stopped in middle of a SPUI while traffic proceeds on either side.

Figure 5.3: U-turn lane accommodates large trucks.

The design of the structures varied from state to state. They are generally much longer than those of conventional diamond interchanges. Some of the spans measured were found to be greater than 146 meters (480 feet) in length. Often there are three spans of nearly equal lengths. Some of the structures were noisy and the resulting booms could be heard for several kilometers. This noise was reported to be the source of extensive residential complaints. Because of the large widths and lengths, the road under the structures was dark. Lighting was often provided under the structures during the day and visibility at locations that utilized light color bridge paints (e.g. sand or concrete) were noticeably better than those with dark color bridge paints. These characteristics were not evident when the cross-road went over the freeway.

The impact of continuous frontage roads on the overall operation of a SPUI was a key area of interest. It was explicitly desired to observe the operation of a SPUI with parallel frontage roads whose intersections with the cross-road are signalized and accommodate significant through traffic. Two of the states visited were anticipated to have these type of frontage roads based on responses from the e-mail and telephone surveys. However, these frontage roads did not satisfy MDOT's requirements. One of the state's frontage roads are what would be considered to be ramps with private driveways. The other state had a frontage road that was a two-way road which did not appear to generate the desired through traffic. Several of the district traffic engineers expressed strong opinions that providing for continuous frontage roads with a SPUI is a poor design and negates the advantages of a SPUI.

The geometry of the exit ramps often flared from one lane to three at the ramp terminus. Of these three lanes, two were for left-turning traffic and one for right-turning traffic. The right- and left-turning lanes are separated by a large channelized island. The dual left-turning traffic on the off-ramp backs up during peak periods. This blocks right-turning traffic from exiting and locks up the ramp (Figure 5.4).

The geometry of the on-ramps normally consisted of three lanes near the interchange, which reduce down to one lane before entering the freeway. The initial three lanes of the on-ramp are fed by two left-turn lanes, under signal control, and a free-flow right-turn lane. Several engineers commented that this merge results in a sideswipe crash problem. However, the crash reporting systems are structured in such a way that these sideswipe crashes are not referenced to the interchange. Thus, it is difficult to get a clear picture of the crash experience of the interchange.

Figure 5.4: Dual left-turning traffic is backing up, blocking right-turning traffic.

Most of the SPUI designs, regardless of state, added several additional lanes to the cross-road basic laneage at the interchange. A typical design would have a 6 lane cross-road being widened to nine lanes at the interchange. The additional lanes are typically a right-turn bay and provision for dual left-turn lanes for the on-ramp. In addition to the auxiliary lanes, most of the cross-roads had raised, concrete medians ranging in width from 1.2 meters (4 feet) to 3.6 meters (12 feet).

5.2 Signal Operation

The operation and placement of traffic signals were of special interest. The MUDI configuration operates similar to a conventional boulevard intersection. However, each state's practice differed significantly for the SPUI. The cycle lengths varied from 80 seconds to 180 seconds. The SPUIs reviewed that had longer cycle lengths and usually had fully actuated signal phases for all movements.

Of special interest was the ability to progress traffic on the cross-road. Two of the SPUIs reviewed have a cross-road arterial which was part of a pre-timed progressed strategy. While the interchange was operating well below capacity, it was obvious that providing progression would not be a problem. These interchanges were the smaller designs which result in shorter clearance times and allows for a shorter signal cycle. However, the impact of the SPUI on intersections downstream must be considered. Comments were made to the effect that the SPUI dumps traffic on the downstream nodes causing a migration of delay. This was hard to judge in the field as none of the SPUIs reviewed were operating near their capacities.

Most of the SPUIs reviewed had a 3 phase signal operation. The 3 phases were usually: left-turn entrance ramp movements, left-turn exit ramp movements, and, cross-road through movements. One state provided for a right-turn exit ramp green arrow during the left-turn entrance ramp phase. Usually the exiting right turn was accommodated via a free-flow, channelized merge with the cross-road traffic. However, a skewed intersection affects the operation of the signal phasing. At these locations, there are 4 signal phases: first exit ramp movement, opposing exit ramp movement, left-turn entrance ramp movements, and, cross-road through movements. In addition, the skew requires longer clearance times.

The placement of the traffic signal heads also varied greatly from state to state and by geometric design. In the case of a SPUI where the cross-road goes over the freeway, all of the signal heads are located on a single overhead tubular beam (Figure 5.1). Thus, the 3 phase operation was analogous to a traditional at-grade intersection with a 3-phase signal. This design typically took less Right-Of-Way. This SPUI design was observed to function very well, although the traffic volumes were not heavy. In the case of a SPUI where the freeway goes over the cross-road, the signal heads are mounted on the structure. However, some states have post-mounted signals located on traffic islands. In one interchange alone there were 24 signal heads. With this proliferation of signal heads, it was possible to see green, amber and red indicators at the same time depending on where one looked. In addition, the signal heads when post-mounted were vulnerable to damage from motorists running into them.

The physical size of the interchange also affected the signal operation. If the intersection area is very large, longer clearance times are required for traffic to clear the intersection prior to allowing the next phase. Additionally, the green signal arrow for left-turning traffic was often canted to give the motorist a sense of direction in these large intersection areas. Still, there was driver confusion resulting from the large distances needed to clear the intersection (Figure 5.2). There were three common mistakes observed. The first results when the lead car does not start on green because the driver is (presumably) confused on which signal indication is theirs. The second results when a motorist entering the intersection from the exit ramp on a green light has to drive through a red indication meant for the cross-road. Vehicles were observed stopping in the middle of the interchange and waiting for a green indication. The third results when a motorist starts into the intersection and becomes (presumably) confused about which path to follow due to the large size of the interchange or overlapping pavement markings.

5.3 Pedestrian Control

The ability to accommodate pedestrian movements varied greatly from site to site. Since the MUDI configuration is geometrically similar to a boulevard intersection, it is able to accommodate pedestrians without problems.

When the SPUI was examined, many of the locations simply had no pedestrian movements to accommodate. Where pedestrians were present, it was not difficult for them to move parallel to the cross-road and cross the ramp movements. However, with all movements going through the center of the interchange and a signal operation utilizing fully traffic actuated phases, there is always traffic moving through the intersection. This makes it difficult for pedestrians to cross the cross-road. In addition, the width of the cross-road, often 6 to 8 lanes, makes it difficult for pedestrians to cross the cross-road. Often, pedestrians would become trapped on the concrete channelization of the cross-road when attempting to cross. Some sites actually prohibited pedestrians from crossing. However, this prohibition was often violated, as the only other opportunity to cross was at the next intersection which was typically over 400 meters (quarter of a mile) away.

5.4 Pavement Markings

With the potential for snow as in Michigan, the need to rely heavily on traffic lane markings is a concern. The MUDI has no lane markings that are unique when compared to a standard intersection. However, for the most part the larger SPUIs have supplemental lane markings to assist the motorist with the left-turn movement. The need for these pavement markings is paramount when the SPUI is large. These pavement markings may overlap creating driver confusion (Figure 5.5). In a skewed configuration, this overlap is more pronounced and it can be confusing even to a driver familiar with the interchange. However, the need for supplemental lane lines for the turning movement was not evident for the locations where the cross-road went over the freeway or the interchange was small in size.

One location had lights placed in the pavement to help illuminate the turning path. When left turning traffic was given a green light, these "runway" lights would light up green along the path to be taken by the motorist (Figure 5.6). These lights were reported to be a maintenance problem as they may fill with dirt, which obscures the lens. The engineer responsible for maintaining the operation of this location expressed a concern that the lights may also raise several tort liability issues. For example, if the runway lights are not working at the time of an accident, it may be construed that one of the traffic control devices (TCDs) was not working.

Many of the SPUIs reviewed have channelized islands to help guide drivers as they negotiate though the single-point intersection. On the center island, typically there was also directional signing present. The location of this signing may make it vulnerable to damage from motorists who stray onto the island. During the field review, it became obvious that

Figure 5.5: Pavement marking overlap creates driver confusion.

Figure 5.6: "Runway" lighting to help illuminate the turning path.
Note the buildup of debris.

motorists frequently strike these islands. Channelized islands are not as popular in Michigan because of their interference with snow plowing operations.

5.5 Conclusions from the Field Review

Based on this field review, subjective observations can be made about the design and operation of the SPUI. These observations were grouped into the areas of geometric design, signal operation, pedestrian control, and pavement markings.

The most significant geometric design difference of the SPUIs reviewed is between a SPUI with the cross-road going over the freeway and a SPUI with the freeway going over the cross-road. The SPUI with the cross-road going over the freeway was found to look and operate more like a conventional signalized intersection. Another design difference was related to the physical size of the interchange. SPUIs without dedicated U-turn lanes appeared to accommodate U-turns as well as those with dedicated U-turn lanes. The smaller designs were observed to function better than the larger designs. In addition, the Right-of-Way requirements are less with the smaller designs. In some cases, the structures were noisy resulting in complaints from nearby residents. Because of the large size of these structures required when the freeway goes over, the roadway under the structure is dark. These undesirable structure characteristics are not present when the cross-road goes over the freeway.

Furthermore, in the case where the freeway goes over the cross-road, sight distance is a concern. Several engineers expressed strong opinions that the use of continuous frontage roads with a SPUI negates the advantages of the design. Finally, the geometry of the typical on-ramps may result in a sideswipe crash problem.

The signal operation strategy employed by each state differed significantly. Cycle lengths varied from 80 seconds to 180 seconds, with longer cycle lengths usually having fully actuated signal phases for all movements. The interchanges reviewed were operating below capacity and, at this level, progression of the cross-road was not a problem. If the interchange area was very large, the clearance times became quite long and there was significant driver confusion. Finally, the best placement of traffic signal heads occurred in designs where the cross-road went over the freeway, allowing the signal heads to be located on a single overhead tubular beam.

The ability to accommodate pedestrians varied greatly between designs. Typically, it was not difficult for pedestrians to move parallel to the cross-road and cross the ramp movements. However, due to the characteristics of the SPUI, there is always traffic moving through the intersection. This makes it difficult for pedestrians to cross the cross-road.

The need for pavement marking in large SPUIs is paramount. However, these pavement markings can overlap and cause driver confusion. This resultant driver confusion is most pronounced when the cross-road is skewed.

Based on the field review, the conclusion of the study team is that the Single Point Urban Interchange (SPUI), properly situated, is an effective design. However, the large size of some designs may be counterproductive due to the need for an increase in clearance times.

Chapter 6

METHODOLOGY

Sufficient traffic volumes were not present at any of the locations visited during the field review to allow for a field determination of operation at capacity. Thus, to compare the relative operational characteristics of the interchange configurations in question, computer modeling of each geometric configuration was used.

6.1 Selection of the Computer Model

The concept of traffic control is giving way to the broader philosophy of Transportation Systems Management (TSM), in which the purpose is not to move vehicles, but to optimize utilization of transportation resources in order to improve the movement of people and goods without impairing other community values (1). To better understand this optimization, computer simulation techniques have been developed. These models describe a system's or network's operational performance based on various data inputs. This minimizes the need for an existing facility to be expanded or a proposed facility to be constructed to evaluate their performance.

The computer simulation approach is considered more practical for evaluation of network changes or operation than field experiments for the following reasons:

- It is less costly
- Results are obtained relatively quickly

- The data generated by simulation includes many measures of effectiveness that cannot easily be obtained from field studies
- Disruption of traffic operations, which often accompany a field experiment, is avoided (1).

TRAF-NETSIM is a stochastic, microscopic model which describes the operational performance of a road system based on several measures of effectiveness (MOEs). The internal logic of this model describes the movements of individual vehicles responding to external stimuli including traffic control devices, the performance of other vehicles, pedestrian activity, and driver performance characteristics. NETSIM applies interval-based simulation to describe traffic operations. Each vehicle is a distinct object which is moved every second, and that every variable control device (traffic signals) and event are updated every second. Each time a vehicle is moved, its position (both lateral and longitudinal) on the links and its relationship to other vehicles nearby are recalculated. Its speed and acceleration are also recalculated. Vehicles are moved according to car following logic, response to traffic control devices and response to other demands (1). For these reasons, the TRAF-NETSIM model was selected for use in this study.

6.2 Network Configuration

To compare the operation of a diamond interchange (Figures 6.1 and 6.2), a MUDI (Figures 6.3 and 6.4), and a SPUI (Figures 6.5 and 6.6), several decisions were made about the network geometry. First, it was decided to model the arterial crossroad as both a five-lane and seven-lane pavement. The cross-section of the five-lane facility consists of four through lanes

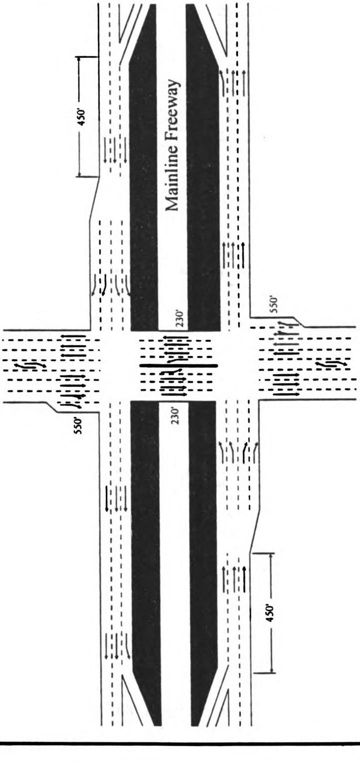


Figure 6.1: Typical Diamond Interchange Configuration with Frontage Roads
(Not to Scale)

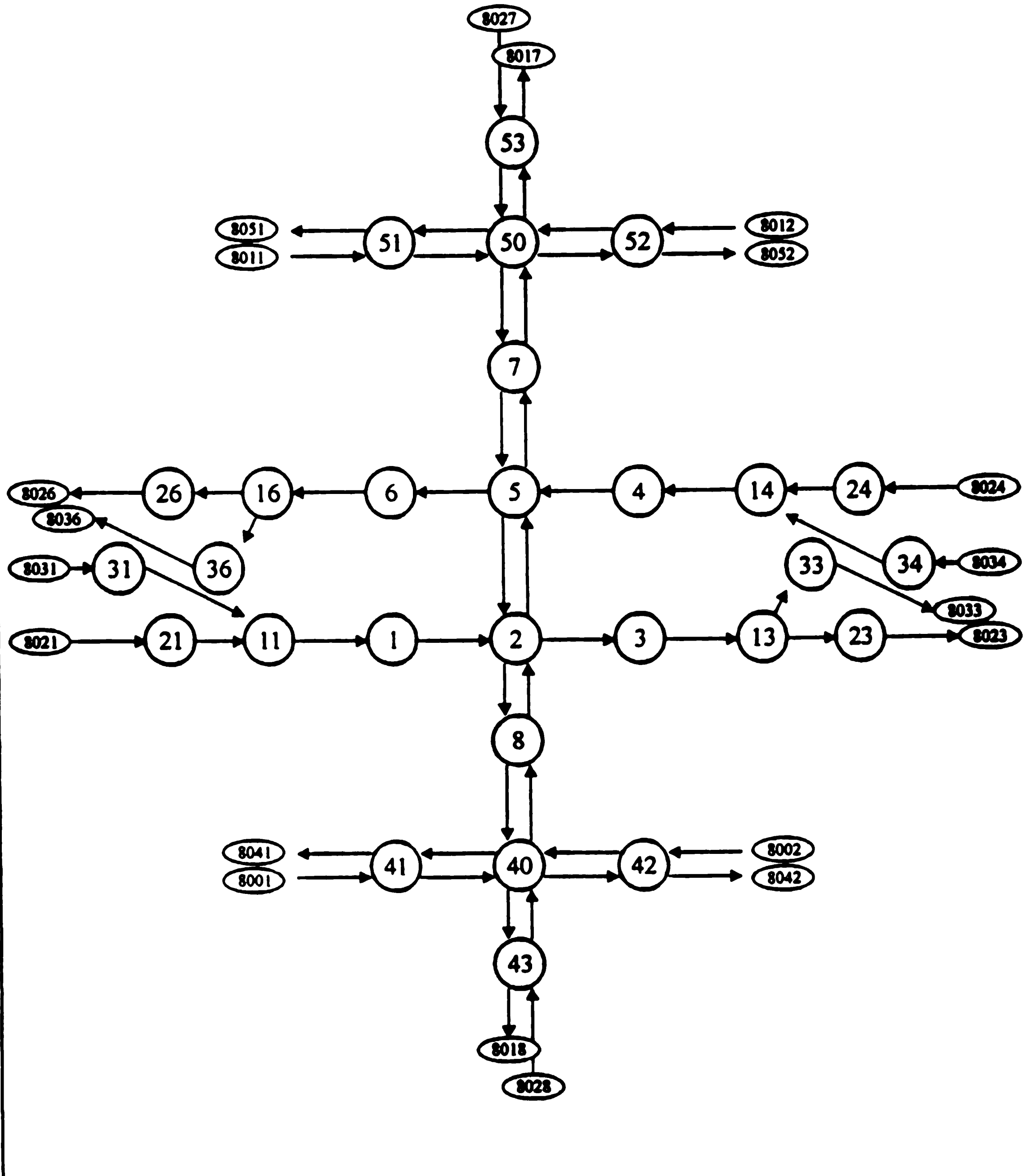


Figure 6.2: Link/Node Diagram for Diamond Configuration
(Not to Scale)

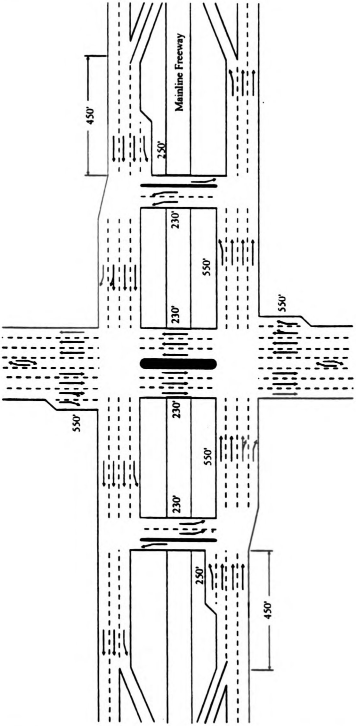


Figure 6.3: Typical Michigan Urban Diamond Interchange (MUDI) Configuration with Frontage Roads
(Not to Scale)

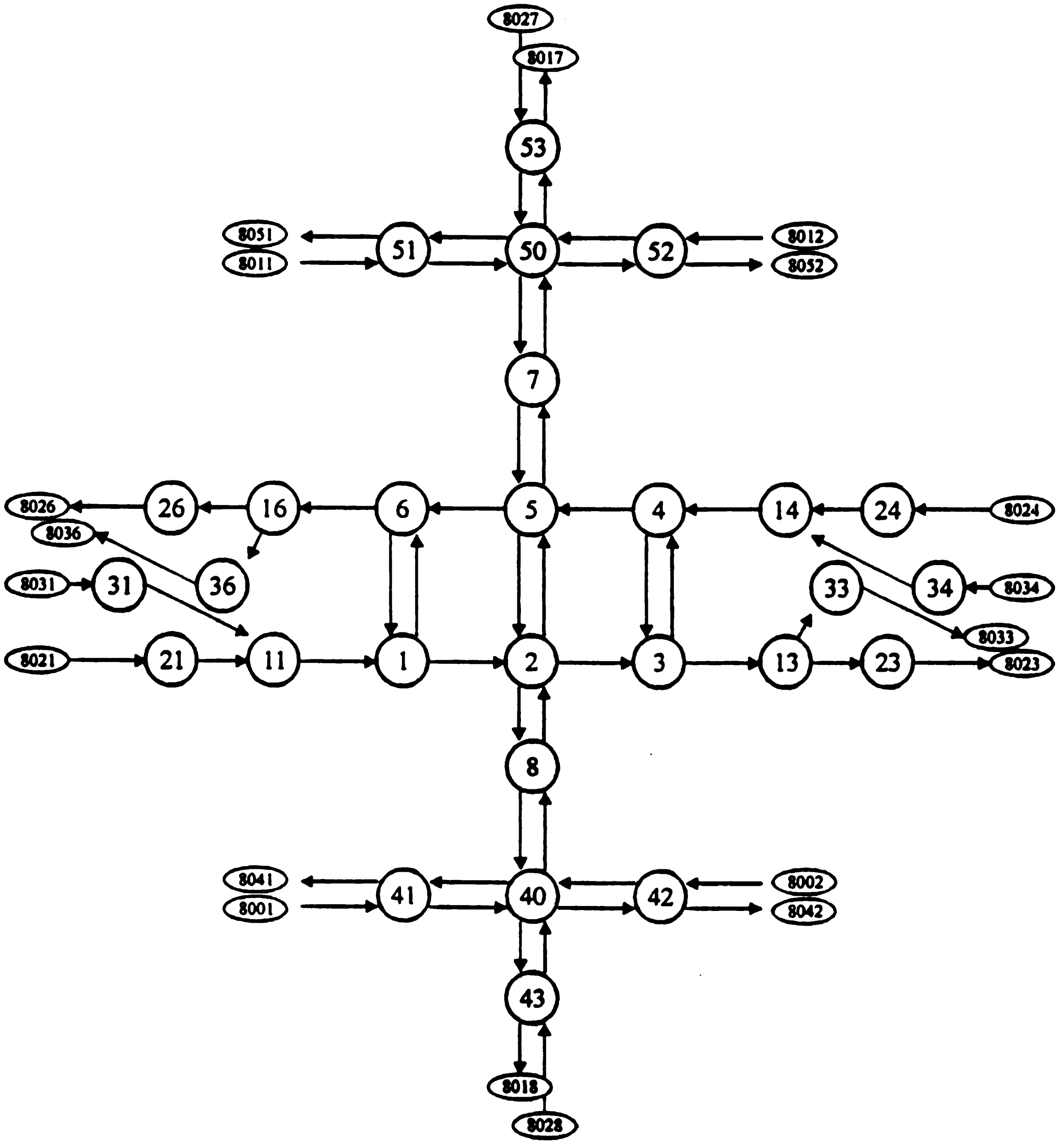


Figure 6.4: Link/Node Diagram for MUDI Configuration
(Not to Scale)

MUDI

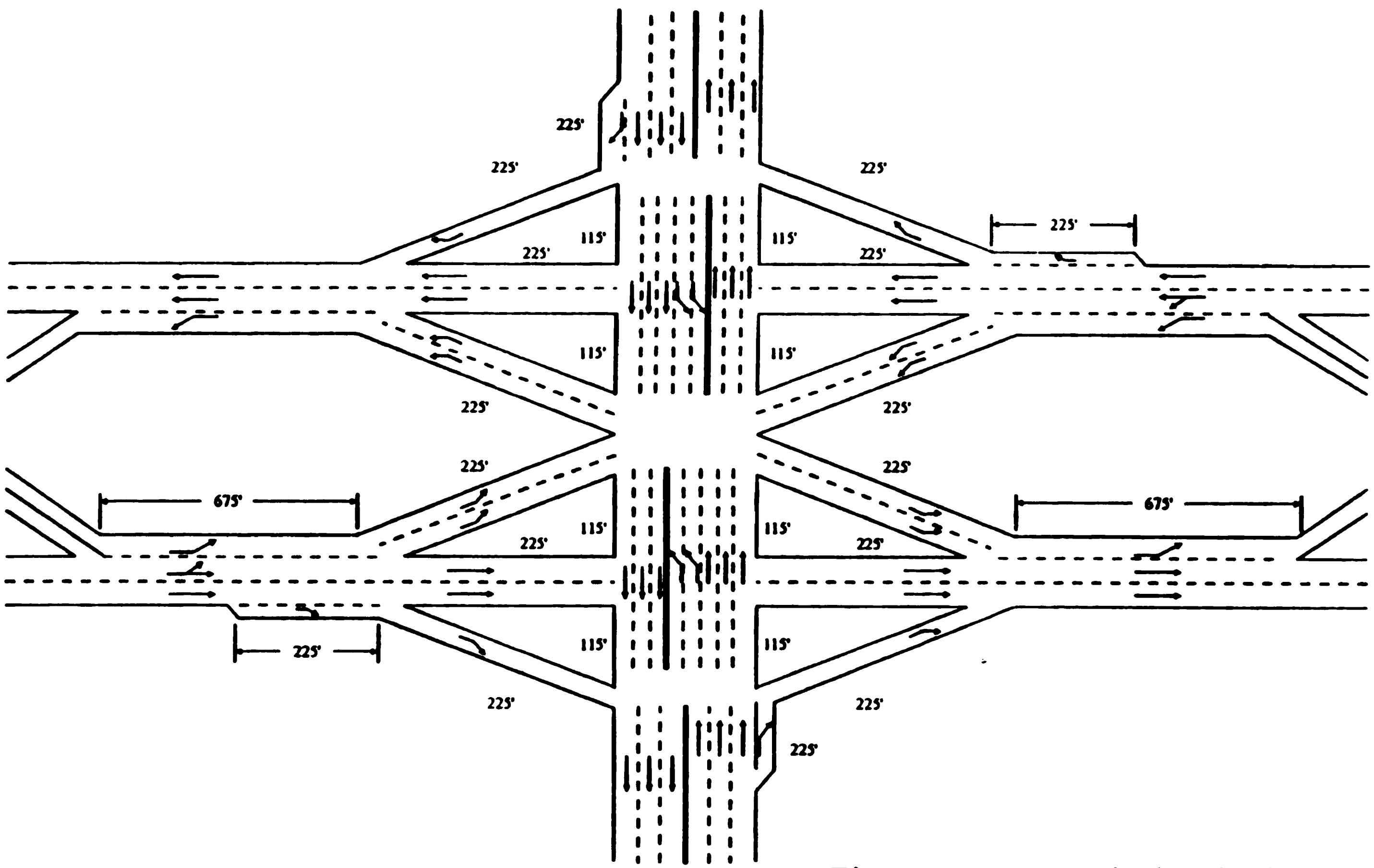


Figure 6.5: Typical Single Point Urban Interchange (SPUI) Configuration with Frontage Roads
(Not to Scale)

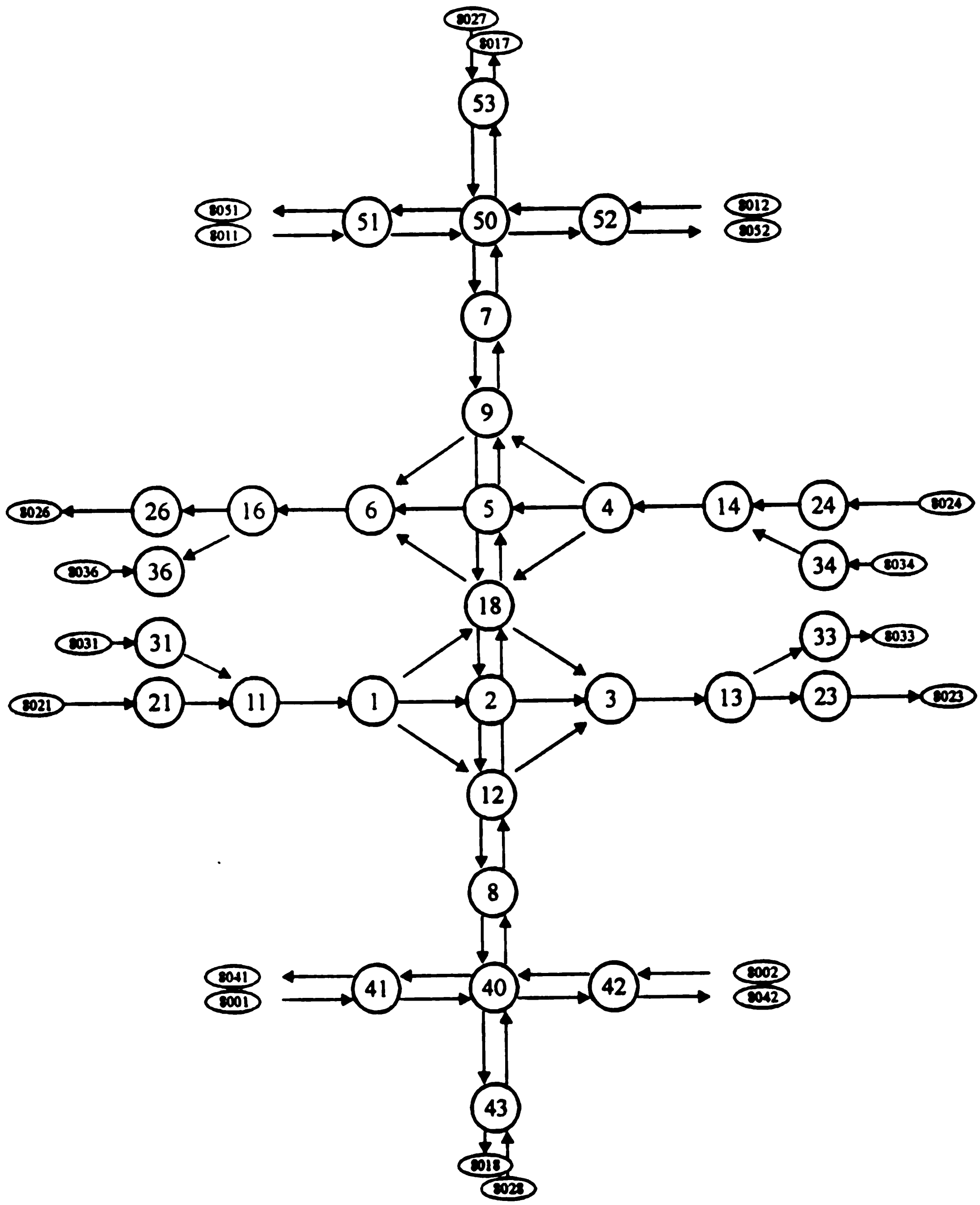


Figure 6.6: Link/Node Diagram for SPUI Configuration
(Not to Scale)

SPUI

(two in each direction) and a continuous center left-turn lane (CCLTL), while the seven-lane facility consists of six through lanes (three in each direction) and a CCLTL.

Next, the size of the network had to be determined. Since a major concern with regard to interchange operation is the interchange's effect on the downstream nodes of the arterial, it was decided to model both the interchange area and one arterial downstream node on either side of the interchange. These downstream nodes were modeled as the intersection of the arterial with a five-lane arterial with a CCLTL. Since an arterial is said to have "perfect geometry" if the intersections are 0.8 kilometers (one-half mile) or 1.6 kilometers (one mile) apart, these downstream intersections were initially placed at 1.6 kilometers from the interchange. The perfect geometric spacing of these intersections allows for optimal signal progression, thus minimizing delay. The impact of minor crossroads and driveways was not modeled.

Once the spacing of these downstream intersections had been determined, their geometry had to be defined. For each approach to the downstream intersections, a 168 meter (550 foot) left and right turning bay was provided. In the interchange area, a 168 meter (550 foot) right turn bay was provided on the arterial approach for both the MUDI and diamond interchange. Additionally, a 168 meter (550 foot) right turn bay was provided on the frontage road for traffic wishing to make a right turn from the frontage road to the arterial for both configurations. In the SPUI interchange area, the length of the right turn bays was shortened to 69 meters (225 feet), as the right turn was operating in a free-flow condition.

6.3 Signal Operation

For the purposes of the computer model, a free flow speed of 72 kph (45 mph), or 20 meters per second (66 feet per second), was assumed for the arterial, minor crossroads and frontage roads. Based on this free flow speed and an intersection separation of 1.6 kilometers (one mile), the optimal cycle lengths were determined to be a multiple of 40 seconds. Longer cycle lengths will accommodate more vehicles per hour due to the lower frequency of starting delays and clearance intervals. Thus, an 80 second cycle was selected for the downstream nodes for all cases. An 80 second cycle was also selected for the operation of the MUDI. However, since the modeled arterial was to be operated in a progressed-coordinated system, a 160 second cycle (double cycle) was selected for the interchange signals in both the SPUI and the diamond interchange due to the need for long phase changes and clearance intervals. Further, given the freeflow speed of 72 kph (45 mph), the minimum phase change interval (yellow and overlapping red) for each phase was determined to be 5 seconds. This phase change interval ensures that approaching vehicles can either stop or clear the intersection without conflicts.

The modeled arterial was to be operated in a progressed-coordinated system, so a definite time relationship exists between the start of green intervals at adjacent intersection signals. Thus, signal offsets had to be determined. Since both downstream intersections were placed with perfect geometric spacing from the interchange, the free flow speed was assumed to be 72 kph (45 mph), and a cycle length of either 80 or 160 seconds was used, an offset of 0 seconds was selected to best provide for progression of traffic along the arterial. When the spacing of the closest downstream intersection was changed to 0.8 kilometers (one-half mile), this offset was changed to one half a cycle or 40 seconds. Furthermore, when the spacing of the closest downstream intersection was changed to 1.2 kilometers (three-fourths mile), this offset was changed to 20 seconds for the closest node and 60 seconds for the node placed at 2.0 kilometers (one and one-quarter mile).

The number of phases used depends upon the geometry of the intersection (number of approaches, lanes) and the volumes and directional movements of traffic. The purpose of phasing is to minimize the potential conflicts at an intersection by separating conflicting traffic movements. However, as the number of phases increases, the total delay is increased and the total carrying capacity of the intersection may be reduced. Thus, it is desirable to use the minimum number of phases that will accommodate the traffic demands.

The signal phasing diagram for the intersection of the minor five-lane CCLTL and the arterial was the same for both downstream nodes. It was assumed that the volume ratio between the arterial and the minor crossroads would be 70/30. Thus, the green split between the arterial and crossroad would also be 70/30.

The phasing diagram for the MUDI signals was determined (Figures 6.7 and 6.8) using a green split of 60/40. In addition, an offset had to be determined for the crossover signals of the MUDI design. At the free flow speed of 72 kph (45 mph), or 20 mps (66 fps), a vehicle requires 8.3 seconds to traverse the 168 meters (550 feet) from the intersection to the crossover. The desired offset for the crossover signal is one which reduces the delay to arterial traffic wishing to make an indirect left turn while not adversely affecting the progression of the arterial. If a vehicle left the stop bar of the crossroad intersection at the free-flow speed and there were no cars at the crossover signal, this offset would be 8.3 seconds. However, there is typically a queue of vehicles, mostly comprised of exiting freeway traffic wishing to make an indirect left turn onto the

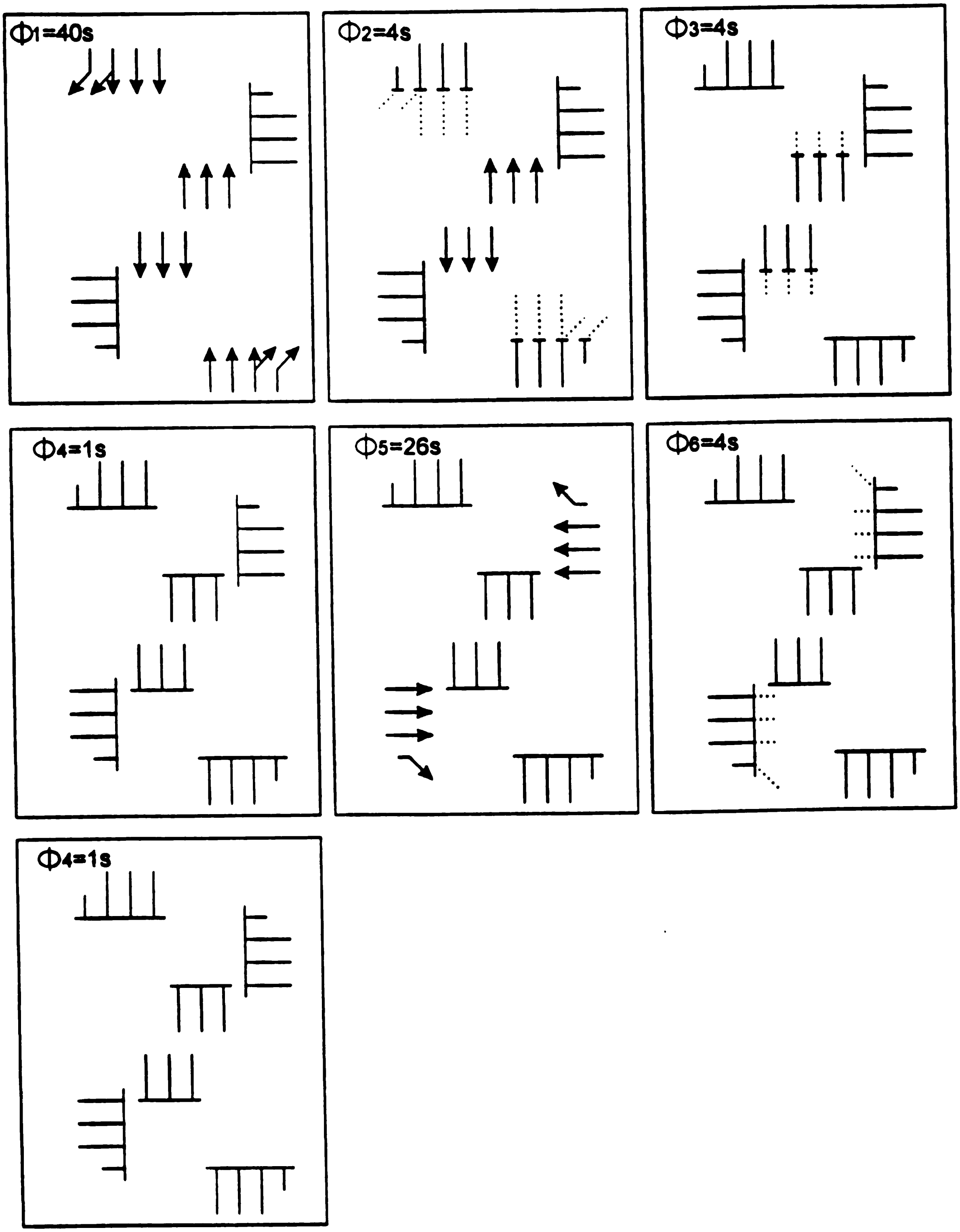


Figure 6.7: Phasing Diagram for MUDI Configuration

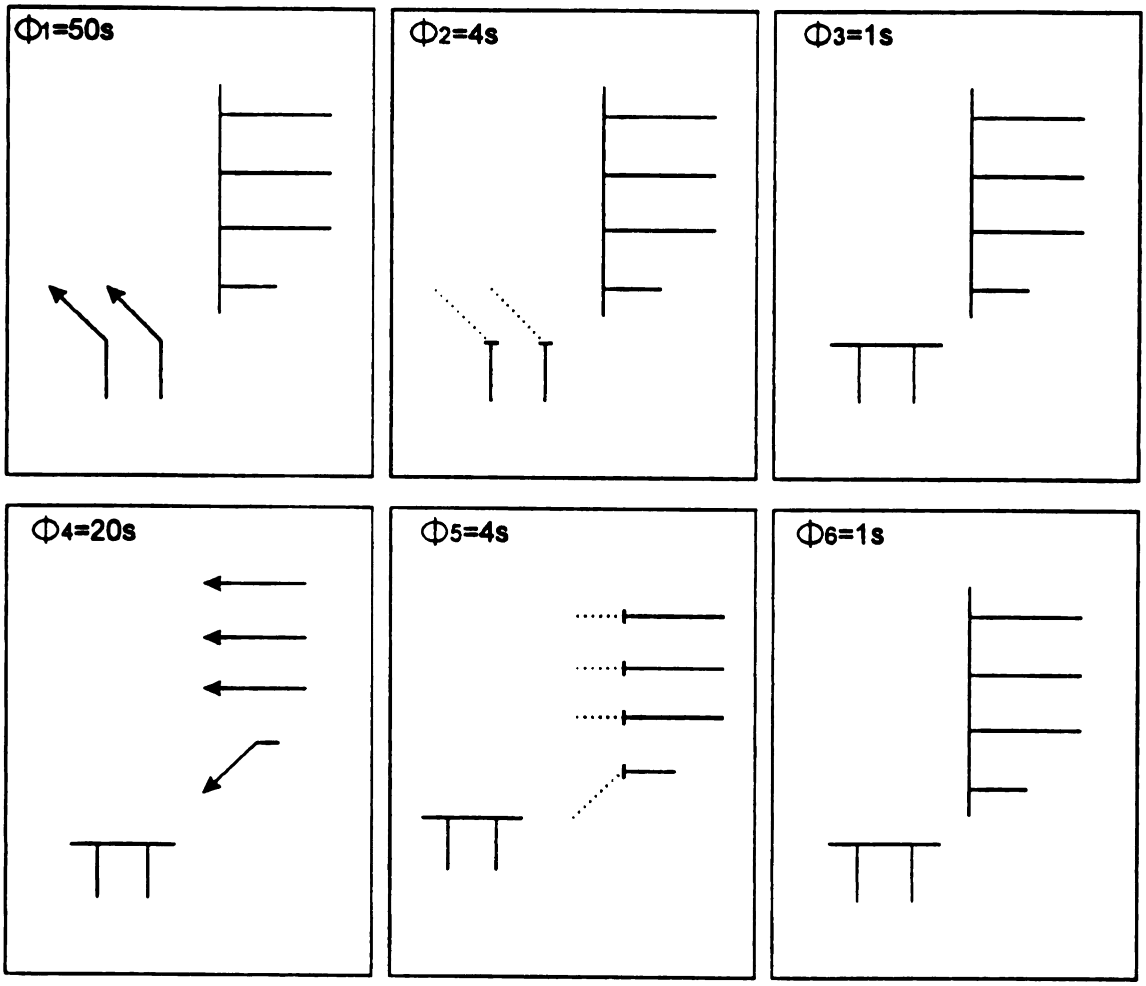


Figure 6.8: Phasing Diagram for MUDI Cross-overs

arterial, waiting at the crossover signal. For the best progression of the arterial traffic, this queue must begin to dissipate before indirect left turning traffic from the arterial reaches the crossover signal. This will result in an offset that is less than the 8.3 seconds. The study done by Dorothy, et. al. (8) determined the best crossover signal offset to be four seconds.

A signal phasing diagram was developed for the SPUI for the case where no frontage roads were present (Figure 6.9) and for the case where frontage roads were present (Figure 6.10). A concern with signalizing the SPUI is the need for a long phase change interval to allow traffic to clear the intersection. Thus, the minimum phase change interval of 5 seconds was increased to 9 seconds for all SPUI movements except for the frontage road movements.

Finally, the signal phasing diagram for the diamond (Figure 6.11) was determined. A concern with signalizing the diamond interchange is the need for a clearance interval to allow time for traffic which has turned left from the ramp and is stored on the structure to begin clearing before releasing arterial traffic. Thus, a 12 second clearance interval was provided. This clearance interval advances the green time for traffic stored in the median of the diamond, allowing it to clear the median area before giving the remaining arterial traffic a green indication.

6.4 Variables And Measures Of Effectiveness

There were four major variables of interest addressed in this study: traffic volumes, turning percentages, frontage roads and distance to the closest downstream node.

The networks were loaded by considering the percent saturation of the entry links of the arterial. For the entry links of the arterial, it was assumed that each entry lane had a

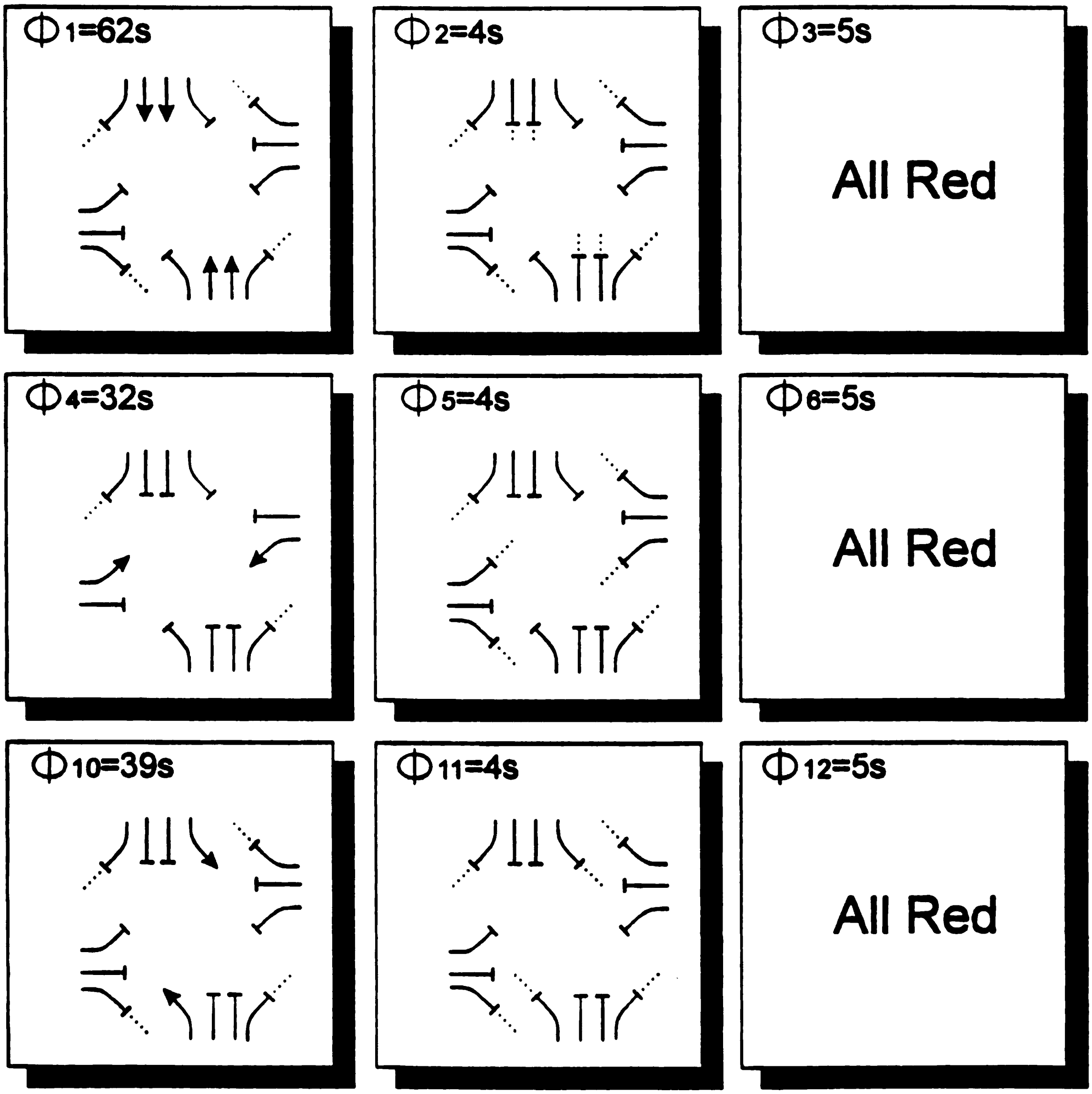


Figure 6.9: Phasing Diagram for SPUI Configuration without Frontage Roads

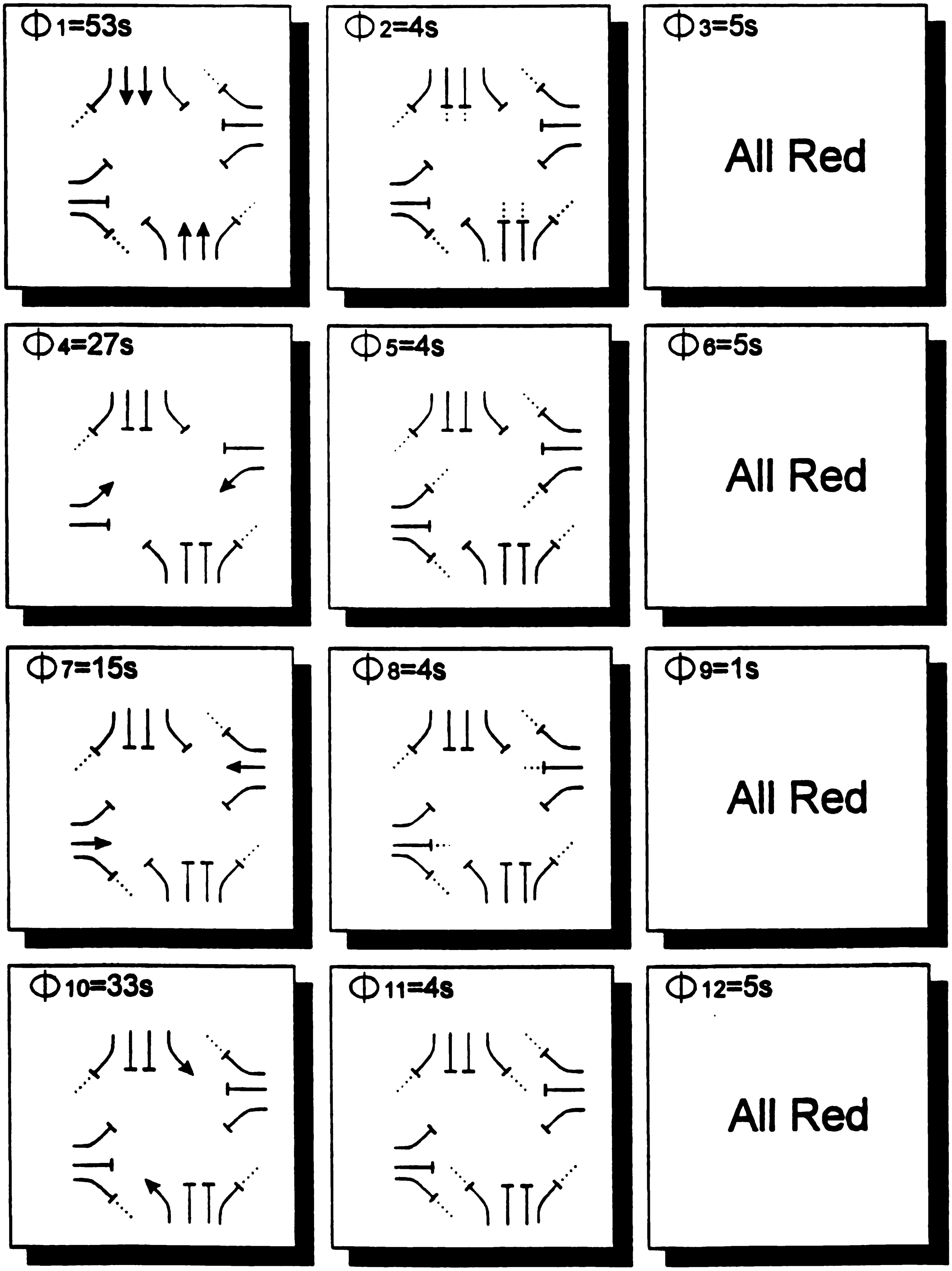


Figure 6.10: Phasing Diagram for SPUI Configuration
with Frontage Roads

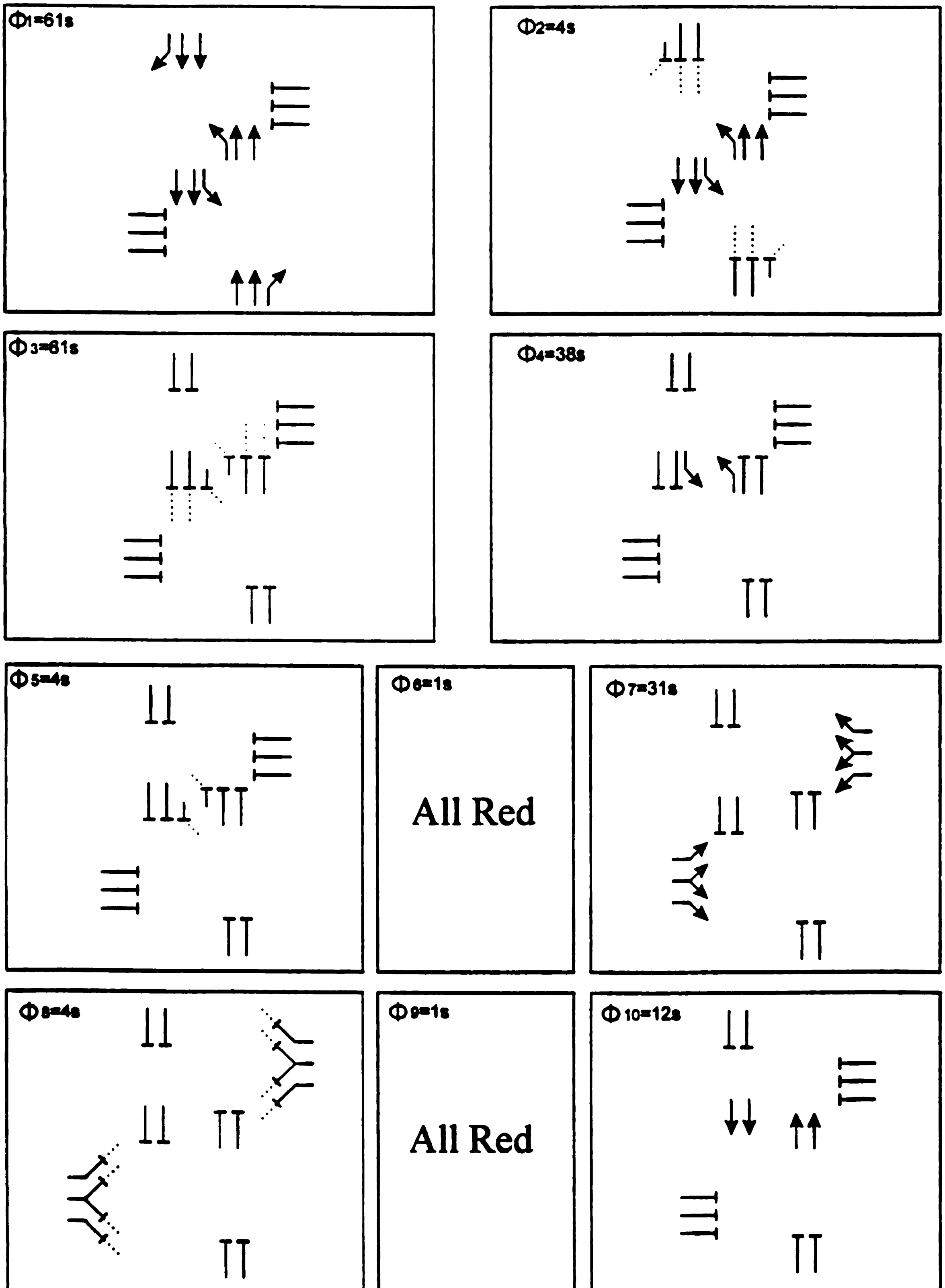


Figure 6.11: Phasing Diagram for Diamond Configuration

capacity of 1800 vehicles per hour of green. With this in mind, a simple incremental volume structure was identified for study based on arterial entry link saturation values of 0.3, 0.5, 0.7, 0.9, and 1.0. The minor downstream crossroad entry links were assumed to have a per lane hourly volume ratio of 30/70 when compared to the arterial entry links. Furthermore, the network was modeled with an in-balance in traffic flow for both the frontage roads and exit ramps. It was assumed that there was a 70/30 imbalance in flow between traffic approaching from the left and traffic approaching from the right (Figures 6.1, 6.3, and 6.5). The maximum frontage road volume was assumed to be 600 vehicles per hour.

The second variable addressed was turning percentages. First, turns from the minor crossroad to the arterial were fixed at 20 percent toward the interchange and 10 percent away from the interchange. Turns from the arterial to the minor crossroad were fixed at 10 percent left and 10 percent right. Second, for arterial traffic approaching the interchange, it was assumed that 25 percent wanted to turn left to access the on-ramp, 25 percent wanted to turn right to access the other on-ramp, and 50 percent wanted to continue on the arterial. Third, turning traffic exiting the freeway was varied to test the sensitivity of the designs to the volume of left turning traffic. Thus, values of 30, 50, and 70 percent left turns from the exit ramps were modeled. Finally, it was assumed that the volume of traffic entering on a particular frontage road would also exit on that frontage road.

The third variable addressed was the existence of frontage roads. In Michigan, depressed freeway segments typically are built with frontage roads to access the adjacent properties. Thus, the operation of a particular interchange configuration with and without frontage roads was of interest.

The final variable addressed was the distance to the closest downstream node. Early in the project, a concern was raised about the effect that an interchange would have on a closely spaced intersection. In addition, it was desired to determine how an interchange configuration would function in an arterial that did not have perfect geometry. Thus, the distance to the closest downstream node was varied. To keep the size of the network constant, as a downstream node was moved closer to the interchange area, its counterpart on the other side of the interchange was moved and equal distance away from the interchange. The first value modeled was a spacing of 1.6 kilometers (one mile) to either side of the interchange area allowing for perfect progression on the arterial. The second value modeled was a spacing of 0.8 kilometers (one-half mile) on one side and 2.4 kilometers (one and one-half mile) on the other side. This spacing still allows for perfect progression of the arterial. However, the proximity of one of the downstream nodes to the interchange may be a factor. Finally, a spacing of 1.2 kilometers (three-fourths mile) to one side and 2.0 kilometers (one and one-quarter mile) to the other side of the interchange was modeled. This configuration does not allow for perfect progression along the arterial, but does keep a larger separation between the closest intersection and the interchange.

A TRAF-NETSIM simulation run produces an output that summarizes the traffic movements and various measures of effectiveness (MOEs) for both the network as a whole and for individual links. The MOEs that were selected for this study were: interchange area total time and downstream area total time. In TRAF-NETSIM, the MOE "total time" is made up of move time and delay time.

An effort was made to delineate an interchange area and a downstream area in the computer model. The physical size of these areas was the same for all models. However, inside the area, the size of the interchange may vary. The nodes numbered 7 and 8 were coded as dummy nodes (i.e. no change in the traffic stream occurs at them) to allow MOEs to be gathered for both the interchange area (the area bounded on the top by node 7 and on the bottom by node 8) and the downstream area (the area above node 7 plus the area below node 8).

A criticism of the indirect left-turn strategy used by the MUDI configuration is that while conflicts from left turning vehicles have been removed from the intersection, these drivers are penalized by being forced to travel a greater distance to use the cross over. Thus, delay cannot be used as a MOE, as it would be unclear if the delay savings at an intersection were being offset by the extra travel time imposed on left-turning traffic. Therefore, total time, which represents the amount of time all vehicles spent in the network as a combination of travel time and delay time, was selected as a MOE.

Chapter 7

SIMULATION RESULTS

Based on the variables selected for study, an hour of operation for 300 individual models was simulated. Each model used the same random number seed. Since TRAF-NETSIM brings the simulated network to equilibrium before starting to collect statistics and the network will be simulated for one hour of operation, the results should be repeatable and independent of the random number seed. The network was simulated for a saturation up to 100 percent to aid in determining when simulation results become invalid due to delay occurring outside the environment of the analysis. However, TRAF-NETSIM may produce unreliable results when run at levels of saturation approaching 100 percent. Thus, the results of the 100 percent saturation runs will not be discussed.

The values of start-up lost time and headway were not calibrated or validated based on field data. Since Michigan does not have a SPUI, it was not possible to determine what values would be applicable for Michigan drivers utilizing a SPUI. However, the default values imbedded in the model for start-up lost time (2.0 seconds) and headway (1.8 seconds), which are based upon national averages, were used for each interchange type.

A representative example of the findings are presented in the text, while all findings are presented in both tabular and graphical format in the appendices.

7.1 Interchange Performance without Frontage Roads

Figure 7.1 illustrates the performance of the interchange configurations without the presence of frontage roads and with a five-lane arterial cross-section. The situation modeled in this scenario is for the extreme case of 70 percent of the vehicles exiting the freeway and desiring to turn left onto the arterial. At 30 percent saturation, all three interchange configurations performed approximately the same. However, at 50 percent saturation, the total time for the MUDI and SPUI configurations was only 60 percent of that for the traditional diamond. Additionally, at 70 percent saturation, the total time for the MUDI configuration was 25 percent less than the SPUI and 36 percent less than the traditional diamond. Finally, at 90 percent saturation, the total time for the MUDI configuration was 16 percent less than the SPUI and 20 percent less than the traditional diamond.

Figure 7.2 illustrates the results when the percent left turns is reduced to 50 percent. Although the operational advantage of the MUDI is less, it is still meaningful and follows the same pattern as the 70 percent left-turn case outlined above. At 30 percent saturation, all three interchange configurations still performed approximately the same. At 50 percent saturation, the total time for the MUDI and SPUI configurations was 50 percent of that for the traditional diamond. Moreover, at 70 percent saturation, the total time for the MUDI configuration was 18 percent less than the SPUI and 38 percent less than the traditional diamond. Finally, at 90 percent saturation, the total time for the MUDI configuration was 23 percent less than the SPUI and 32 percent less than the traditional diamond.

As the percentage of left turns is decreased to 30 percent (Figure 7.3), the operational characteristics of both the MUDI and the SPUI configuration change at higher levels of

Figure 7.1: Interchange Area Total Time For 70% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial

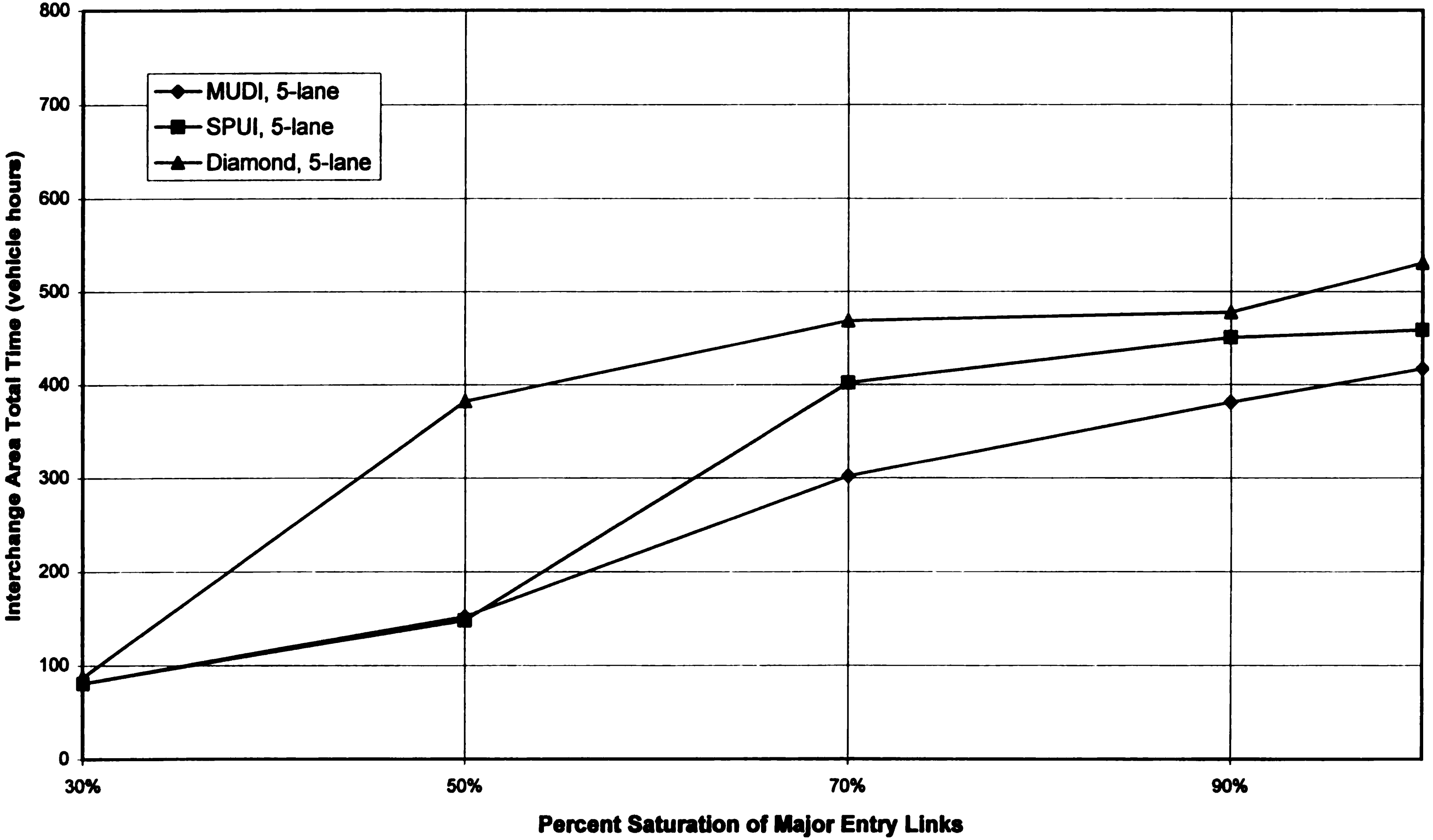


Figure 7.2: Interchange Area Total Time For 50% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial

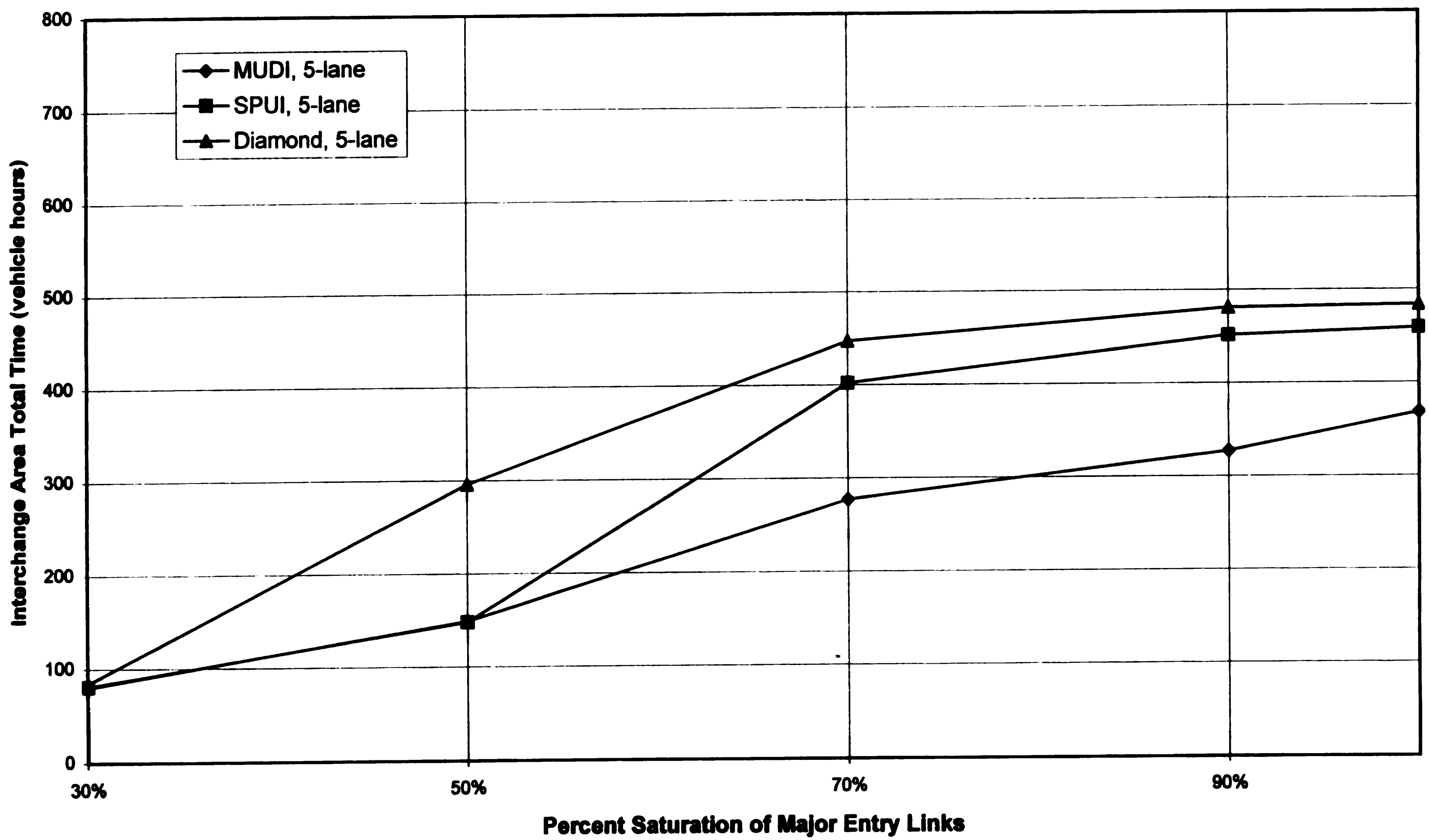
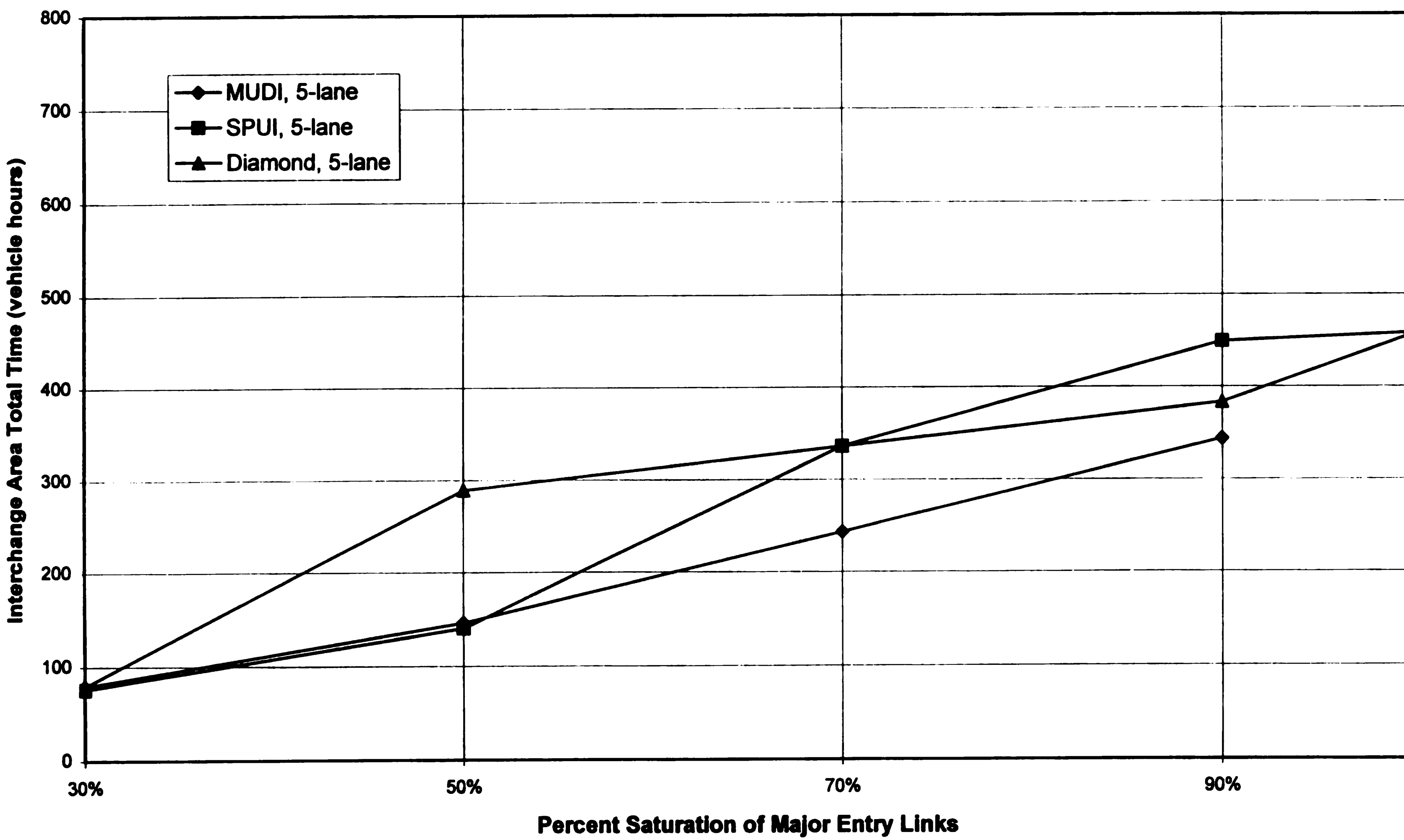


Figure 7.3: Interchange Area Total Time For 30% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial



saturation. At 30 percent saturation, all three interchange configurations are again approximately equal. In addition, at 50 percent saturation, the total time for the MUDI and SPUI configuration was again 50 percent that of the traditional diamond. However, at 70 percent saturation, the total time for the MUDI was 28 percent less than both the SPUI and traditional diamond, which perform approximately the same. Finally, at 90 percent saturation, the total time for the MUDI was 23 percent less than the SPUI and 10 percent less than the traditional diamond. Thus, at 90 percent saturation, the traditional diamond is operationally superior to the SPUI, as measured by this single MOE.

Much of the same pattern is shown when the arterial cross-section is changed from a five-lane cross-section to a seven-lane cross-section (Figures 7.4-7.6). The major differences are that at 30 percent saturation, the total time for both the MUDI and SPUI was 35 to 40 percent less than the traditional diamond for all turning percentages. In addition, the MUDI with a seven-lane arterial begins to operationally outperform the SPUI at 50 percent saturation as opposed to at 70 percent saturation with a five-lane arterial.

Based on the MOE "interchange area total time", in all cases, the MUDI configuration either equals or exceeds the operational performance of the SPUI and traditional diamond configuration. These operational advantages are most pronounced when the percentage of left-turning traffic is high and the level of saturation is high. The operational advantages of the SPUI are greatly reduced as the percentage of left-turning traffic is reduced, with the traditional diamond outperforming the SPUI at high levels of saturation and low levels of left-turning traffic.

Figure 7.4: Interchange Area Total Time For 70% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial

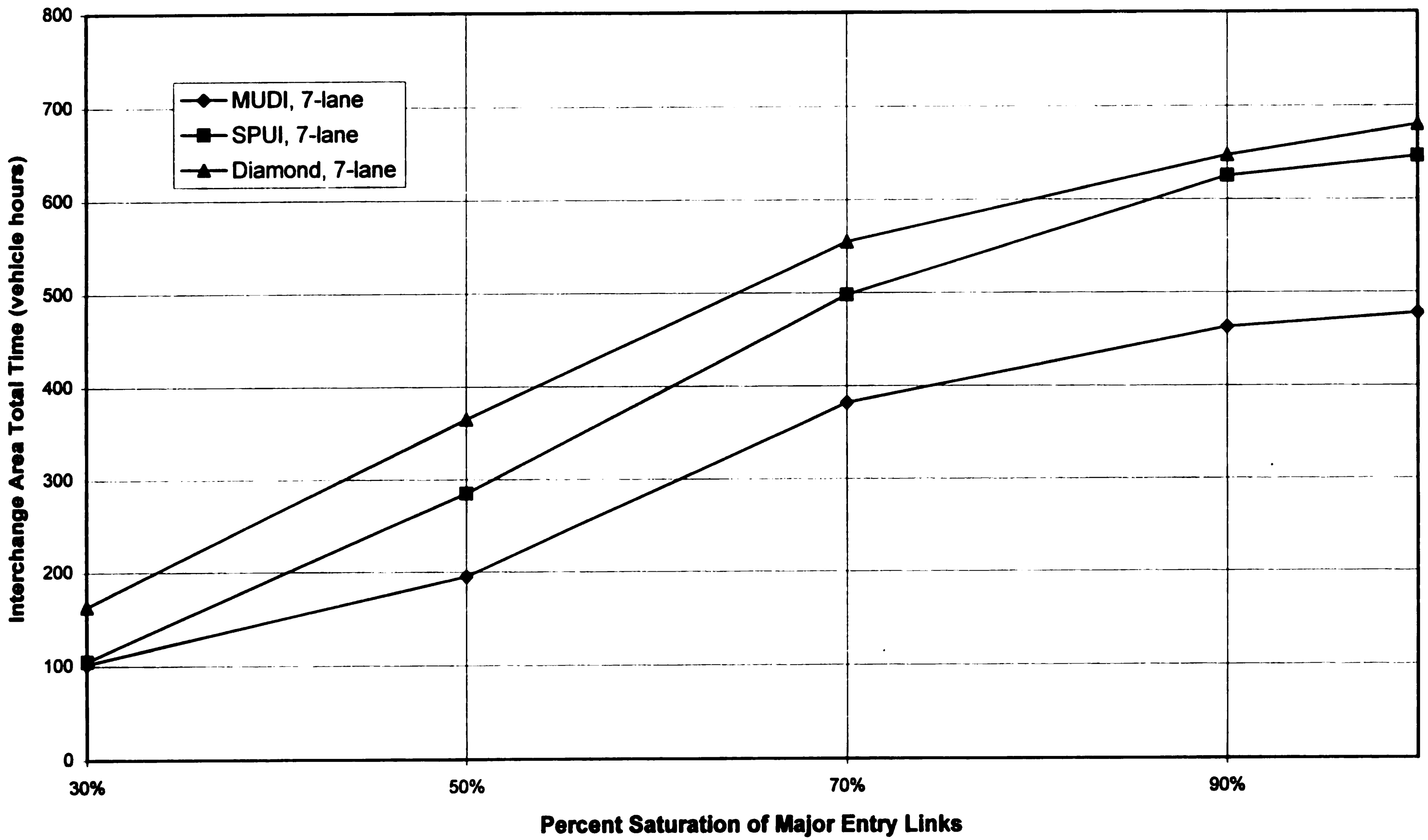


Figure 7.5: Interchange Area Total Time For 50% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial

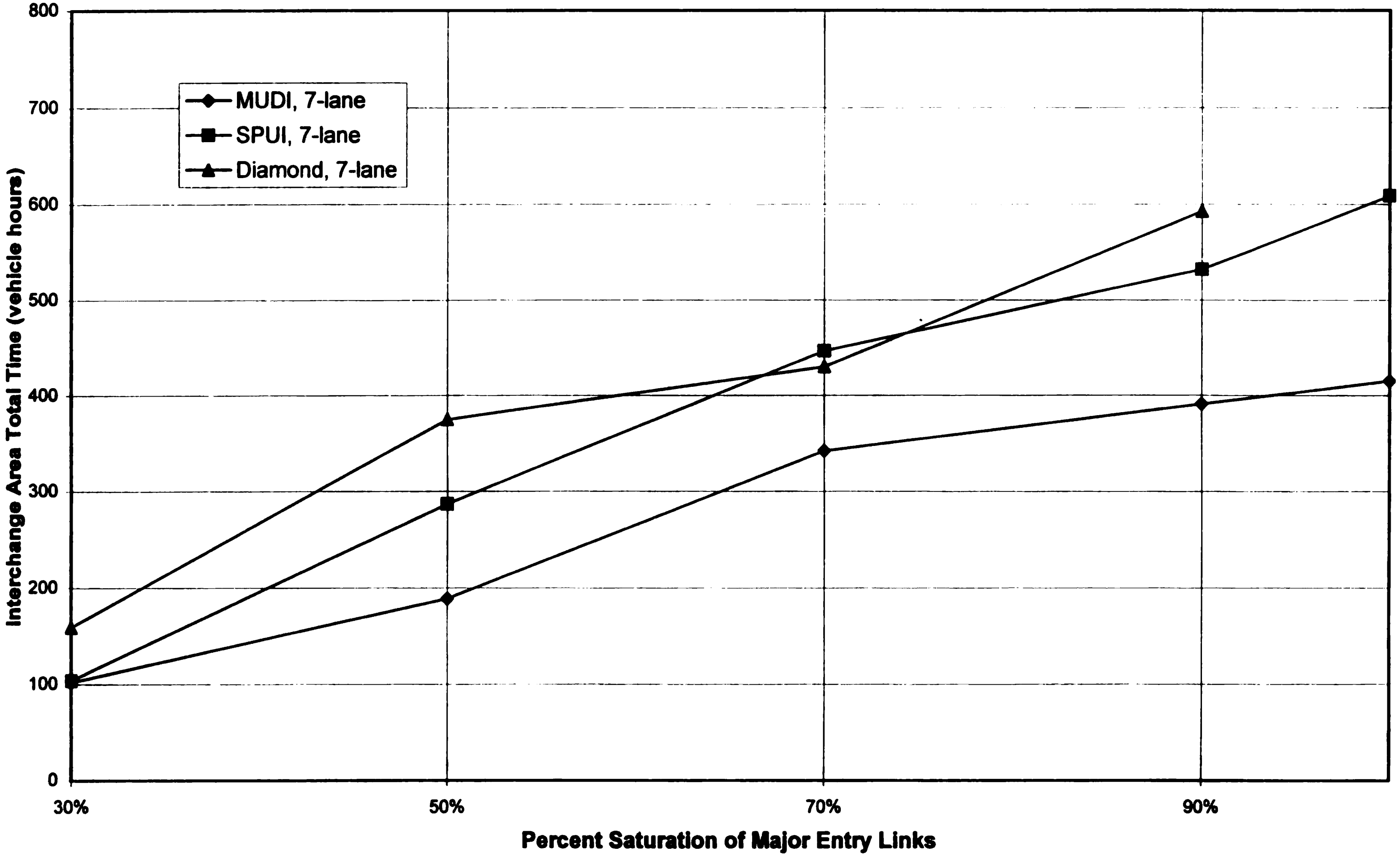
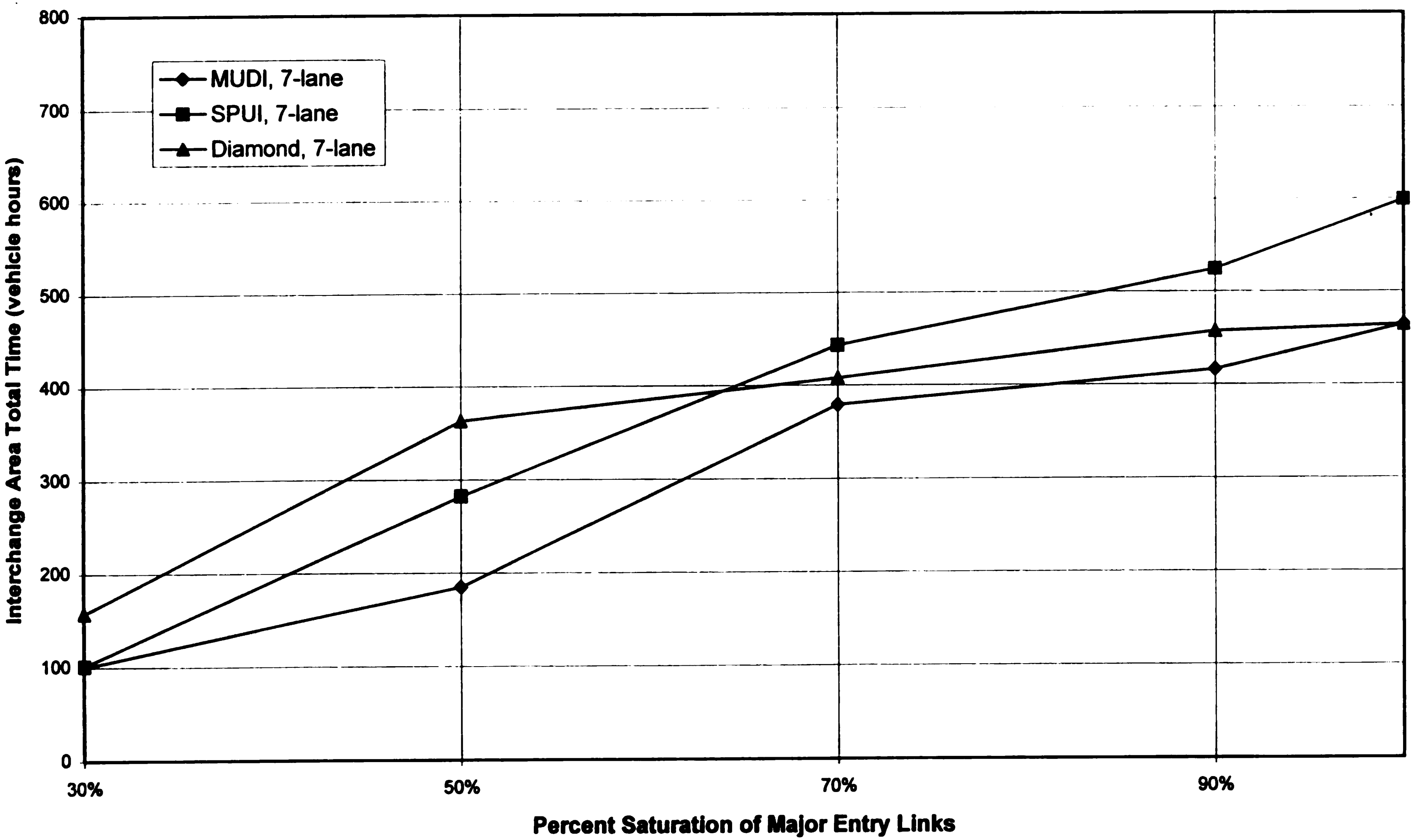


Figure 7.6: Interchange Area Total Time For 30% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial



7.2 Migration of Delay without Frontage Roads

In this research effort, there is concern that greatly enhanced urban interchange configurations may demonstrate an improved operation at the freeway, but may merely move the delay to the first signalized intersection upstream or downstream. Thus, their advantages (if any) may be exaggerated. Therefore, this analysis also evaluated the operation of the downstream nodes.

As illustrated in Figure 7.7, which is a specific case with 50 percent left turns, five-lane arterial cross-section and no frontage roads, there was no evidence that either the MUDI or SPUI configuration resulted in moving delay to the downstream nodes. However, the total time for the downstream area when fed by traffic from the traditional diamond interchange is greater for all but the 30 percent saturation level, suggesting a migration of delay. In addition, when the specific case with 50 percent left turns, seven-lane arterial cross-section and no frontage roads (Figure 7.8) is examined, this trend continues for the traditional diamond. At 70 percent saturation, the modeling of the SPUI also shows this effect.

7.3 Interchange Performance with Frontage Roads

Many, if not most, of the MUDIs in Michigan are located where frontage roads are provided. Usually these frontage roads parallel the urban freeway for a considerable distance and provide access to abutting property. The need for local access in a major urban area was a primary consideration in the evolution of the MUDI design since frontage roads would need to be provided.

Figure 7.9 illustrates the performance of the interchange configurations with the presence of frontage roads, a left-turning percentage of 70 percent and a five-lane cross-road.

Figure 7.7: Downstream Area Total Time For 50% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial

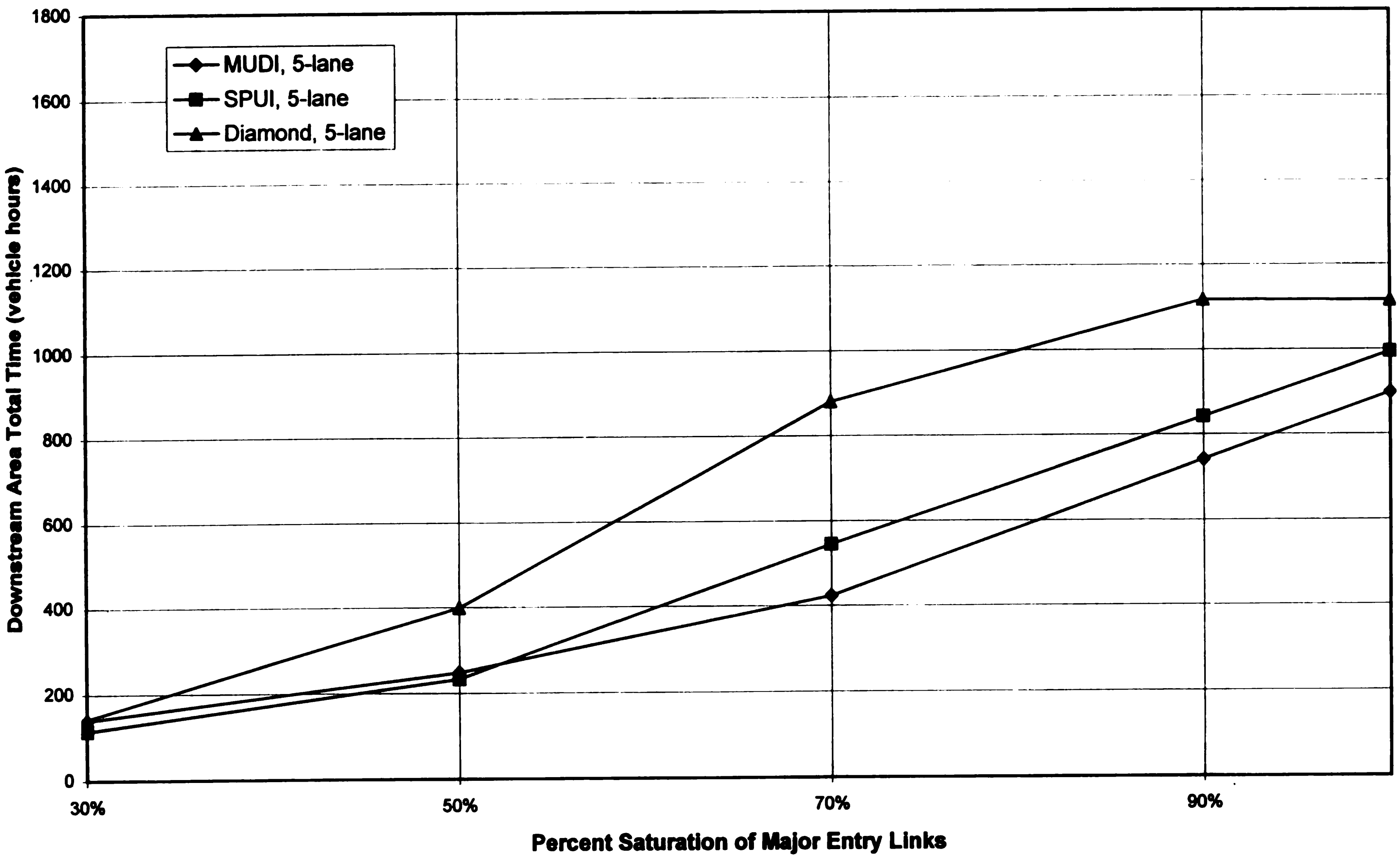


Figure 7.8: Downstream Area Total Time For 50% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial

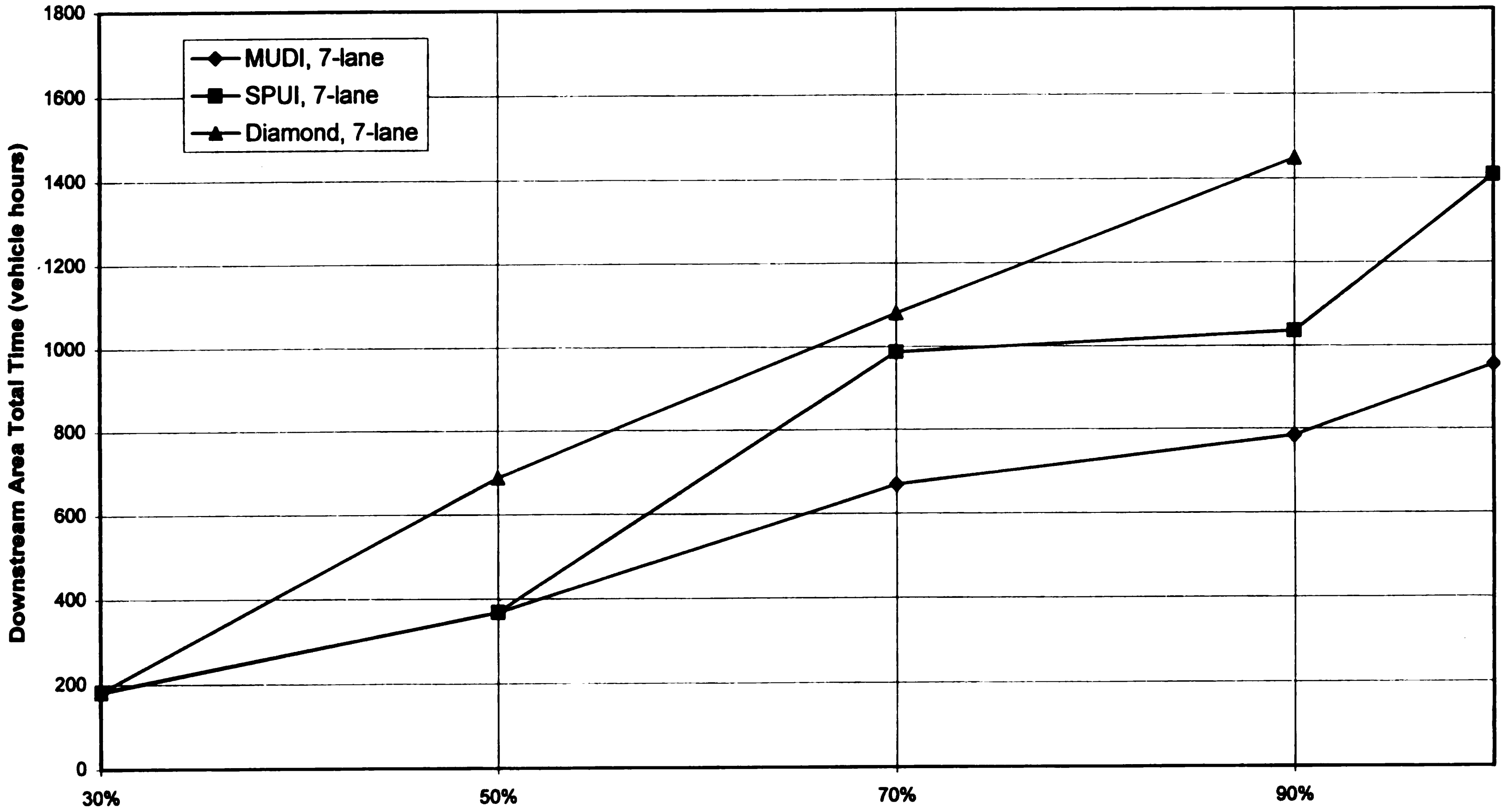
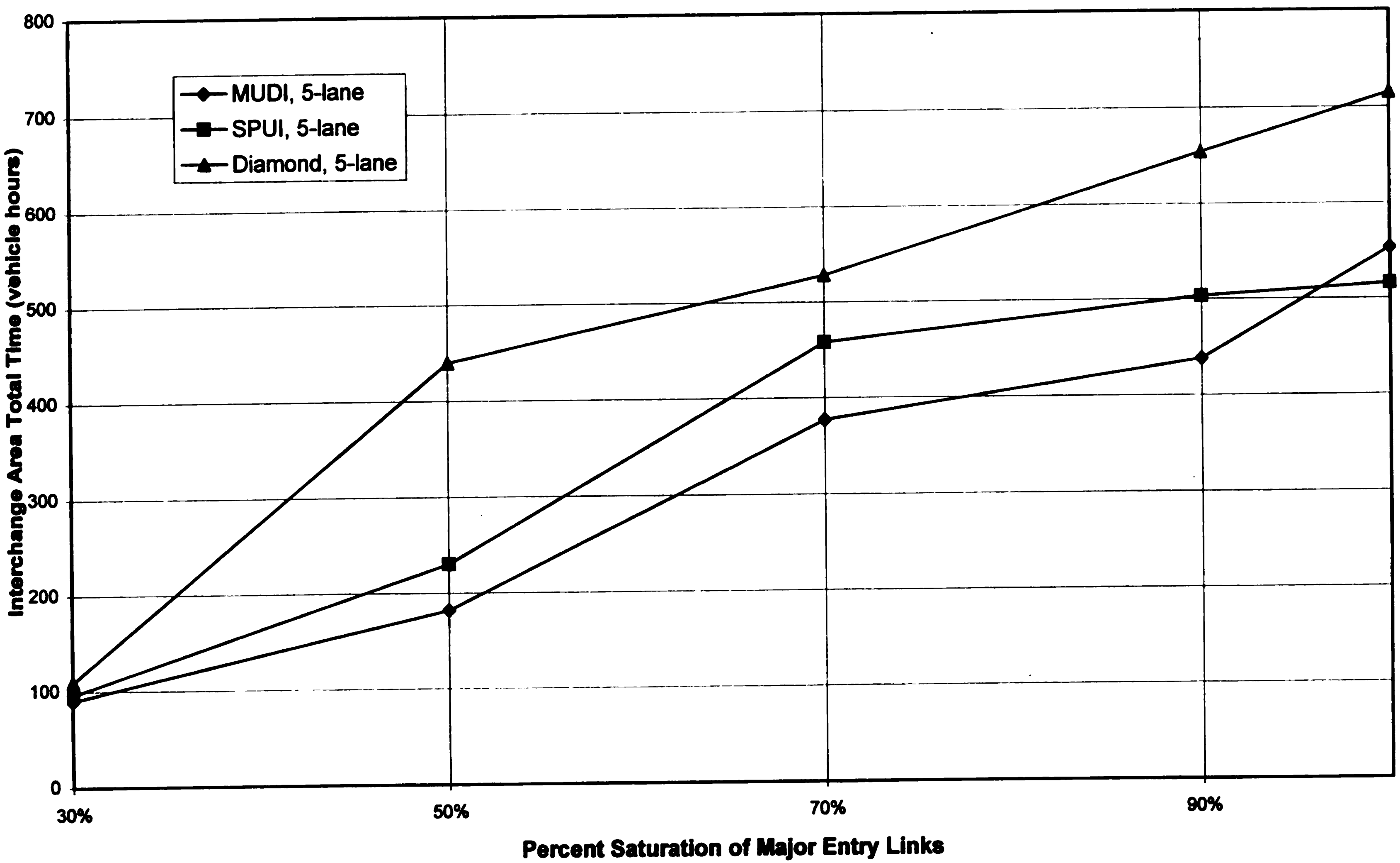


Figure 7.9: Interchange Area Total Time For 70% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial



At 30 percent saturation, all three interchange configurations performed approximately the same, which is consistent with the results from simulations without frontage roads. However, at 50 percent saturation, the total time for the MUDI configuration was 21 percent less than the SPUI and 59 percent less than the traditional diamond. This represents a divergence from the results of simulations without frontage roads, in which the MUDI and SPUI performed the same at 50 percent saturation. At 70 percent saturation, the total time for the MUDI configuration was 18 percent less than the SPUI and 29 percent less than the traditional diamond. Finally, at 90 percent saturation, the total time for the MUDI configuration was 13 percent less than the SPUI and 33 percent less than the traditional diamond.

Figure 7.10 further illustrates the performance of the interchange configurations with both frontage roads and five-lane arterial cross-sections. However, the percentage of left-turning traffic has been reduced to 50 percent in this case. At 30 percent saturation, all three interchange configurations continue to perform approximately the same. At 50 percent saturation, the total time for the MUDI configuration was 12 percent less than the SPUI and 59 percent less than the traditional diamond. These results are consistent with the scenario involving 70 percent left-turns outlined above. However, the results diverge from the results of the scenario involving no frontage roads, in which the MUDI and SPUI performed similarly at this level of saturation. At 70 percent saturation, the total time for the MUDI configuration was 21 percent less than the SPUI and 38 percent less than the traditional diamond. Finally, at 90 percent saturation, the total time for the MUDI configuration was 23 percent less than the SPUI and 25 percent less than the traditional diamond.

Figure 7.11 illustrates the performance of the interchange configurations with the presence of frontage roads, 30 percent left-turning traffic and a five-lane arterial cross-section.

Figure 7.10: Interchange Area Total Time For 50% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial

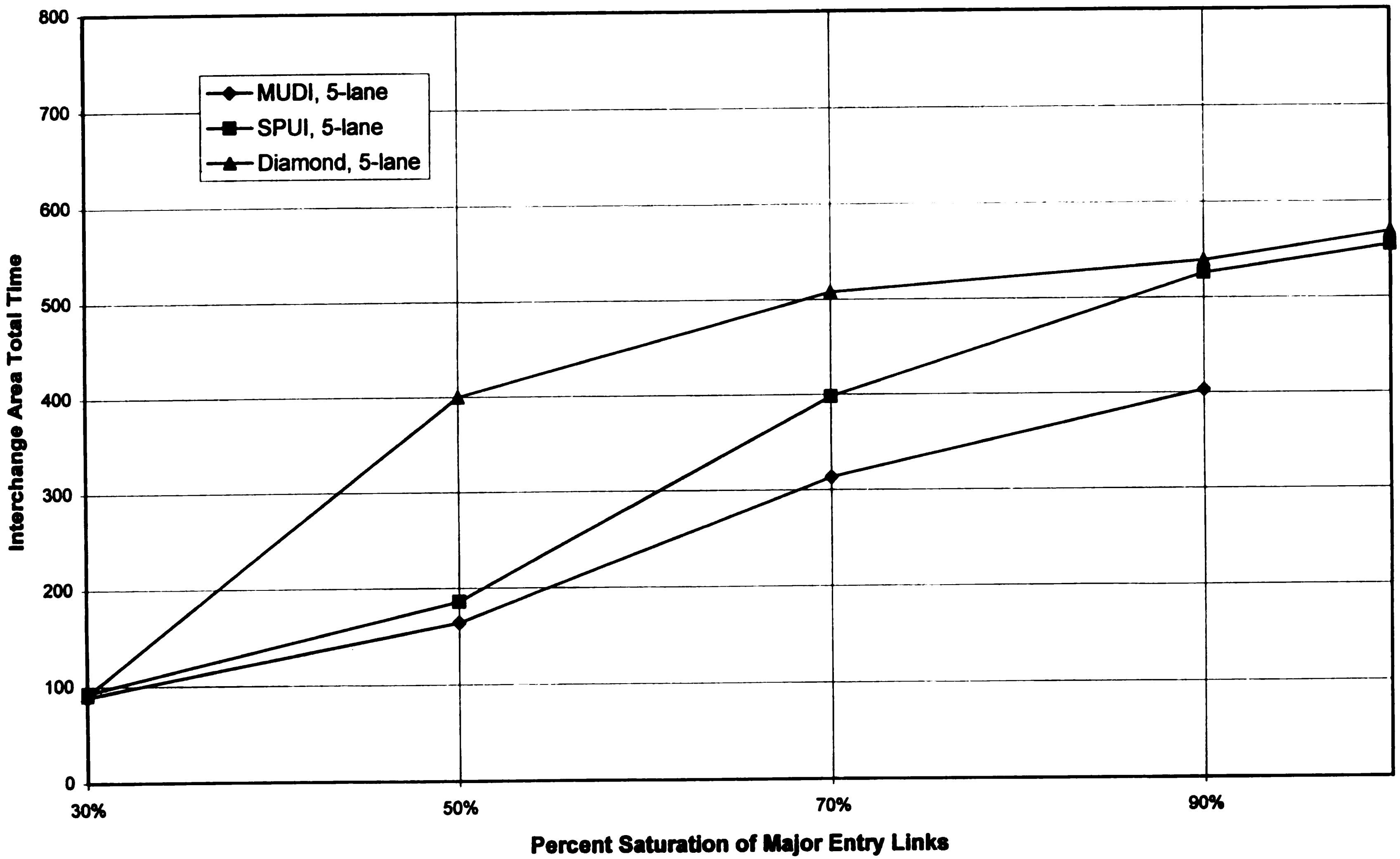
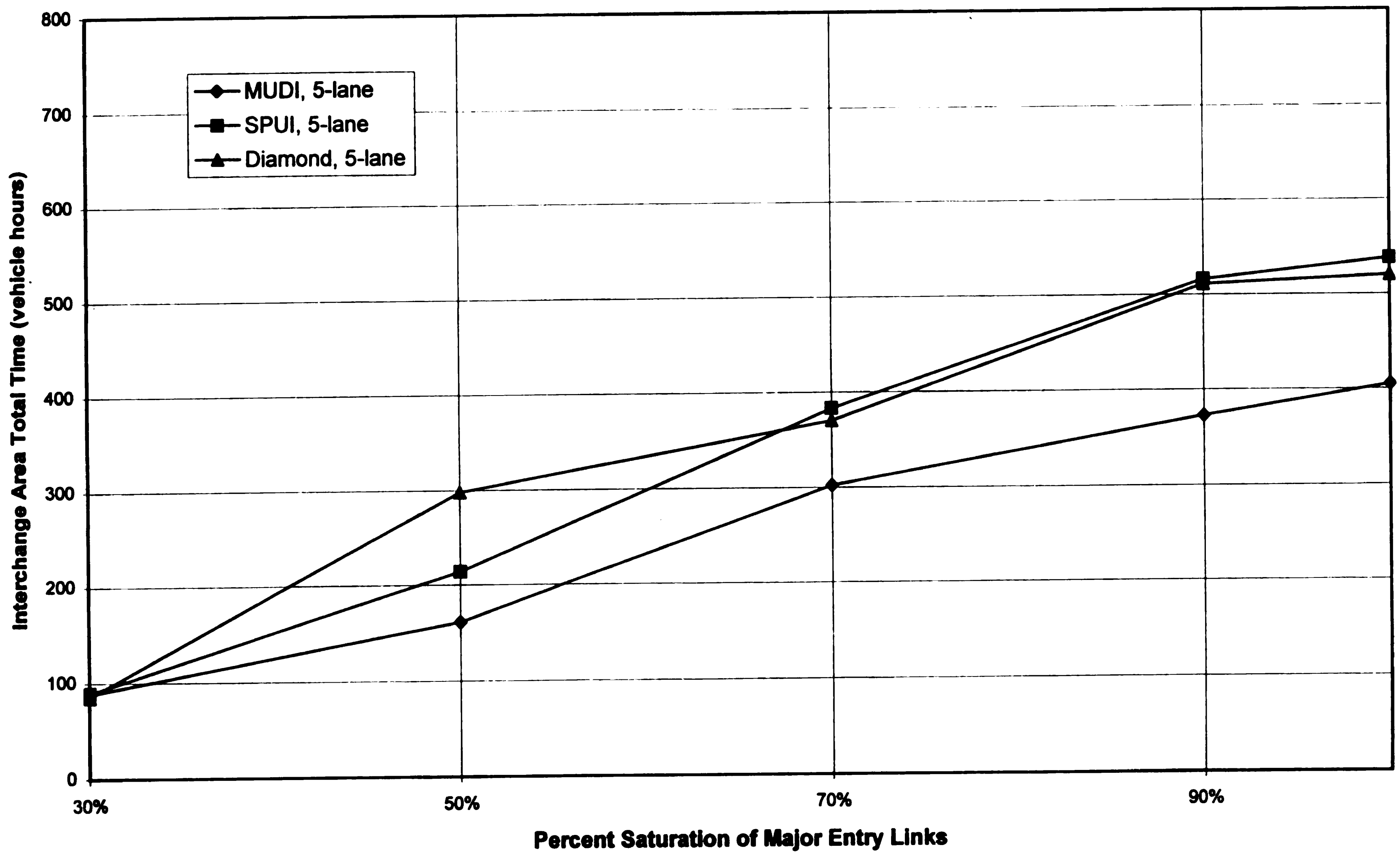


Figure 7.11: Interchange Area Total Time For 30% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial



At the lowest level of saturation, all three interchanges continue to perform approximately the same. However, at 50 percent saturation, the total time for the MUDI configuration was 25 percent less than the SPUI and 46 percent less than the traditional diamond. This continues the trend of the SPUI incurring greater total time than the MUDI (at 50 percent saturation) as was the case for the scenarios without frontage roads. At 70 percent saturation, the total time for the MUDI configuration was 21 percent less than the SPUI and 18 percent less than the traditional diamond. Finally, at 90 percent saturation, the SPUI and traditional diamond continued to perform approximately the same. The total time for the MUDI configuration was 28 percent less than the SPUI and 27 percent less than the traditional diamond.

Much the same pattern is shown when the arterial cross-section is changed from a five-lane to a seven-lane cross-section (Figures 7.12-7.14). As with the scenarios having no frontage roads, one major difference was that at 30 percent saturation, the total time for both the MUDI and SPUI was 35 to 40 percent less than that of a traditional diamond for all turning percentages. Additionally, for all left turning percentages, at 90 percent saturation, the traditional diamond operationally outperforms the SPUI. Moreover, for left-turning percentages of 50 and 30 percent, the SPUI performed similar to the traditional diamond at saturation levels of 50 and 70 percent. However, in the scenario where the left-turning percentage was set at 70 percent, the results of the MUDI simulations are not valid past the 70 percent saturation mark. This is due to a spillback of traffic on one of the model's entry links, which resulted in delay occurring outside the environment of the analysis.

As with the scenarios involving the performance of the interchange configurations without frontage roads, based on the MOE "interchange area total time," the MUDI

Figure 7.12: Interchange Area Total Time For 70% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial

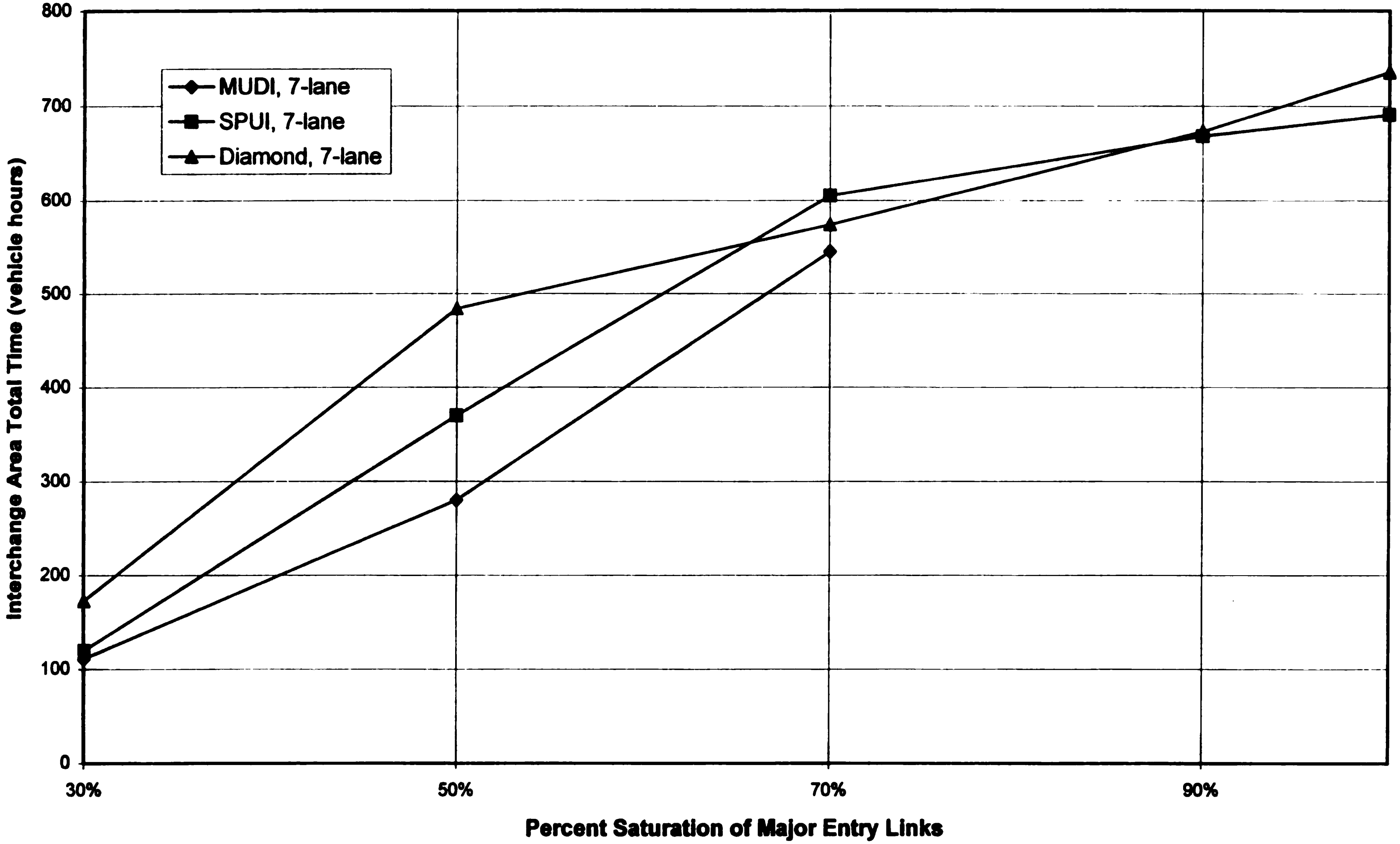


Figure 7.13: Interchange Area Total Time For 50% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial

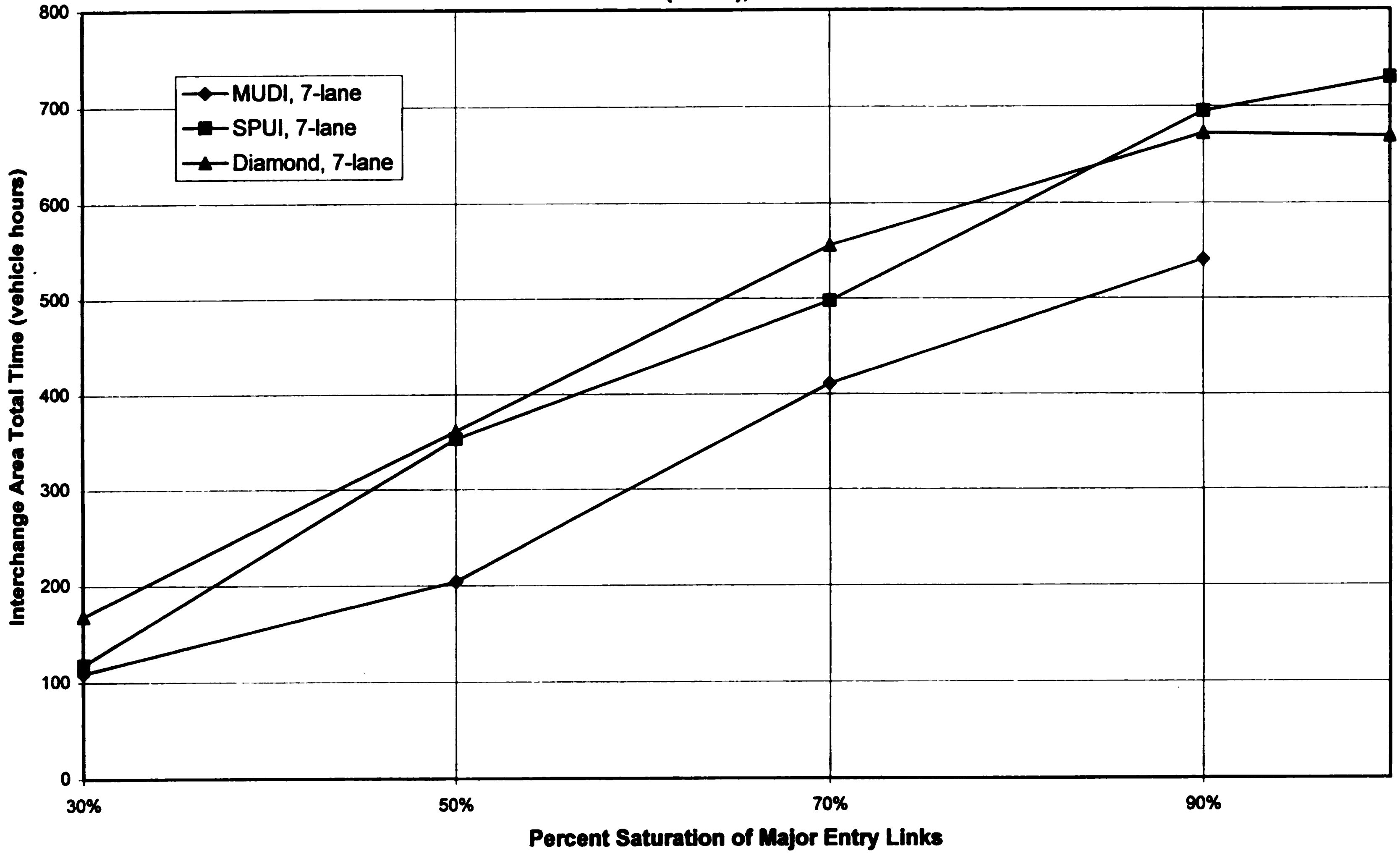
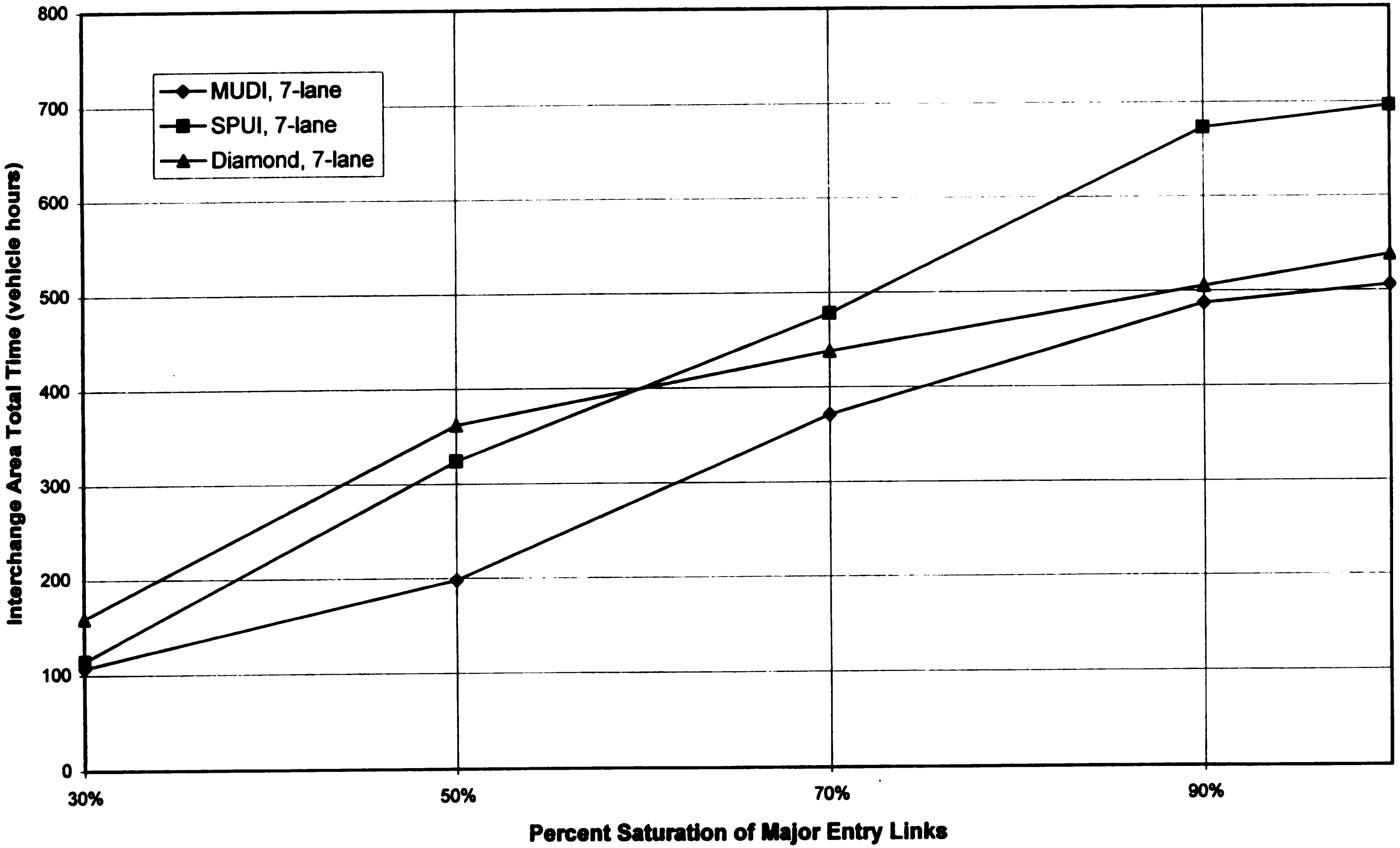


Figure 7.14: Interchange Area Total Time For 30% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial



configuration with frontage roads either equaled or outperformed both the SPUI and the traditional diamond, except where the MUDI could not be evaluated.

7.4 Migration of Delay with Frontage Roads

When the operation of the downstream nodes was examined for evidence of the migration of delay, a trend was evident. For example, in the scenario representing 50 percent left-turning traffic, frontage roads and a five-lane arterial cross-section (Figure 7.15), there is evidence of a migration effect from both the SPUI and traditional diamond interchange configurations. This trend is also exhibited when the arterial cross-section is widened to seven-lanes (Figure 7.16). Thus, for all cases involving frontage roads, the MUDI was operationally superior in having less migration of delay to the downstream intersections.

7.5 Sensitivity to Proximity of Closest Downstream Node

The effect that the proximity of the closest downstream node has on either the MUDI or SPUI interchange operation was also studied. Three spacing scenarios were considered:

- 1.6 kilometers (one mile) which allows for perfect progression along the arterial while maintaining adequate separation between the intersection and interchange area;
- 1.2 kilometers (three-quarter mile) which does not allow perfect progression along the arterial, but still maintains adequate separation between the intersection and interchange area;
- 0.8 kilometers (one-half mile) which allows for perfect progression along the arterial, but the proximity of the intersection to the interchange area may affect operation.

Figure 7.15: Downstream Area Total Time For 50% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial

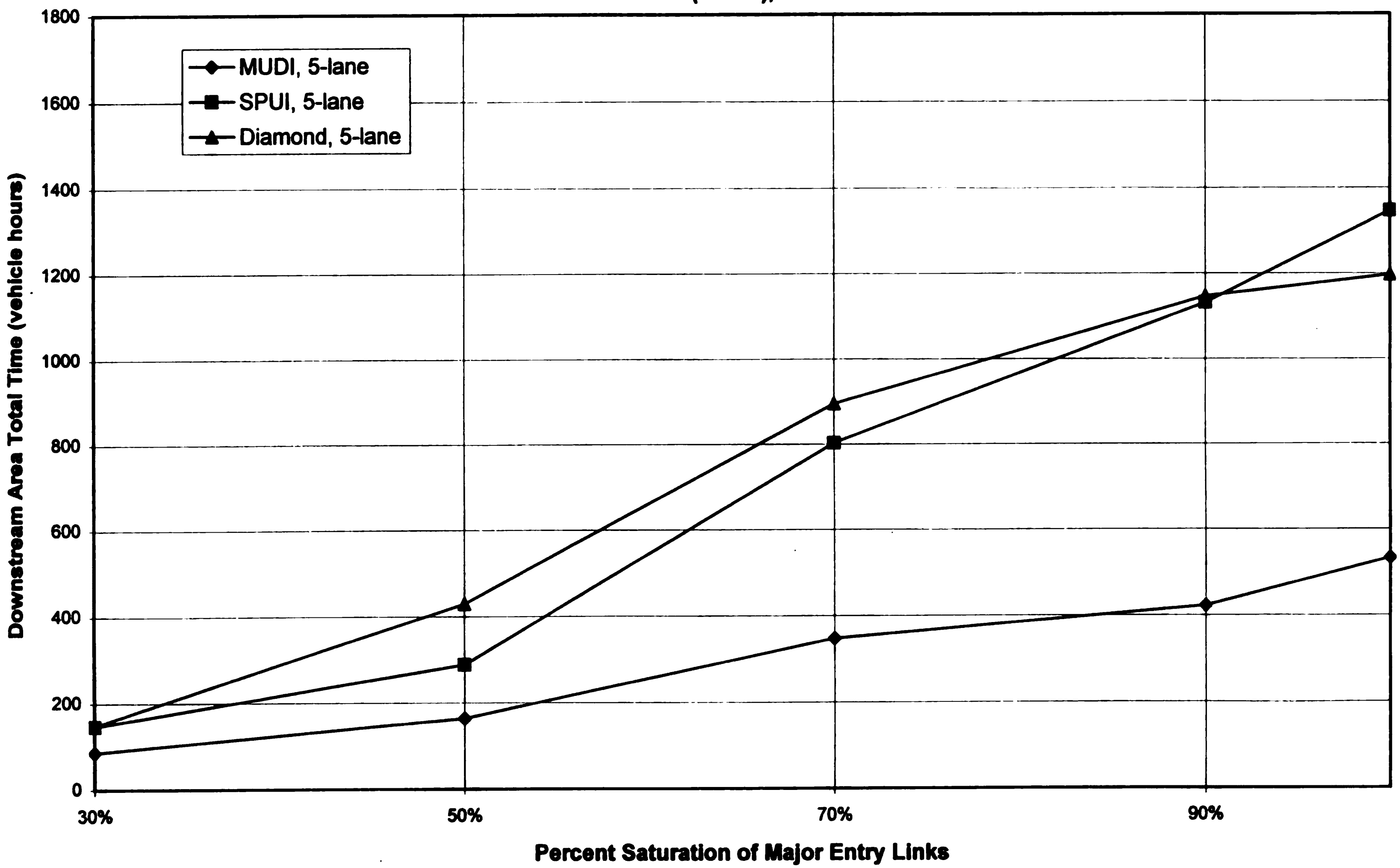
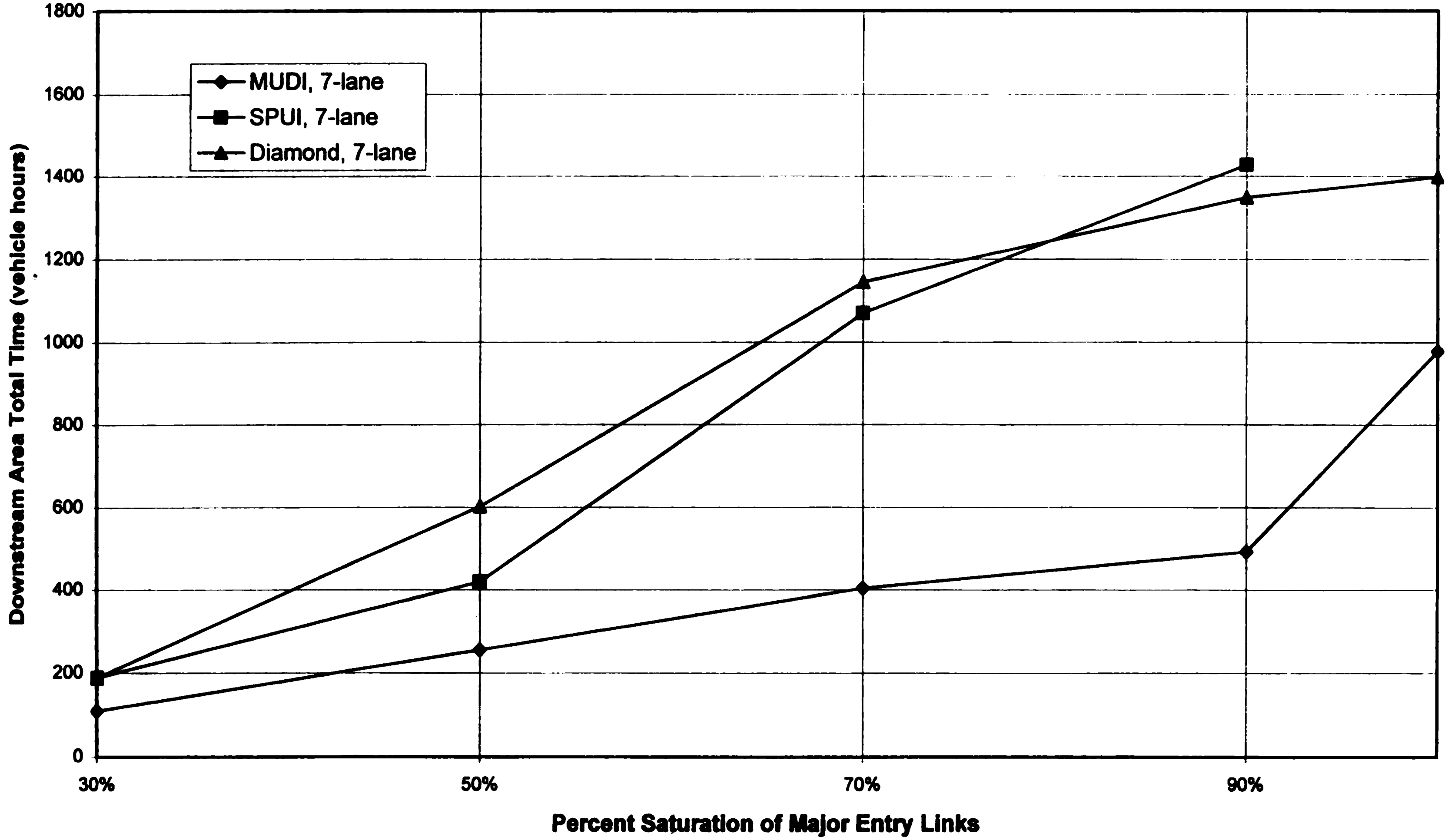


Figure 7.16: Downstream Area Total Time For 50% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial



All the scenarios involving sensitivity testing of the proximity of the downstream node were modeled without the presence of frontage roads.

When modeled with a five-lane arterial cross-section, 70 percent left-turns and 30 to 50 percent saturation, the MUDI configuration (Figure 7.17) performed approximately the same for all three spacing scenarios. In addition, the MUDI configurations with the closest downstream node placed at 1.6 kilometers (one-mile) and 1.2 kilometers (three-quarter mile) from the interchange continued to perform approximately the same for all levels of saturation. However, at 70 percent saturation and greater, the MUDI configuration with the closest downstream node placed at 0.8 kilometers (one-half mile) from the interchange exhibited greater interchange area total time than the other two MUDI spacing scenarios. At 70 percent saturation, the MUDI 0.8 kilometer spacing scenario had approximately 40 percent more total time than the other MUDI spacing scenarios, while at 90 percent saturation, the total time was 35 percent more.

When the percent left-turns was reduced to 50 percent (Figure 7.18), the simulation results for the MUDI configuration were similar to that of the 70 percent left-turning scenario described above. However, when the percent left-turns was reduced to 30 percent (Figure 7.19), the MUDI configuration performed approximately the same for all three spacing scenarios and all levels of saturation. In addition, when the arterial cross-section was changed to seven lanes, the MUDI configuration performed approximately the same for all three spacing scenarios and all levels of saturation.

Thus, the only conditions where the MUDI configuration was affected by the spacing of the closest downstream node were the scenarios using 70 percent left turning traffic, an

Figure 7.17: Interchange Area Total Time for 70% Left Turns, w/out Frontage Roads, 5-lane Arterial, Varying Spacing Scenarios

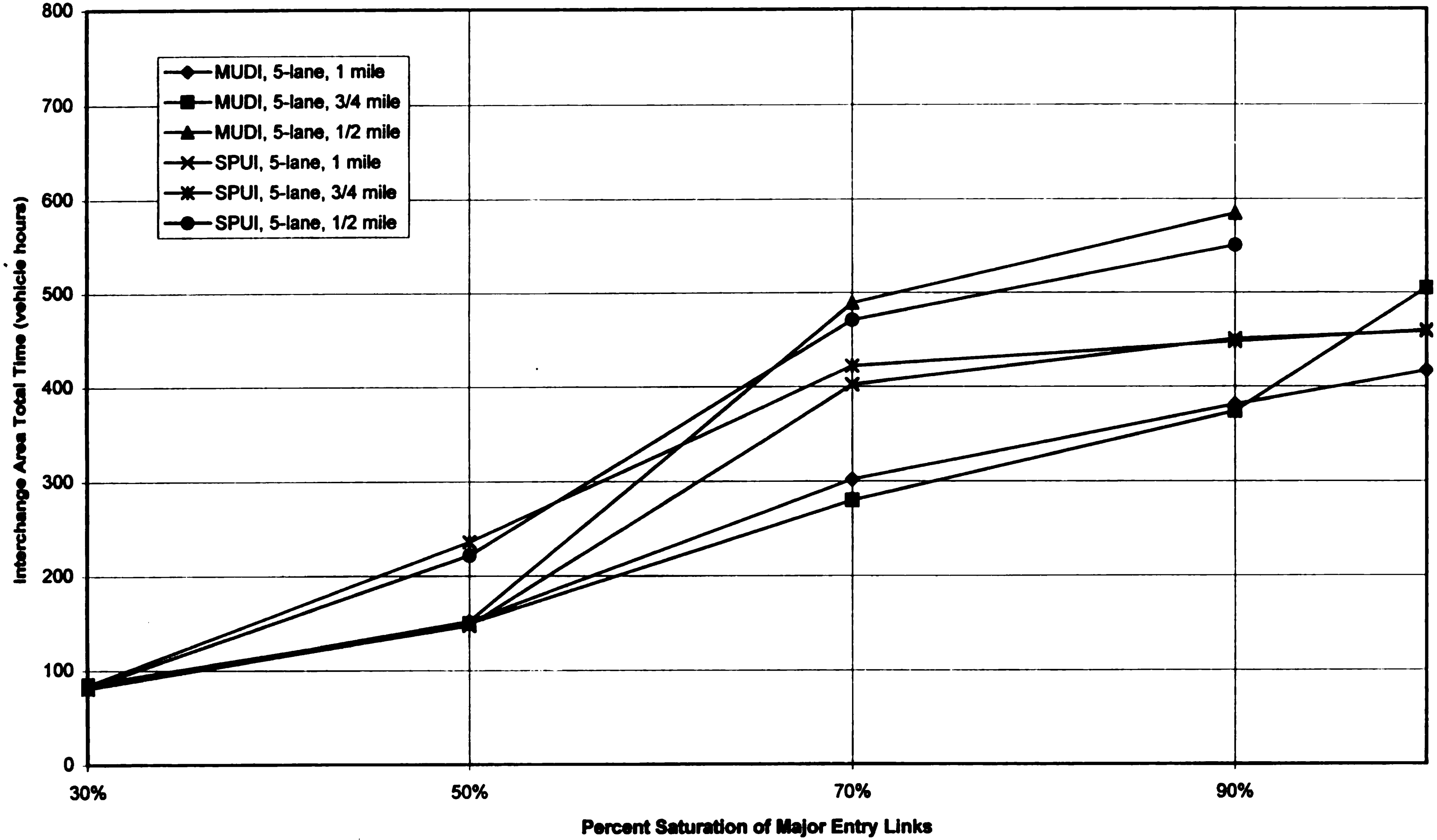


Figure 7.18: Interchange Area Total Time for 50% Left Turns, w/out Frontage Roads, 5-lane Arterial, Varying Spacing Scenarios

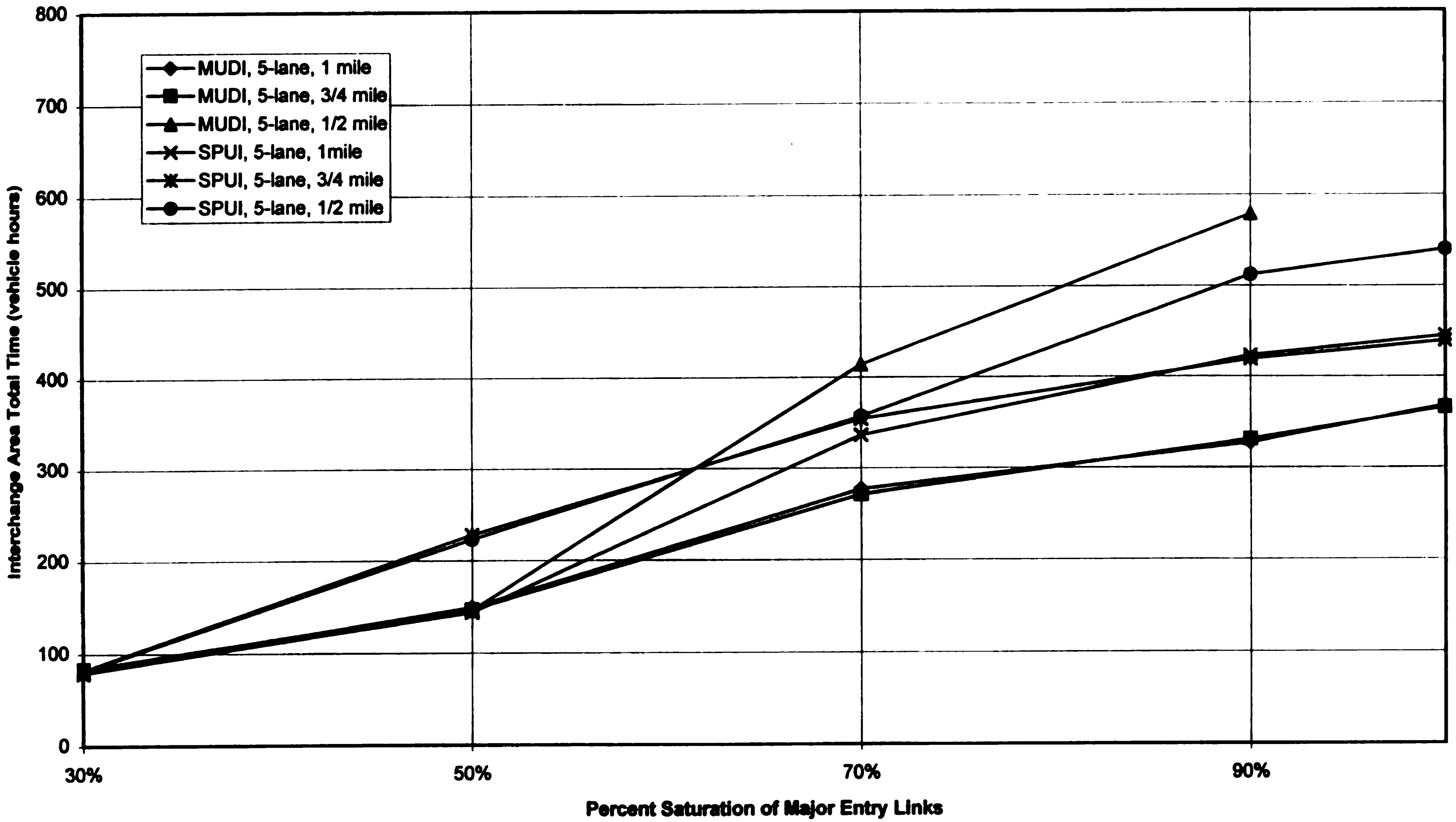
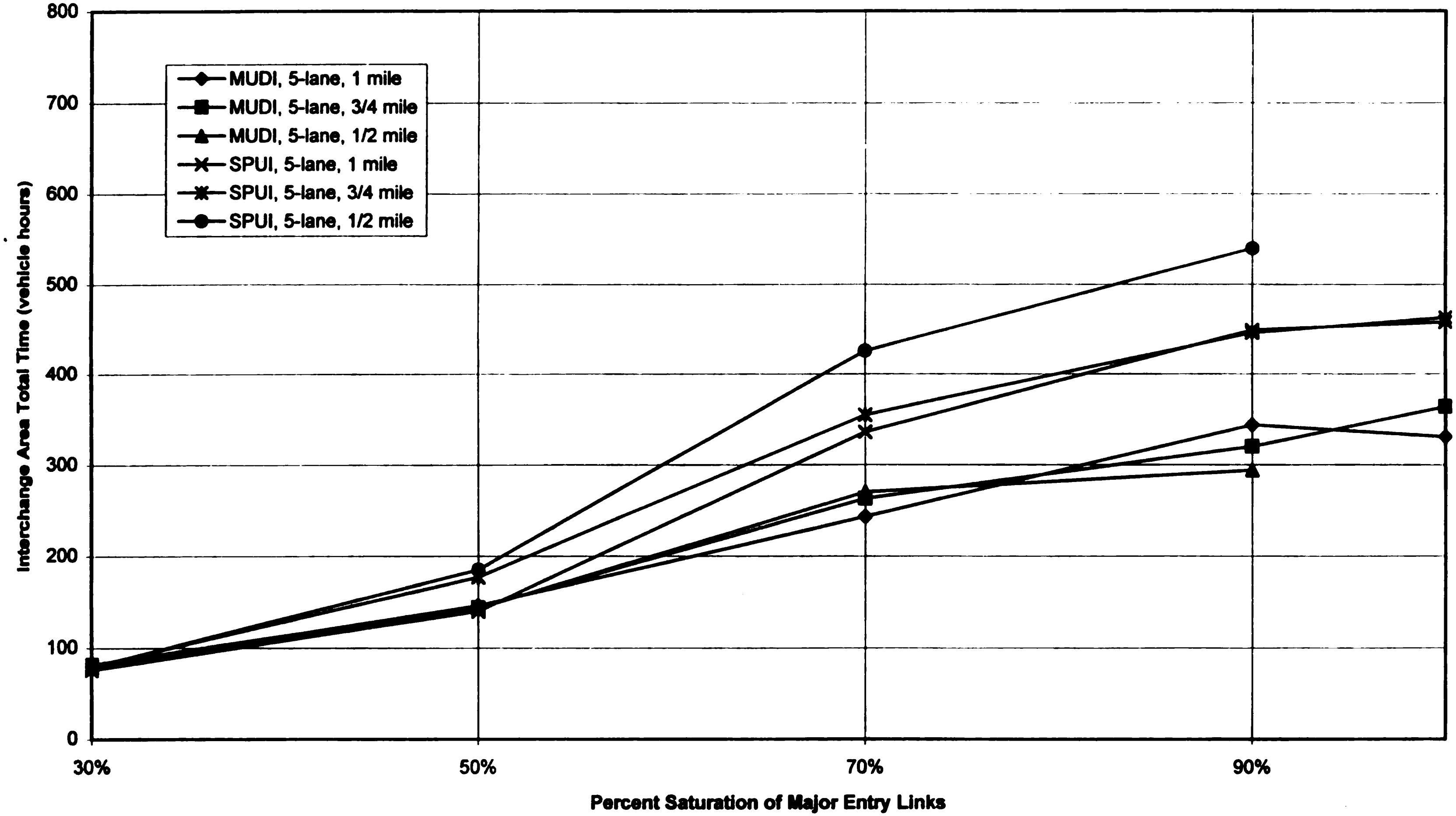


Figure 7.19: Interchange Area Total Time for 30% Left Turns, w/out Frontage Roads, 5-lane Arterial, Varying Spacing Scenarios

Figure 7.19: Interchange Area Total Time for 30% Left Turns, w/out Frontage Roads, 5-lane Arterial, Varying Spacing Scenarios



arterial cross-section of five lanes, saturation levels of 70 percent or greater and a proximity of 0.8 kilometers (one-half mile). However, the models were coded with an imbalance in traffic flow of 70/30 between traffic approaching from the left and traffic approaching from the right (Figure 6.3). Since, this increase in total time only appeared with a left-turning percentage of 70 percent, an arterial cross-section of five lanes and when the model was operating at near capacity, the most likely cause of this increase is a spillback from the limited storage available between the downstream intersection and the interchange.

When modeled with a five-lane arterial cross-section, 70 percent left turns and 30 percent saturation, the SPUI configuration (Figure 7.17) performed approximately the same for all three spacing scenarios. However, at 50 percent saturation, the interchange area total time for the SPUI configurations with 1.2 kilometer (three-quarter mile) and 0.8 kilometer (one-half mile) separation was approximately 35 percent greater when compared to the 1.6 kilometer (one mile) spacing scenario. At saturation levels of 70 percent or greater, the SPUI configuration with 1.6 kilometer (one mile) separation performed approximately the same as the SPUI configuration with a 1.2 kilometer (three-quarter mile) separation. However, at 70 and 90 percent saturation, the total time for the SPUI configuration with 0.8 kilometer (one-half mile) separation was approximately 15 percent and 20 percent greater, respectively, when compared to the other SPUI spacing scenarios. These results are also reflected in the performance of the SPUI configuration with a seven-lane arterial cross-section.

When the percent left-turns was reduced to 50 percent (Figure 7.18), the simulation results were similar to that of the 70 percent left-turn scenario for saturation levels of 30, 50, and 90 percent. However, at 70 percent saturation, the SPUI configuration performed approximately the same for all spacing scenarios. When the percent left-turns was reduced to

30 percent (Figure 7.19), the simulation results were also similar to the 70 percent left-turn scenario for all saturation levels. At both 50 and 30 percent left-turning traffic, the scenarios modeled with a seven-lane arterial cross-section reflected similar results.

Unlike the MUDI configuration, the total time for the SPUI configuration was adversely affected for all percent left-turning scenarios when the spacing to the closest downstream node was reduced to 0.8 kilometers (one-half mile). In addition, at 50 percent saturation, the scenarios modeling a separation of 1.2 kilometers (three-quarter mile) resulted in greater total time than the comparable models with a separation of 1.6 kilometers (one mile).

In all cases, the performance of the MUDI configuration with a separation of 1.6 kilometers (one mile) or 1.2 kilometers (three-quarter mile) either equals or exceeds the operational performance of the SPUI. In addition, for levels of saturation of 50 percent or less, the MUDI configuration with a separation of 0.8 kilometers (one-half mile) also either equals or exceeds the operational performance of the SPUI. Furthermore, at higher saturation levels, the operational performance of the SPUI configuration was adversely affected by a separation of 0.8 kilometers (one-half mile). Thus, in most cases, the MUDI configuration appears to be insensitive to the proximity of the closest downstream node, while the SPUI configuration is sensitive to the proximity of the downstream node.

For both arterial cross-sections and all three spacing scenarios of the downstream node, the MUDI configuration (Figures 7.20 and 7.21) showed no evidence of migration of delay. In addition, the SPUI configuration with a five-lane cross-section and 1.6 kilometer (one mile) spacing also showed no evidence of migration of delay to the downstream nodes.

Figure 7.20: Downstream Area Total Time for 50% Left Turns, w/out Frontage Roads, 5-lane Arterial, Varying Spacing Scenarios

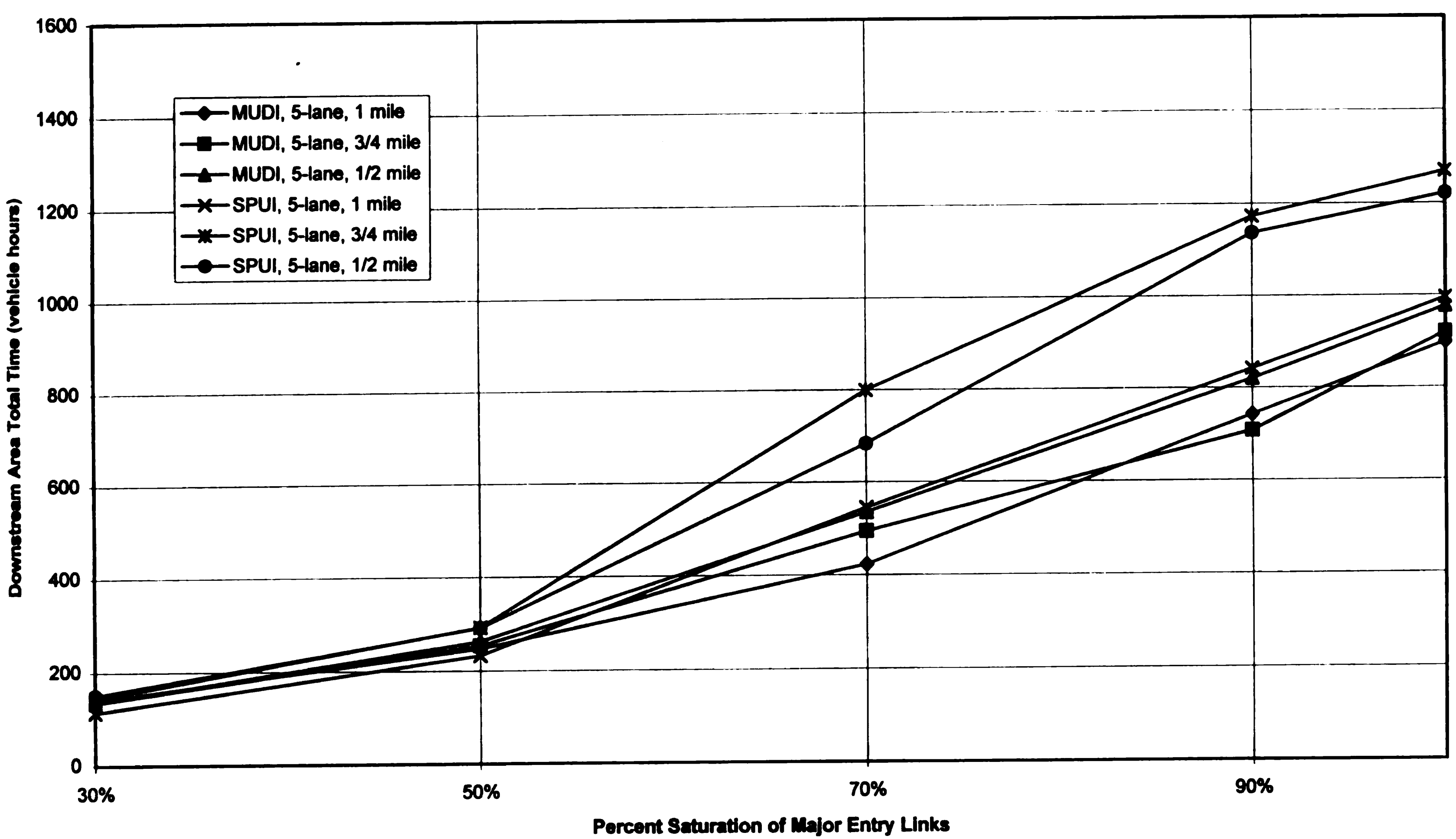
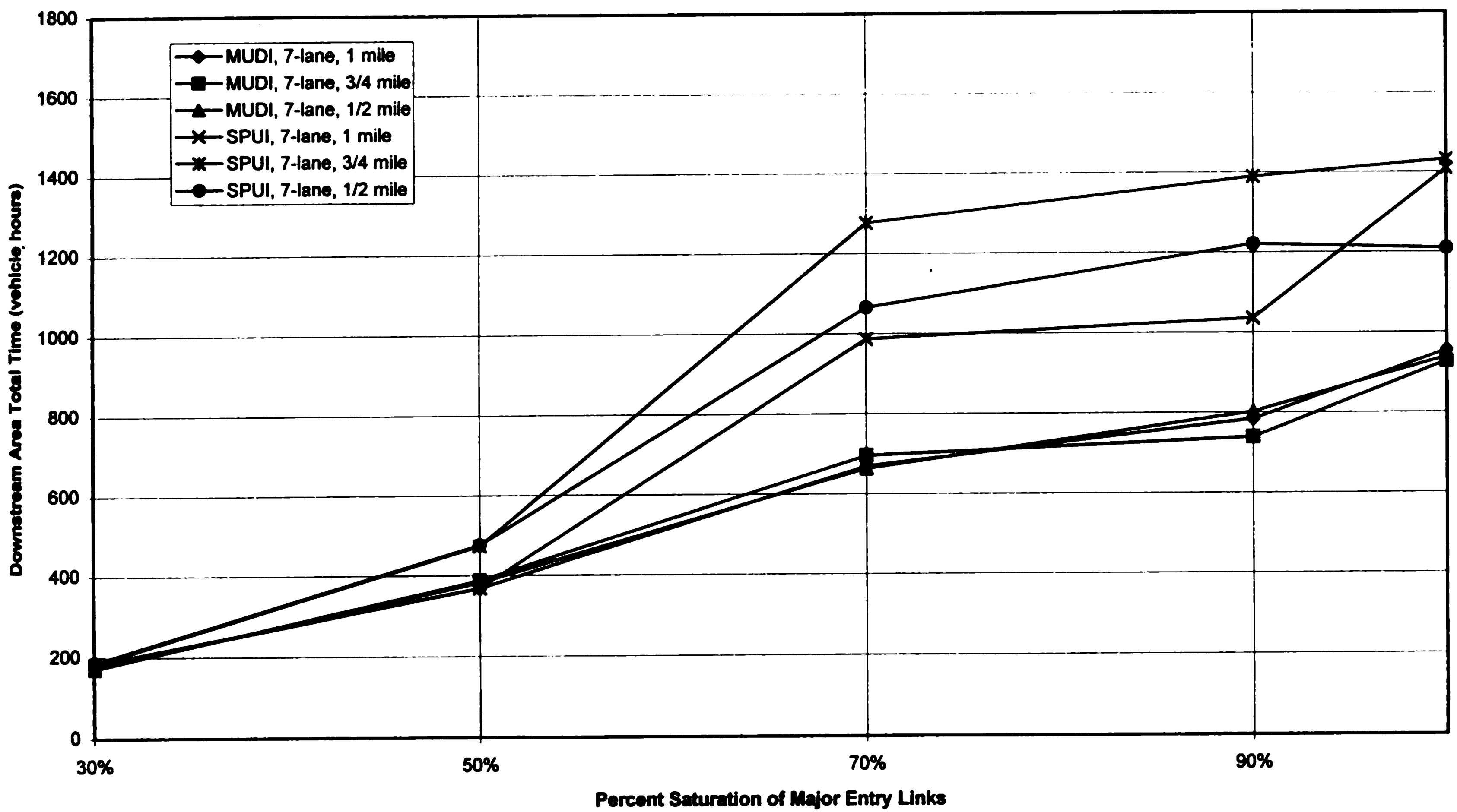


Figure 7.21: Downstream Area Total Time for 50% Left Turns, w/out Frontage Roads, 7-lane Arterial, Varying Spacing Scenarios



However, for levels of saturation of 50 percent or greater, all other SPUI configuration scenarios resulted in higher total times, suggesting a migration of delay.

Chapter 8

CONCLUSIONS

8.1 Conclusions from the State of the Practice and Field Review

Since no Single Point Urban Interchanges exist in Michigan, it was necessary to determine the state of the practice for SPUI design from other states. This was accomplished by conducting a literature review, AASHTO e-mail survey, telephone survey and field review. Special attention was paid to characteristics of the design which may affect urban interchange operation and design in Michigan.

Much of the published literature on SPUI design and operation was either generated from a study by Bonneson and Messer (3, 4, 5, 6, 7, and 14) at the Texas Transportation Institute (TTI) or referenced this effort. The authors identified a wide variation in the geometry and signal strategies of existing SPUI designs. The capacity analysis performed by the authors show that a SPUI utilizing a 3 phase signal is slightly more effective than a tight urban diamond interchange, but the advantage diminishes as the size of the SPUI becomes larger. The addition of a fourth phase to accommodate frontage roads resulted in a reduced capacity for the SPUI. Further, the authors stated that the typical SPUI signal phasing does not provide for a protected pedestrian phase to occur across the cross-road arterial.

Leisch, et. al. (13) raised the concern that the SPUI has little potential for expansion and modification. Merrit (15) stated additional concerns that drivers who are unfamiliar with the SPUI design may encounter some initial difficulty. Thus, the SPUI design needs to rely heavily on guide signing, pavement markings, and lane use signing for the necessary positive guidance to drivers. Merrit further stated that the proximity of nearby intersections is a concern.

A survey was submitted by e-mail to each of the other 49 state departments of transportation requesting information on the design and operation of SPUIs. Although the survey was as succinct as possible, only 14 state DOTs responded, of which only 4 had existing SPUIs. The responses received from different states varied widely. This, coupled with the limited number of responses, limited the usefulness of the survey information.

The review of the literature and the response to the e-mail survey, while helpful, had significant inconsistencies and lacked information in key areas. Thus, a telephone survey was conducted to collect more information and to locate the most appropriate sites for field review.

Based on the telephone survey, sites in Indiana, Illinois, Minnesota, Florida, Missouri and Arizona were chosen for field review. The observations made in the field review were grouped into the areas of geometric design, signal operation, pedestrian control and pavement markings.

The most significant difference in the geometric designs of SPUIs was between a SPUI with the cross-road going over the freeway and a SPUI with the freeway going over the cross-road. The SPUIs with the cross-road going over the freeway were found to look and operate more like a conventional signalized intersection. Because of this less driver confusion was observed. In addition, routing the freeway over the cross-road exposes the freeway and major traffic volume to preferential icing in cold weather climates. Another geometric design difference was related to the physical size of the interchange. SPUIs without dedicated U-turn lanes appeared to accommodate U-turns, for all but the largest of trucks, as well as those with dedicated U-turn lanes. The resulting increase in size to accommodate U-turn lanes may be counterproductive due to an increase in clearance times.

The signal operation strategy employed by each state differed significantly. Cycle lengths varied from 80 seconds to 180 seconds, with longer cycle lengths usually having fully actuated signal phases for all movements. The placement of traffic signal heads in designs where the cross-road went over the freeway resulted in the signal heads being located on a single overhead tubular beam.

The ability to accommodate pedestrians varied between designs. Typically, it was not difficult for pedestrians to move parallel to the cross-road and cross the ramp movements. However, it was difficult for pedestrians to cross the cross-road.

The need for pavement markings in large SPUIs is paramount. These pavement markings can overlap and cause driver confusion. This resulting driver confusion is more pronounced when the cross-road is skewed.

8.2 Conclusions from the Simulation Modeling

The literature review showed that while some evaluation of the SPUI had been done in the past, no comparative analysis had been published with regard to the ability to progress the arterial cross-road, compatibility with frontage roads, sensitivity to left-turning traffic, migration of delay, or traffic levels nearing capacity. Additionally, while the operational characteristics of a boulevard intersection have been studied and the results published, the Michigan Urban Diamond Interchange (MUDI) design, which is unique to Michigan, has never been formally studied. Thus, the SPUI and MUDI designs were computer modeled using TRAF-NETSIM to facilitate a comparison of their respective operational characteristics. Furthermore, a traditional diamond interchange was modeled to generate a frame of reference for the results.

An hour of operation for 300 individual modeling scenarios was simulated. The results of the simulation modeling are based on this finite number of scenarios defined by the four main variables addressed by this study: traffic volumes, turning percentages, frontage roads and distance to the closest downstream intersection. Only one size of interchange was modeled for each interchange configuration. Additionally, only one cycle length and one fixed, interchange area signal timing were coded for each interchange configuration with the cross-road arterial being modeled as a progressed-coordinated system.

Not all modeling scenarios that were simulated returned results that were valid. In a limited number of scenarios, a spillback of traffic on one of the model's entry links resulted in delay occurring outside the environment of the analysis.

The measures of effectiveness (MOEs) selected for this study were interchange area total time and downstream area total time, where "total time" is made up of both move time and delay time.

Based on the MOE interchange area total time, MUDI operation, in most situations, is superior to that of a SPUI and traditional diamond interchange configurations. This is true of scenarios modeled both with and without the presence of frontage roads. These operational advantages are most pronounced when the percentage of left-turning traffic is high and the level of saturation is high. In addition, the operational advantages of the SPUI are greatly reduced as the percentage of left-turning traffic is reduced, with the traditional diamond outperforming the SPUI at high levels of saturation and low levels of left-turning traffic.

The concern that greatly enhanced urban interchange configurations may demonstrate an improved operation at the freeway, but may merely move the delay to the first signalized intersection upstream or downstream was addressed. Based on the MOE downstream area total time, there was less migration of delay to downstream intersections with a MUDI configuration than with either a SPUI or traditional diamond configuration. For all scenarios without the presence of frontage roads, the traditional diamond interchange configuration resulted in moving delay to the downstream nodes. While there was no evidence that the SPUI configuration resulted in moving delay to the downstream nodes when modeled with a five-lane arterial cross-road, when modeled with a seven-lane cross-road, the SPUI configuration shows this effect at high levels of saturation. Both the SPUI and the traditional diamond show this effect when modeled with the presence of frontage roads.

The affect that the proximity of the closest downstream node has on either the MUDI or SPUI interchange operation was also studied for scenarios without the presence of frontage roads. Three spacing scenarios were considered: 1.6 kilometers (one mile) which allows for perfect progression along the arterial while maintaining adequate separation between the intersection and interchange area; 1.2 kilometers (three-quarter mile) which does not allow perfect progression along the arterial, but still maintains adequate separation between the intersection and interchange area; and, 0.8 kilometers (one-half mile) which allows for perfect progression along the arterial, but the proximity of the intersection to the interchange area may affect operation.

Based on the MOEs interchange area total time and downstream area total time, MUDI operation, in most situations, is insensitive to the proximity of the closest downstream node, while the SPUI operation is sensitive to the proximity of the closest downstream node. In all cases, the performance of the MUDI configuration with a separation of 1.6 kilometers (one mile) or 1.2 kilometers (three-quarter mile) either equals or exceeds the operational performance of the SPUI. Furthermore, at higher saturation levels, the operational performance of the SPUI configuration was adversely affected by a separation of 0.8 kilometers (one-half mile). For both arterial cross-sections (five-lane and seven-lane) and all three spacing scenarios of the downstream node, the MUDI configuration showed no evidence of migration of delay. In addition, the SPUI configuration with a five-lane cross-section and 1.6 kilometer (one mile) spacing also showed no evidence of migration of delay to the downstream nodes. However, for higher levels of saturation, all other SPUI configuration scenarios resulted in higher total times, suggesting a migration of delay.

APPENDICES

APPENDIX A

MUDI w/out Frontage Roads Major Crossroad = 5-lane Distance to Closest Intersection = 1 mile							
Interchange Area Major Flow		Major Entry Link		Minor Entry Link Volume	Ramp Volume	Interchange Area Total Time	Downstream Area Total Time
% Left Turns	% Right Turns	% Sat	Volume		70%/30%	(veh. hours)	(veh. hours)
70	30	0.3	1080	463	540/231	81	139
70	30	0.5	1800	771	900/386	152	264
70	30	0.7	2520	1080	1260/540	302	530
70	30	0.9	3240	1388	1620/694	381	645
70	30	1.0	3600	1543	1800/771	417	1030
50	50	0.3	1080	463	540/231	80	139
50	50	0.5	1800	771	900/386	149	247
50	50	0.7	2520	1080	1260/540	277	423
50	50	0.9	3240	1388	1620/694	327	742
50	50	1.0	3600	1543	1800/771	368	899
30	70	0.3	1080	463	540/231	79	139
30	70	0.5	1800	771	900/386	146	263
30	70	0.7	2520	1080	1260/540	243	507
30	70	0.9	3240	1388	1620/694	344	662
30	70	1.0	3600	1543	1800/771	331	912

Table A.1: Simulation Results for Modeling Scenarios Involving the Michigan Urban Diamond Interchange (MUDI), without Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial.

MUDI w/out Frontage Roads Major Crossroad = 7-lane Distance to Closest Intersection = 1 mile							
Interchange Area Major Flow		Major Entry Links		Minor Entry Links Volume	Ramp Volume	Interchange Area Total Time (veh. hours)	Downstream Area Total Time (veh. hours)
% Left Turns	% Right Turns	% Sat	Volume		70%/30%		
70	30	0.3	1080	463	540/231	102	180
70	30	0.5	1800	771	900/386	195	373
70	30	0.7	2520	1080	1260/540	382	599
70	30	0.9	3240	1388	1620/694	463	780
70	30	1.0	3600	1543	1800/771	478	940
50	50	0.3	1080	463	540/231	102	181
50	50	0.5	1800	771	900/386	189	368
50	50	0.7	2520	1080	1260/540	342	669
50	50	0.9	3240	1388	1620/694	391	784
50	50	1.0	3600	1543	1800/771	415	955
30	70	0.3	1080	463	540/231	100	180
30	70	0.5	1800	771	900/386	185	390
30	70	0.7	2520	1080	1260/540	379	686
30	70	0.9	3240	1388	1620/694	416	754
30	70	1.0	3600	1543	1800/771	464	957

Table A.2: Simulation Results for Modeling Scenarios Involving the Michigan Urban Diamond Interchange (MUDI), without Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial.

SPUI w/out Frontage Roads Major Crossroad = 5-lane Distance to Closest Intersection = 1 mile							
Interchange Area Major Flow		Major Entry Links		Minor Entry Links Volume	Ramp Volume	Interchange Area Total Time	Downstream Area Total Time
% Left Turns	% Right Turns	% Sat	Volume		70%/30%	(veh. hours)	(veh. hours)
70	30	0.3	1080	463	540/231	81	113
70	30	0.5	1800	771	900/386	148	226
70	30	0.7	2520	1080	1260/540	402	578
70	30	0.9	3240	1388	1620/694	451	839
70	30	1.0	3600	1543	1800/771	459	968
50	50	0.3	1080	463	540/231	79	114
50	50	0.5	1800	771	900/386	144	232
50	50	0.7	2520	1080	1260/540	336	545
50	50	0.9	3240	1388	1620/694	423	842
50	50	1.0	3600	1543	1800/771	445	995
30	70	0.3	1080	463	540/231	76	114
30	70	0.5	1800	771	900/386	140	234
30	70	0.7	2520	1080	1260/540	336	582
30	70	0.9	3240	1388	1620/694	449	854
30	70	1.0	3600	1543	1800/771	458	932

Table A.3: Simulation Results for Modeling Scenarios Involving the Single Point Urban Interchange (SPUI), without Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial.

SPUI w/out Frontage Roads Major Crossroad = 7-lane Distance to Closest Intersection = 1 mile							
Interchange Area Major Flow		Major Entry Links		Minor Entry Links	Ramp	Interchange Area	Downstream Area
% Left Turns	% Right Turns	% Sat	Volume	Volume	Volume 70%/30%	Total Time (veh. hours)	Total Time (veh. hours)
70	30	0.3	1080	463	540/231	105	184
70	30	0.5	1800	771	900/386	285	370
70	30	0.7	2520	1080	1260/540	498	998
70	30	0.9	3240	1388	1620/694	625	1122
70	30	1.0	3600	1543	1800/771	646	1394
50	50	0.3	1080	463	540/231	104	183
50	50	0.5	1800	771	900/386	287	369
50	50	0.7	2520	1080	1260/540	447	987
50	50	0.9	3240	1388	1620/694	532	1036
50	50	1.0	3600	1543	1800/771	609	1409
30	70	0.3	1080	463	540/231	101	149
30	70	0.5	1800	771	900/386	282	379
30	70	0.7	2520	1080	1260/540	443	1146
30	70	0.9	3240	1388	1620/694	524	1149
30	70	1.0	3600	1543	1800/771	599	1148

Table A.4: Simulation Results for Modeling Scenarios Involving the Single Point Urban Interchange (SPUI), without Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial.

DIAMOND w/out Frontage Roads Major Crossroad = 5-lane Distance to Closest Intersection = 1 mile							
Interchange Area Major Flow		Major Entry Links		Minor Entry Links	Ramp Volume	Interchange Area Total Time	Downstream Area Total Time
% Left Turns	% Right Turns	% Sat	Volume	Volume	70%/30%	(veh. hours)	(veh. hours)
70	30	0.3	1080	463	540/231	87	143
70	30	0.5	1800	771	900/386	382	393
70	30	0.7	2520	1080	1260/540	469	895
70	30	0.9	3240	1388	1620/694	478	1015
70	30	1.0	3600	1543	1800/771	531	1245
50	50	0.3	1080	463	540/231	84	142
50	50	0.5	1800	771	900/386	296	399
50	50	0.7	2520	1080	1260/540	447	881
50	50	0.9	3240	1388	1620/694	481	1117
50	50	1.0	3600	1543	1800/771	484	1115
30	70	0.3	1080	463	540/231	79	143
30	70	0.5	1800	771	900/386	289	422
30	70	0.7	2520	1080	1260/540	336	887
30	70	0.9	3240	1388	1620/694	383	1147
30	70	1.0	3600	1543	1800/771	456	1219

Table A.5: Simulation Results for Modeling Scenarios Involving the Traditional Diamond Interchange, without Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial.

DIAMOND w/out Frontage Roads Major Crossroad = 7-lane Distance to Closest Intersection = 1 mile							
Interchange Area Major Flow		Major Entry Links		Minor Entry Links	Ramp	Interchange Area	Downstream Area
% Left Turns	% Right Turns	% Sat	Volume	Volume	Volume 70%/30%	Total Time (veh. hours)	Total Time (veh. hours)
70	30	0.3	1080	463	540/231	163	182
70	30	0.5	1800	771	900/386	365	621
70	30	0.7	2520	1080	1260/540	555	1117
70	30	0.9	3240	1388	1620/694	647	1368
70	30	1.0	3600	1543	1800/771	680	1420
50	50	0.3	1080	463	540/231	159	182
50	50	0.5	1800	771	900/386	375	688
50	50	0.7	2520	1080	1260/540	430	1080
50	50	0.9	3240	1388	1620/694	593	1448
50	50	1.0	3600	1543	1800/771	579	1411
30	70	0.3	1080	463	540/231	157	182
30	70	0.5	1800	771	900/386	363	663
30	70	0.7	2520	1080	1260/540	408	1071
30	70	0.9	3240	1388	1620/694	457	1383
30	70	1.0	3600	1543	1800/771	464	1393

Table A.6: Simulation Results for Modeling Scenarios Involving the Traditional Diamond Interchange, without Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial.

MUDI w/ Frontage Roads Major Crossroad = 5-lane Distance to Closest Intersection = 1 mile								
Interchange Area Major Flow		Service Drive Volume	Major Entry Links		Minor Entry Links Volume	Ramp Volume	Interchange Area Total Time	Downstream Area Total Time
% Left Turns	% Right Turns	70%/30%	% Sat	Volume		70%/30%	(veh. hours)	(veh. hours)
70	30	180/77	0.3	1080	463	540/231	90	84
70	30	300/129	0.5	1800	771	900/386	182	162
70	30	420/180	0.7	2520	1080	1260/540	377	338
70	30	540/231	0.9	3240	1388	1620/694	438	433
70	30	600/257	1.0	3600	1543	1800/771	553	486
50	50	180/77	0.3	1080	463	540/231	89	86
50	50	300/129	0.5	1800	771	900/386	164	164
50	50	420/180	0.7	2520	1080	1260/540	313	347
50	50	540/231	0.9	3240	1388	1620/694	403	422
50	50	600/257	1.0	3600	1543	1800/771	386	533
30	70	180/77	0.3	1080	463	540/231	88	87
30	70	300/129	0.5	1800	771	900/386	161	168
30	70	420/180	0.7	2520	1080	1260/540	302	375
30	70	540/231	0.9	3240	1388	1620/694	373	379
30	70	600/257	1.0	3600	1543	1800/771	405	908

Table A.7: Simulation Results for Modeling Scenarios Involving the Michigan Urban Diamond Interchange (MUDI), with Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial.

MUDI w/ Frontage Roads Major Crossroad = 7-lane Distance to Closest Intersection = 1 mile								
Interchange Area Major Flow		Service Drive Volume	Major Entry Links		Minor Entry Links Volume	Ramp Volume	Interchange Area Total Time	Downstream Area Total Time
% Left Turns	% Right Turns	70%/30%	% Sat	Volume		70%/30%	(veh. hours)	(veh. hours)
70	30	180/77	0.3	1080	463	540/231	111	109
70	30	300/129	0.5	1800	771	900/386	280	243
70	30	420/180	0.7	2520	1080	1260/540	545	364
70	30	540/231	0.9	3240	1388	1620/694	342	271
70	30	600/257	1.0	3600	1543	1800/771	259	200
50	50	180/77	0.3	1080	463	540/231	109	110
50	50	300/129	0.5	1800	771	900/386	204	255
50	50	420/180	0.7	2520	1080	1260/540	411	404
50	50	540/231	0.9	3240	1388	1620/694	541	492
50	50	600/257	1.0	3600	1543	1800/771	527	978
30	70	180/77	0.3	1080	463	540/231	107	113
30	70	300/129	0.5	1800	771	900/386	198	266
30	70	420/180	0.7	2520	1080	1260/540	371	404
30	70	540/231	0.9	3240	1388	1620/694	487	435
30	70	600/257	1.0	3600	1543	1800/771	506	575

Table A.8: Simulation Results for Modeling Scenarios Involving the Michigan Urban Diamond Interchange (MUDI), with Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial.

SPUI w/ Frontage Roads Major Crossroad = 5-lane Distance to Closest Intersection = 1 mile								
Interchange Area Major Flow		Service Drive Volume	Major Entry Links		Minor Entry Links Volume	Ramp Volume	Interchange Area Total Time	Downstream Area Total Time
% Left Turns	% Right Turns	70%/30%	% Sat	Volume		70%/30%	(veh. hours)	(veh. hours)
70	30	180/77	0.3	1080	463	540/231	96	147
70	30	300/129	0.5	1800	771	900/386	230	283
70	30	420/180	0.7	2520	1080	1260/540	458	774
70	30	540/231	0.9	3240	1388	1620/694	503	1097
70	30	600/257	1.0	3600	1543	1800/771	516	1387
50	50	180/77	0.3	1080	463	540/231	93	147
50	50	300/129	0.5	1800	771	900/386	186	288
50	50	420/180	0.7	2520	1080	1260/540	398	803
50	50	540/231	0.9	3240	1388	1620/694	526	1132
50	50	600/257	1.0	3600	1543	1800/771	555	1347
30	70	180/77	0.3	1080	463	540/231	89	141
30	70	300/129	0.5	1800	771	900/386	214	275
30	70	420/180	0.7	2520	1080	1260/540	383	759
30	70	540/231	0.9	3240	1388	1620/694	516	1043
30	70	600/257	1.0	3600	1543	1800/771	538	1243

Table A.9: Simulation Results for Modeling Scenarios Involving the Single Point Urban Interchange (SPUI), with Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial.

SPUI w/ Frontage Roads Major Crossroad = 7-lane Distance to Closest Intersection = 1 mile								
Interchange Area Major Flow		Service Drive Volume	Major Entry Links		Minor Entry Links Volume	Ramp Volume	Interchange Area Total Time	Downstream Area Total Time
% Left Turns	% Right Turns	70%/30%	% Sat	Volume		70%/30%	(veh. hours)	(veh. hours)
70	30	180/77	0.3	1080	463	540/231	120	188
70	30	300/129	0.5	1800	771	900/386	370	419
70	30	420/180	0.7	2520	1080	1260/540	605	1017
70	30	540/231	0.9	3240	1388	1620/694	668	1389
70	30	600/257	1.0	3600	1543	1800/771	691	1324
50	50	180/77	0.3	1080	463	540/231	118	189
50	50	300/129	0.5	1800	771	900/386	353	419
50	50	420/180	0.7	2520	1080	1260/540	498	1070
50	50	540/231	0.9	3240	1388	1620/694	695	1428
50	50	600/257	1.0	3600	1543	1800/771	730	1317
30	70	180/77	0.3	1080	463	540/231	114	188
30	70	300/129	0.5	1800	771	900/386	324	398
30	70	420/180	0.7	2520	1080	1260/540	478	975
30	70	540/231	0.9	3240	1388	1620/694	673	1292
30	70	600/257	1.0	3600	1543	1800/771	696	1397

Table A.10: Simulation Results for Modeling Scenarios Involving the Single Point Urban Interchange (SPUI), with Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial.

DIAMOND w/ Frontage Roads Major Crossroad = 5-lane Distance to Closest Intersection = 1 mile								
Interchange Area Major Flow		Service Drive Volume	Major Entry Links		Minor Entry Links Volume	Ramp Volume	Interchange Area Total Time	Downstream Area Total Time
% Left Turns	% Right Turns	70%/30%	% Sat	Volume		70%/30%	(veh. hours)	(veh. hours)
70	30	180/77	0.3	1080	463	540/231	109	149
70	30	300/129	0.5	1800	771	900/386	440	401
70	30	420/180	0.7	2520	1080	1260/540	528	879
70	30	540/231	0.9	3240	1388	1620/694	654	1129
70	30	600/257	1.0	3600	1543	1800/771	715	1223
50	50	180/77	0.3	1080	463	540/231	91	148
50	50	300/129	0.5	1800	771	900/386	400	429
50	50	420/180	0.7	2520	1080	1260/540	508	895
50	50	540/231	0.9	3240	1388	1620/694	539	1147
50	50	600/257	1.0	3600	1543	1800/771	569	1197
30	70	180/77	0.3	1080	463	540/231	85	148
30	70	300/129	0.5	1800	771	900/386	298	428
30	70	420/180	0.7	2520	1080	1260/540	370	911
30	70	540/231	0.9	3240	1388	1620/694	511	1201
30	70	600/257	1.0	3600	1543	1800/771	520	1254

Table A.11: Simulation Results for Modeling Scenarios Involving the Traditional Diamond Interchange, with Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial.

DIAMOND w/ Frontage Roads Major Crossroad = 7-lane Distance to Closest Intersection = 1 mile								
Interchange Area Major Flow		Service Drive Volume	Major Entry Links		Minor Entry Links Volume	Ramp Volume	Interchange Area Total Time	Downstream Area Total Time
% Left Turns	% Right Turns	70%/30%	% Sat	Volume		70%/30%	(veh. hours)	(veh. hours)
70	30	180/77	0.3	1080	463	540/231	173	173
70	30	300/129	0.5	1800	771	900/386	484	616
,70	30	420/180	0.7	2520	1080	1260/540	574	1154
70	30	540/231	0.9	3240	1388	1620/694	673	1330
70	30	600/257	1.0	3600	1543	1800/771	736	1321
50	50	180/77	0.3	1080	463	540/231	168	188
50	50	300/129	0.5	1800	771	900/386	361	602
50	50	420/180	0.7	2520	1080	1260/540	556	1145
50	50	540/231	0.9	3240	1388	1620/694	672	1349
50	50	600/257	1.0	3600	1543	1800/771	669	1399
30	70	180/77	0.3	1080	463	540/231	159	188
30	70	300/129	0.5	1800	771	900/386	362	619
30	70	420/180	0.7	2520	1080	1260/540	438	1129
30	70	540/231	0.9	3240	1388	1620/694	505	1293
30	70	600/257	1.0	3600	1543	1800/771	538	1241

Table A.12: Simulation Results for Modeling Scenarios Involving the Traditional Diamond Interchange, with Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial.

MUDI w/out Frontage Roads Major Crossroad = 5-lane Distance to Closest Intersection = 3/4 mile							
Interchange Area Major Flow		Major Entry Links		Minor Entry Links	Ramp	Interchange Area Total Time	Downstream Area Total Time
% Left Turns	% Right Turns	% Sat	Volume	Volume	Volume 70%/30%	(veh. hours)	(veh. hours)
70	30	0.3	1080	463	540/231	85	133
70	30	0.5	1800	771	900/386	150	256
70	30	0.7	2520	1080	1260/540	280	492
70	30	0.9	3240	1388	1620/694	374	667
70	30	1.0	3600	1543	1800/771	505	980
50	50	0.3	1080	463	540/231	83	134
50	50	0.5	1800	771	900/386	147	253
50	50	0.7	2520	1080	1260/540	271	494
50	50	0.9	3240	1388	1620/694	331	707
50	50	1.0	3600	1543	1800/771	366	921
30	70	0.3	1080	463	540/231	82	137
30	70	0.5	1800	771	900/386	144	262
30	70	0.7	2520	1080	1260/540	263	614
30	70	0.9	3240	1388	1620/694	320	698
30	70	1.0	3600	1543	1800/771	364	947

Table A.13: Simulation Results for Modeling Scenarios Involving the Michigan Urban Diamond Interchange (MUDI), without Frontage Roads, 1.2 kilometers (3/4 mile), 5-lane Arterial.

MUDI w/out Frontage Roads Major Crossroad = 7-lane Distance to Closest Intersection = 3/4 mile							
Interchange Area Major Flow		Major Entry Links		Minor Entry Links Volume	Ramp Volume	Interchange Area Total Time	Downstream Area Total Time
% Left Turns	% Right Turns	% Sat	Volume		70%/30%	(veh. hours)	(veh. hours)
70	30	0.3	1080	463	540/231	107	170
70	30	0.5	1800	771	900/386	201	377
70	30	0.7	2520	1080	1260/540	372	563
70	30	0.9	3240	1388	1620/694	442	774
70	30	1.0	3600	1543	1800/771	486	941
50	50	0.3	1080	463	540/231	105	172
50	50	0.5	1800	771	900/386	189	388
50	50	0.7	2520	1080	1260/540	338	696
50	50	0.9	3240	1388	1620/694	399	739
50	50	1.0	3600	1543	1800/771	408	927
30	70	0.3	1080	463	540/231	104	175
30	70	0.5	1800	771	900/386	187	362
30	70	0.7	2520	1080	1260/540	359	725
30	70	0.9	3240	1388	1620/694	414	773
30	70	1.0	3600	1543	1800/771	416	934

Table A.14: Simulation Results for Modeling Scenarios Involving the Michigan Urban Diamond Interchange (MUDI), without Frontage Roads, 1.2 kilometers (3/4 mile), 7-lane Arterial.

SPUI w/out Frontage Roads Major Crossroad = 5-lane Distance to Closest Intersection = 3/4 mile							
Interchange Area Major Flow		Major Entry Links		Minor Entry Links	Ramp	Interchange Area	Downstream Area
% Left Turns	% Right Turns	% Sat	Volume	Volume	Volume 70%/30%	Total Time (veh. hours)	Total Time (veh. hours)
70	30	0.3	1080	463	540/231	84	143
70	30	0.5	1800	771	900/386	236	292
70	30	0.7	2520	1080	1260/540	422	806
70	30	0.9	3240	1388	1620/694	448	1101
70	30	1.0	3600	1543	1800/771	460	1242
50	50	0.3	1080	463	540/231	82	143
50	50	0.5	1800	771	900/386	228	292
50	50	0.7	2520	1080	1260/540	354	799
50	50	0.9	3240	1388	1620/694	419	1173
50	50	1.0	3600	1543	1800/771	439	1270
30	70	0.3	1080	463	540/231	80	145
30	70	0.5	1800	771	900/386	177	270
30	70	0.7	2520	1080	1260/540	355	838
30	70	0.9	3240	1388	1620/694	446	1128
30	70	1.0	3600	1543	1800/771	463	1283

Table A.15: Simulation Results for Modeling Scenarios Involving the Single Point Urban Interchange (SPUI), without Frontage Roads, 1.2 kilometers (3/4 mile), 5-lane Arterial.

SPUI w/out Frontage Roads Major Crossroad = 7-lane Distance to Closest Intersection = 3/4 mile							
Interchange Area Major Flow		Major Entry Links		Minor Entry Links Volume	Ramp Volume 70%/30%	Interchange Area Total Time (veh. hours)	Downstream Area Total Time (veh. hours)
% Left Turns	% Right Turns	% Sat	Volume				
70	30	0.3	1080	463	540/231	112	187
70	30	0.5	1800	771	900/386	382	451
70	30	0.7	2520	1080	1260/540	539	1225
70	30	0.9	3240	1388	1620/694	653	1576
70	30	1.0	3600	1543	1800/771	665	1703
50	50	0.3	1080	463	540/231	105	183
50	50	0.5	1800	771	900/386	386	473
50	50	0.7	2520	1080	1260/540	488	1277
50	50	0.9	3240	1388	1620/694	554	1389
50	50	1.0	3600	1543	1800/771	627	1432
30	70	0.3	1080	463	540/231	106	185
30	70	0.5	1800	771	900/386	372	487
30	70	0.7	2520	1080	1260/540	476	1171
30	70	0.9	3240	1388	1620/694	585	1286
30	70	1.0	3600	1543	1800/771	649	1385

Table A.16: Simulation Results for Modeling Scenarios Involving the Single Point Urban Interchange (SPUI), without Frontage Roads, 1.2 kilometers (3/4 mile), 7-lane Arterial.

MUDI w/out Frontage Roads Major Crossroad = 5-lane Distance to Closest Intersection = 1/2 mile							
Interchange Area Major Flow		Major Entry Links		Minor Entry Links Volume	Ramp Volume	Interchange Area Total Time	Downstream Area Total Time
% Left Turns	% Right Turns	% Sat	Volume		70%/30%	(veh. hours)	(veh. hours)
70	30	0.3	1080	463	540/231	82	131
70	30	0.5	1800	771	900/386	152	256
70	30	0.7	2520	1080	1260/540	489	554
70	30	0.9	3240	1388	1620/694	585	652
70	30	1.0	3600	1543	1800/771	369	904
50	50	0.3	1080	463	540/231	81	135
50	50	0.5	1800	771	900/386	146	262
50	50	0.7	2520	1080	1260/540	414	535
50	50	0.9	3240	1388	1620/694	579	820
50	50	1.0	3600	1543	1800/771	537	977
30	70	0.3	1080	463	540/231	79	139
30	70	0.5	1800	771	900/386	143	271
30	70	0.7	2520	1080	1260/540	270	534
30	70	0.9	3240	1388	1620/694	294	746
30	70	1.0	3600	1543	1800/771	409	1011

Table A.17: Simulation Results for Modeling Scenarios Involving the Michigan Urban Diamond Interchange (MUDI), without Frontage Roads, 0.8 kilometers (1/2 mile), 5-lane Arterial.

MUDI w/out Frontage Roads Major Crossroad = 7-lane Distance to Closest Intersection = 1/2 mile							
Interchange Area Major Flow % Left Turns	Interchange Area Major Flow % Right Turns	Major Entry Links % Sat	Major Entry Links Volume	Minor Entry Links Volume	Ramp Volume 70%/30%	Interchange Area Total Time (veh. hours)	Downstream Area Total Time (veh. hours)
70	30	0.3	1080	463	540/231	103	171
70	30	0.5	1800	771	900/386	193	466
70	30	0.7	2520	1080	1260/540	422	548
70	30	0.9	3240	1388	1620/694	478	810
70	30	1.0	3600	1543	1800/771	483	877
50	50	0.3	1080	463	540/231	101	174
50	50	0.5	1800	771	900/386	186	382
50	50	0.7	2520	1080	1260/540	507	664
50	50	0.9	3240	1388	1620/694	416	802
50	50	1.0	3600	1543	1800/771	433	938
30	70	0.3	1080	463	540/231	100	180
30	70	0.5	1800	771	900/386	184	365
30	70	0.7	2520	1080	1260/540	350	645
30	70	0.9	3240	1388	1620/694	538	888
30	70	1.0	3600	1543	1800/771	412	959

Table A.18: Simulation Results for Modeling Scenarios Involving the Michigan Urban Diamond Interchange (MUDI), without Frontage Roads, 0.8 kilometers (1/2 mile), 7-lane Arterial.

SPUI w/out Frontage Roads Major Crossroad = 5-lane Distance to Closest Intersection = 1/2 mile							
Interchange Area Major Flow		Major Entry Links		Minor Entry Links Volume	Ramp Volume 70%/30%	Interchange Area Total Time (veh. hours)	Downstream Area Total Time (veh. hours)
% Left Turns	% Right Turns	% Sat	Volume				
70	30	0.3	1080	463	540/231	82	143
70	30	0.5	1800	771	900/386	222	282
70	30	0.7	2520	1080	1260/540	471	796
70	30	0.9	3240	1388	1620/694	551	1107
70	30	1.0	3600	1543	1800/771	516	1243
50	50	0.3	1080	463	540/231	81	150
50	50	0.5	1800	771	900/386	223	291
50	50	0.7	2520	1080	1260/540	357	683
50	50	0.9	3240	1388	1620/694	512	1137
50	50	1.0	3600	1543	1800/771	540	1221
30	70	0.3	1080	463	540/231	79	146
30	70	0.5	1800	771	900/386	185	274
30	70	0.7	2520	1080	1260/540	426	800
30	70	0.9	3240	1388	1620/694	540	1115
30	70	1.0	3600	1543	1800/771	509	1372

Table A.19: Simulation Results for Modeling Scenarios Involving the Single Point Urban Interchange (SPUI), without Frontage Roads, 0.8 kilometers (1/2 mile), 5-lane Arterial.

SPUI w/out Frontage Roads Major Crossroad = 7-lane Distance to Closest Intersection = 1/2 mile							
Interchange Area Major Flow		Major Entry Links		Minor Entry Links	Ramp	Interchange Area	Downstream Area
% Left Turns	% Right Turns	% Sat	Volume	Volume	Volume 70%/30%	Total Time (veh. hours)	Total Time (veh. hours)
70	30	0.3	1080	463	540/231	110	182
70	30	0.5	1800	771	900/386	364	417
70	30	0.7	2520	1080	1260/540	580	1050
70	30	0.9	3240	1388	1620/694	518	1099
70	30	1.0	3600	1543	1800/771	741	1328
50	50	0.3	1080	463	540/231	107	186
50	50	0.5	1800	771	900/386	395	476
50	50	0.7	2520	1080	1260/540	452	1066
50	50	0.9	3240	1388	1620/694	684	1221
50	50	1.0	3600	1543	1800/771	650	1210
30	70	0.3	1080	463	540/231	105	185
30	70	0.5	1800	771	900/386	371	454
30	70	0.7	2520	1080	1260/540	476	1234
30	70	0.9	3240	1388	1620/694	505	1038
30	70	1.0	3600	1543	1800/771	573	1107

Table A.20: Simulation Results for Modeling Scenarios Involving the Single Point Urban Interchange (SPUI), without Frontage Roads, 0.8 kilometers (1/2 mile), 7-lane Arterial.

APPENDIX B

Figure B.1: Interchange Area Total Time For 70% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial

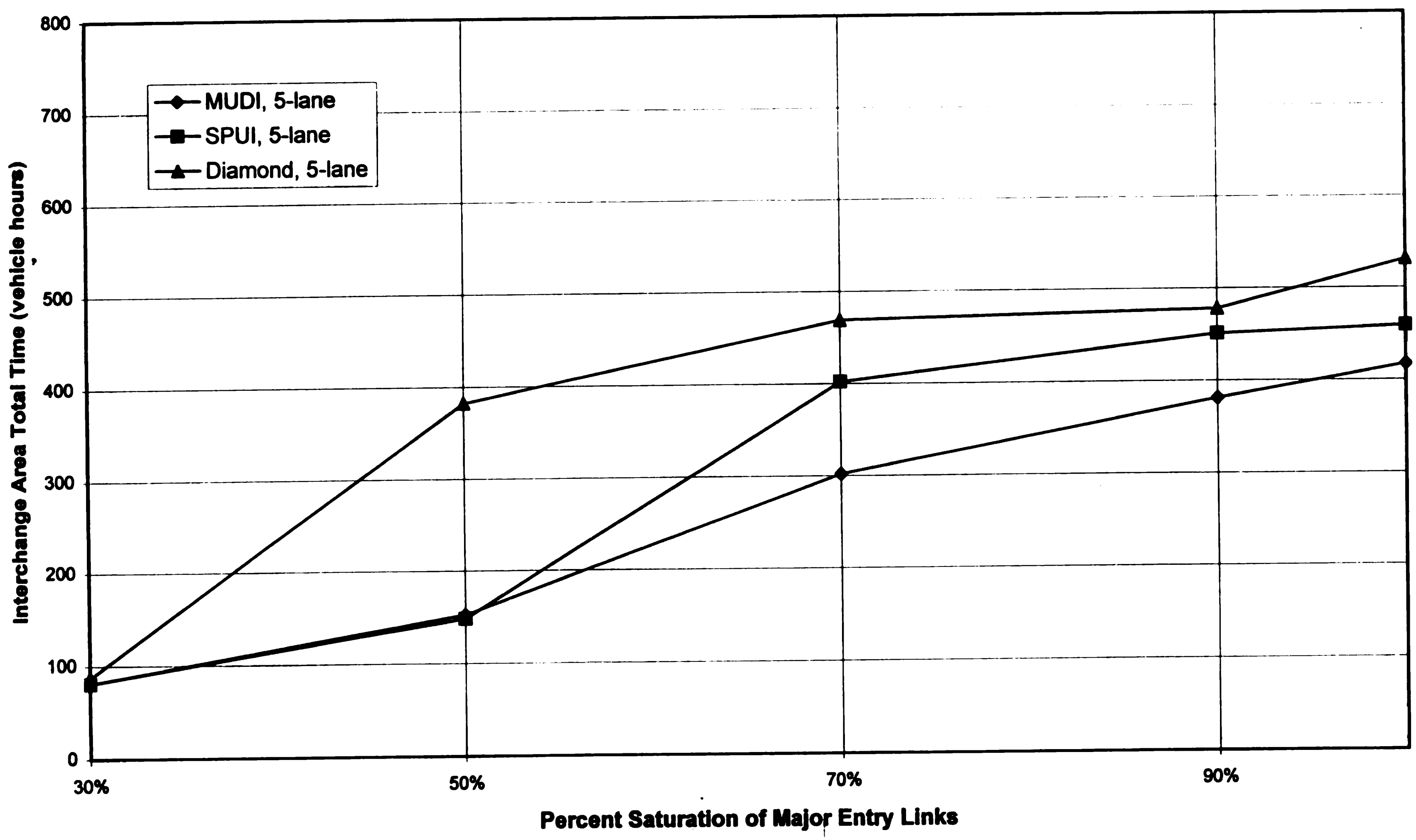


Figure B.2: Interchange Area Total Time For 50% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial

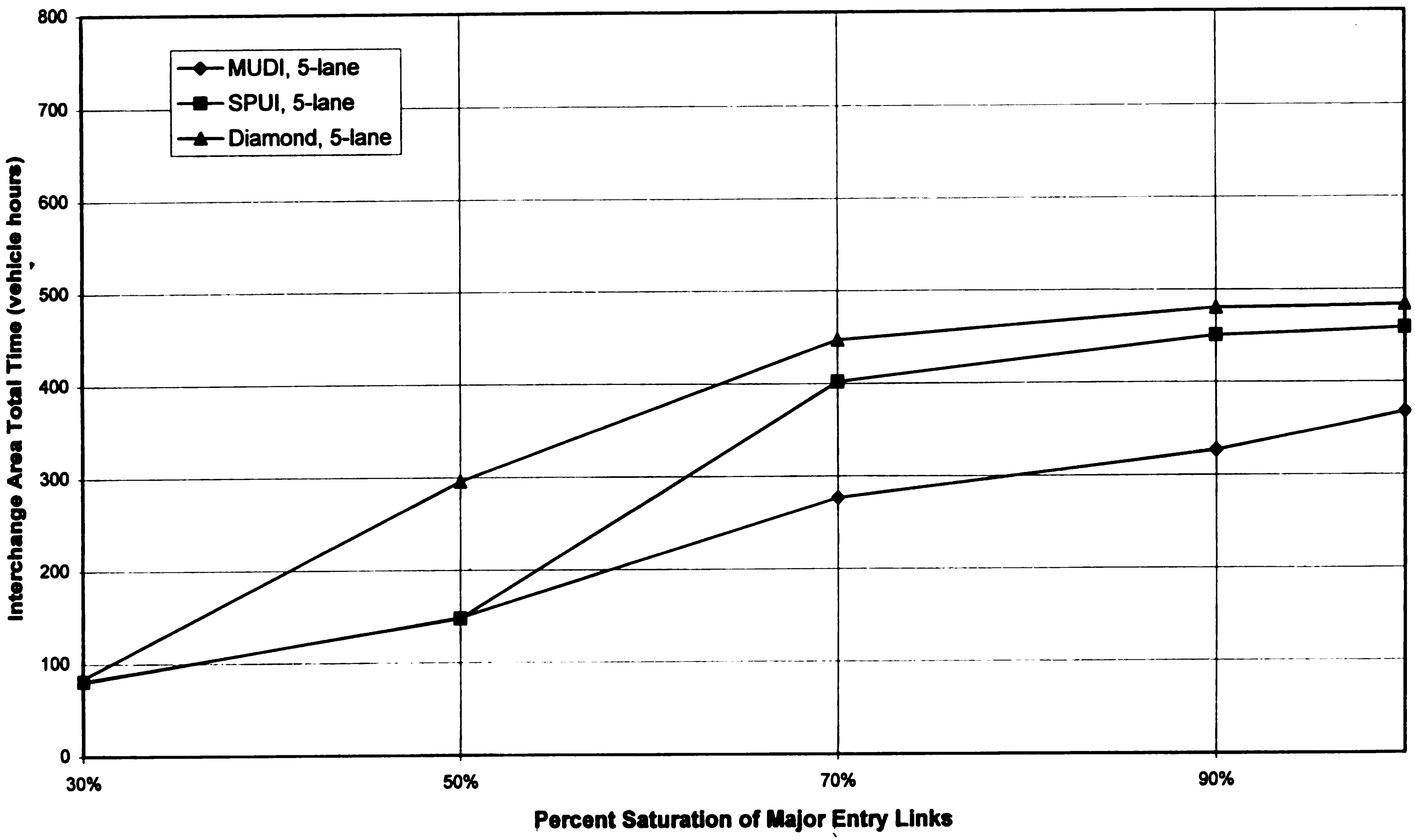


Figure B.3: Interchange Area Total Time For 30% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial

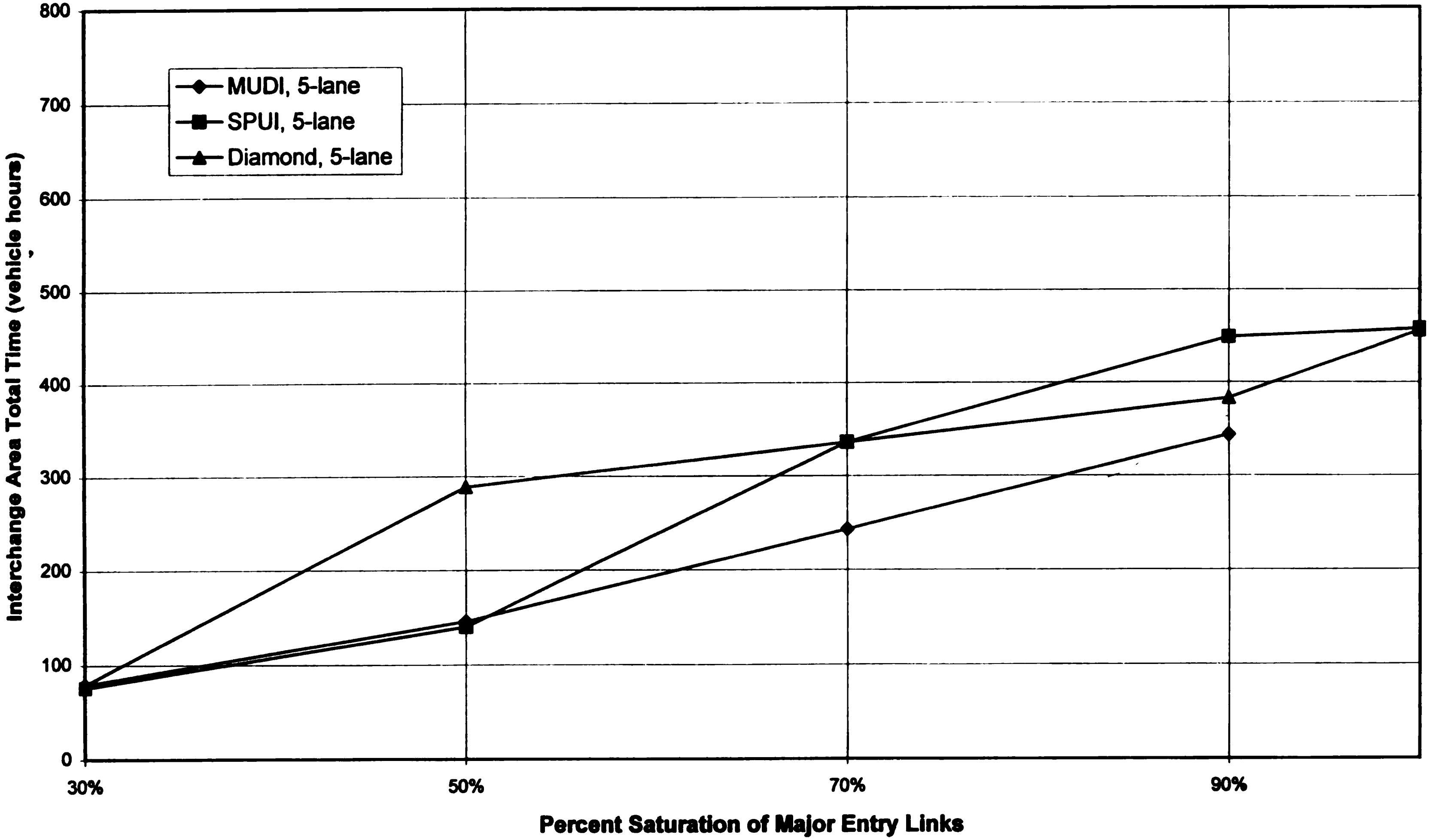


Figure B.4: Interchange Area Total Time For 70% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial

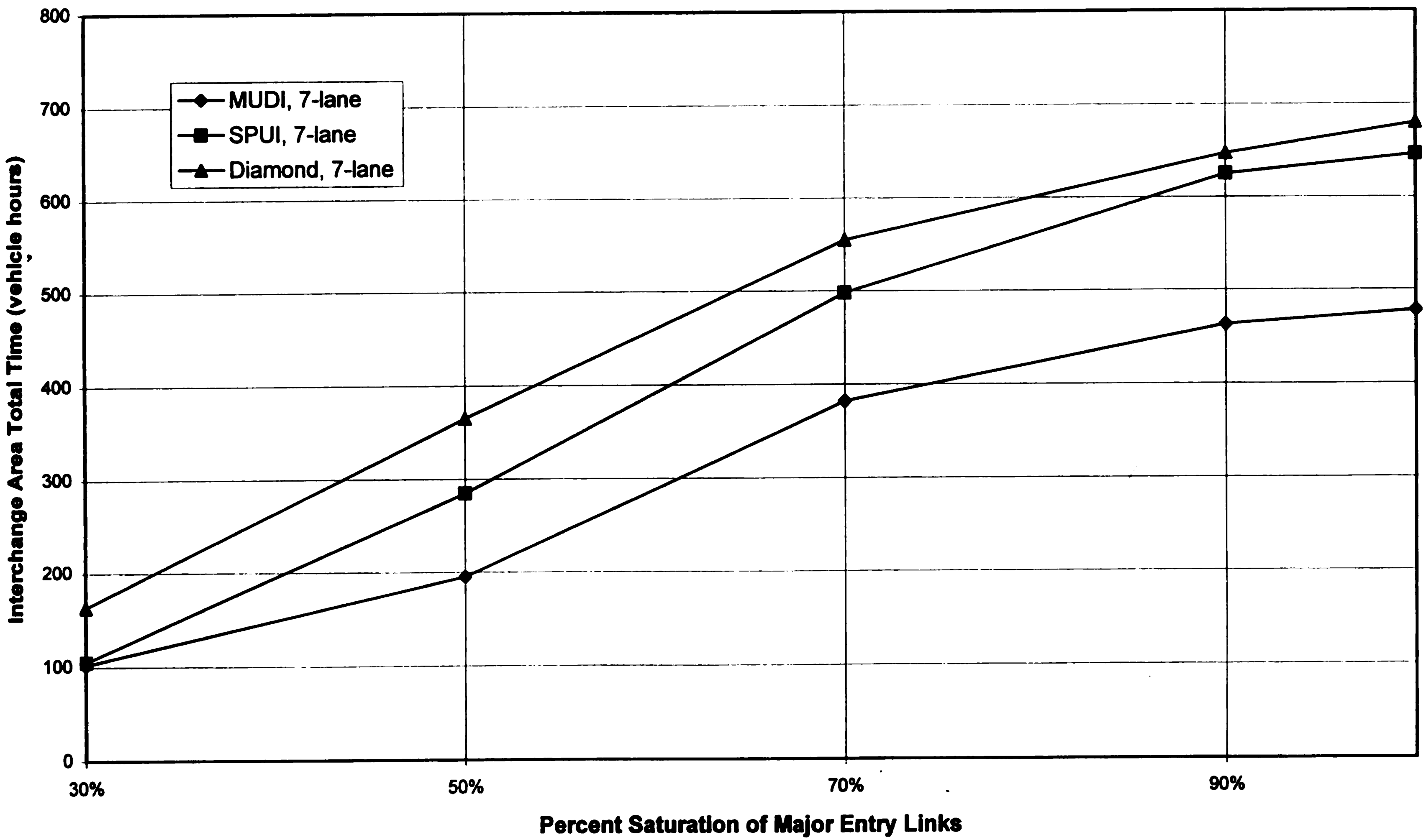


Figure B.5: Interchange Area Total Time For 50% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial

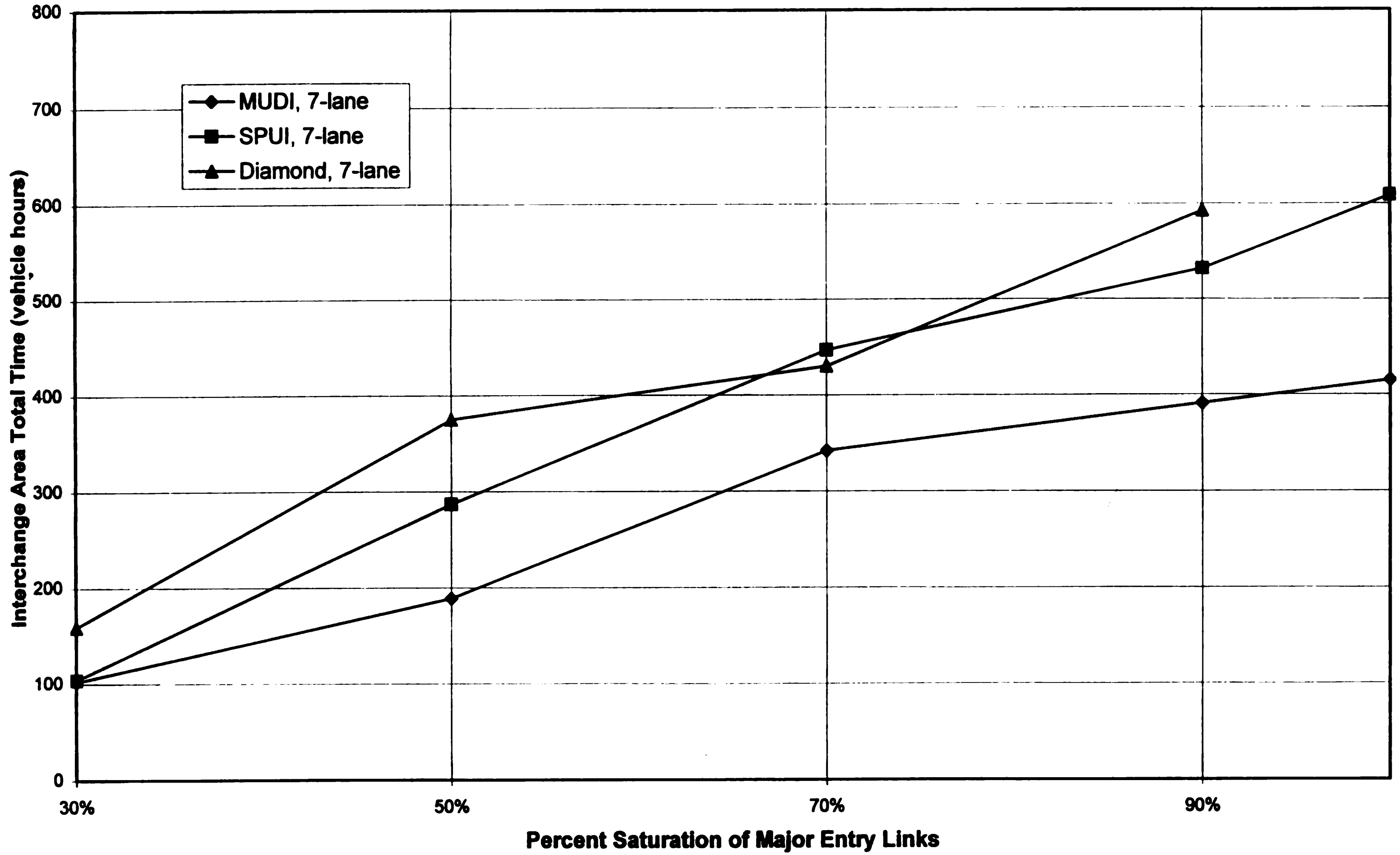


Figure B.6: Interchange Area Total Time For 30% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial

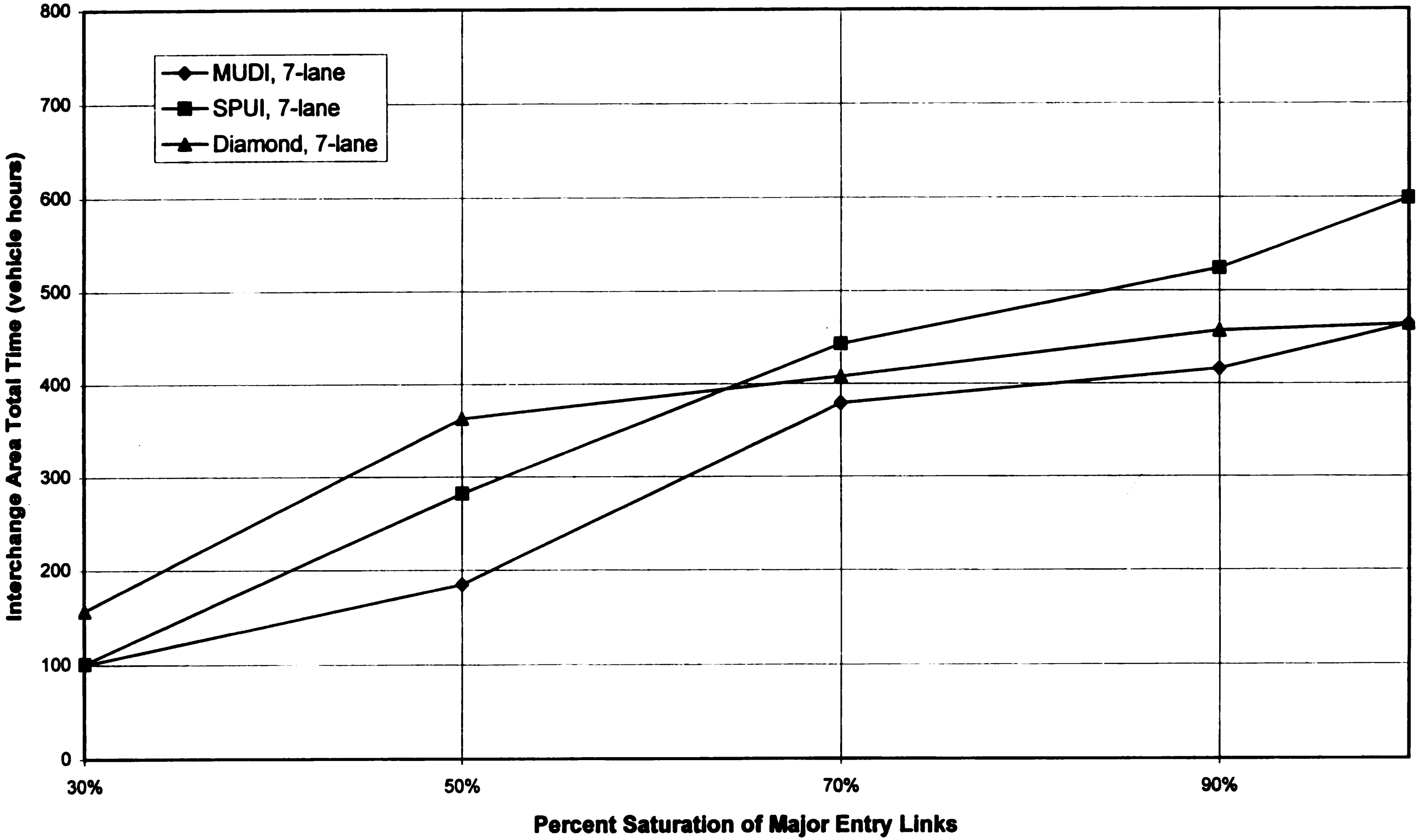


Figure B.7: Downstream Area Total Time For 70% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial

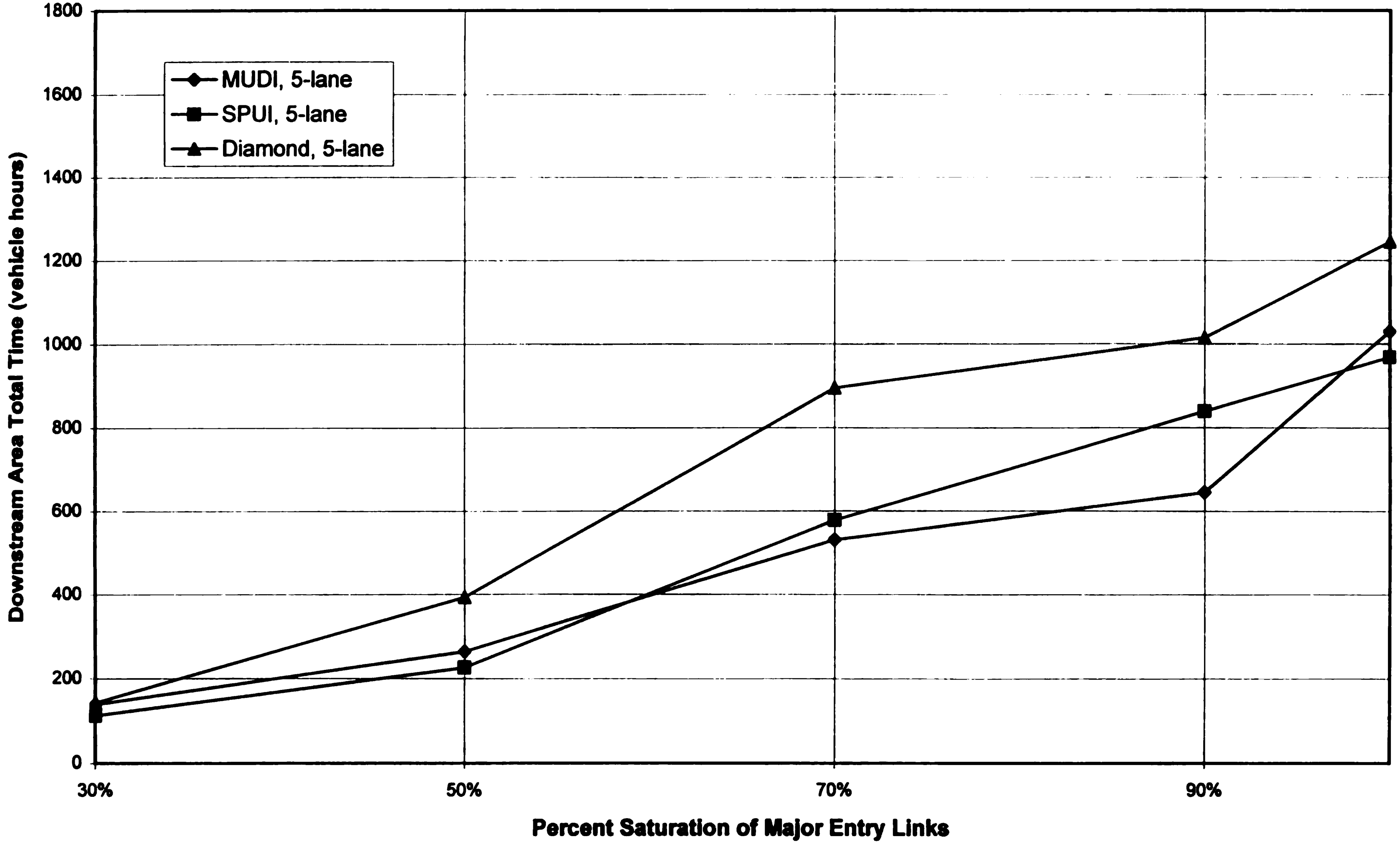


Figure B.8: Downstream Area Total Time For 50% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial

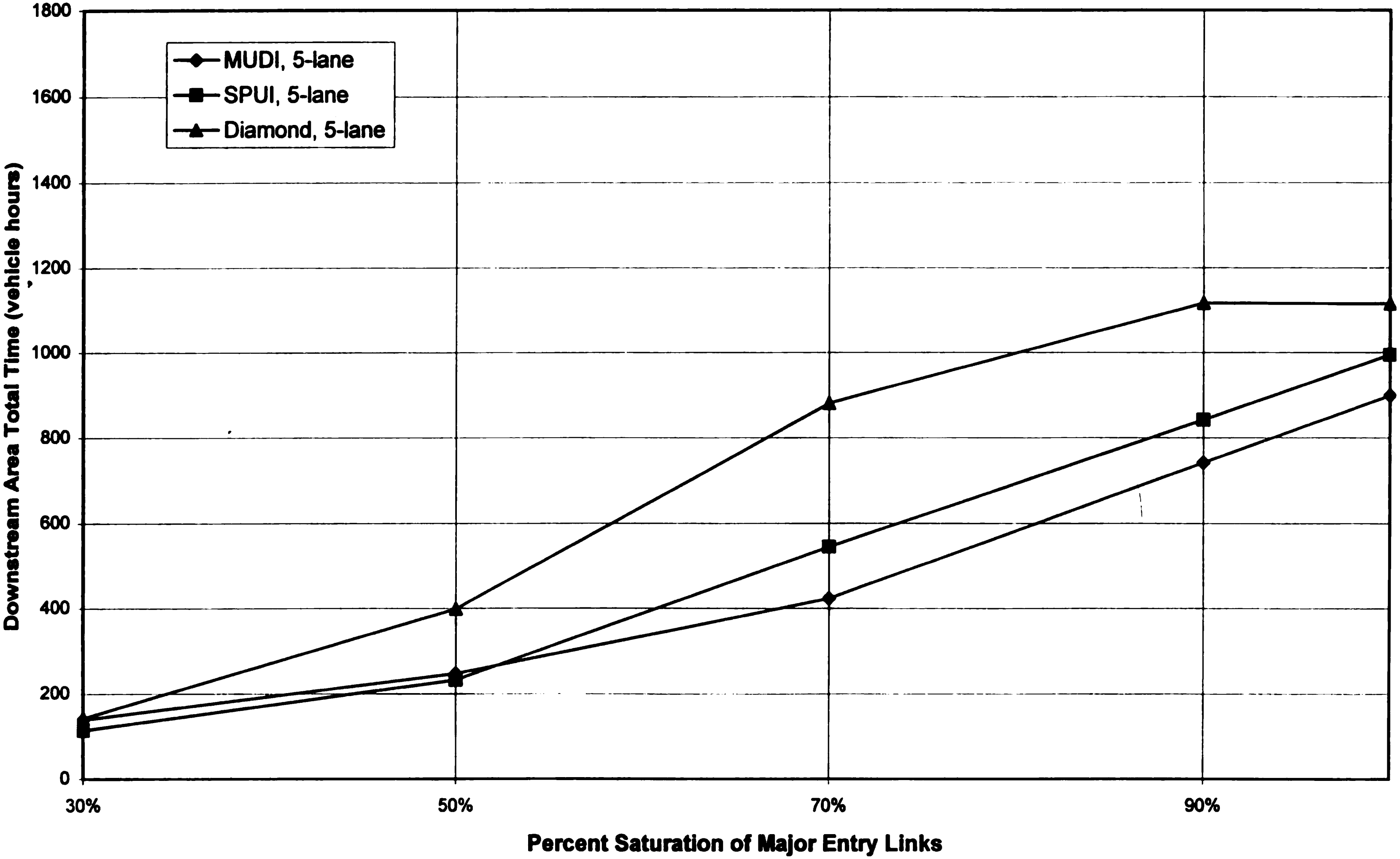


Figure B.9: Downstream Area Total Time For 30% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial

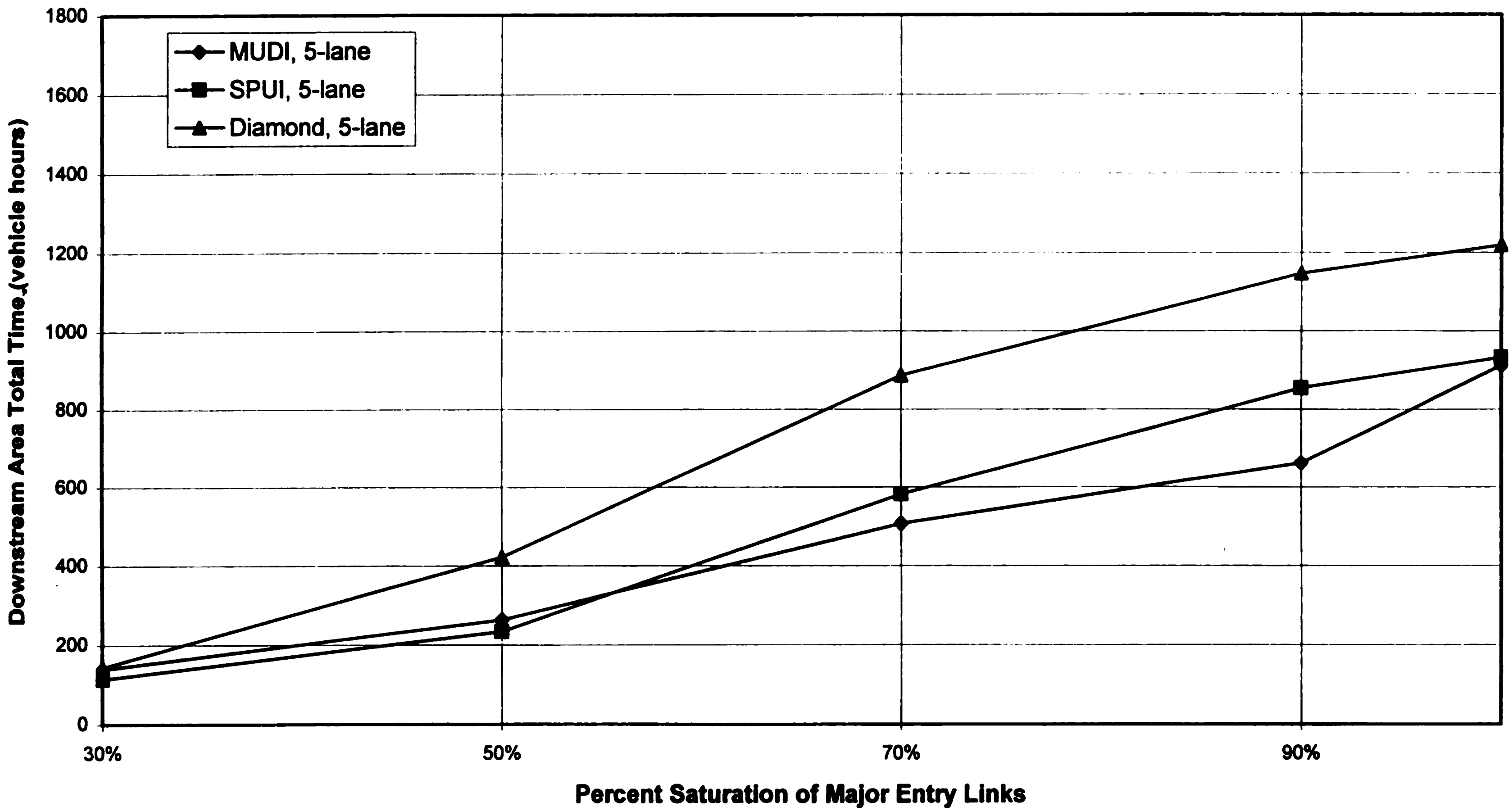


Figure B.10: Downstream Area Total Time For 70% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial

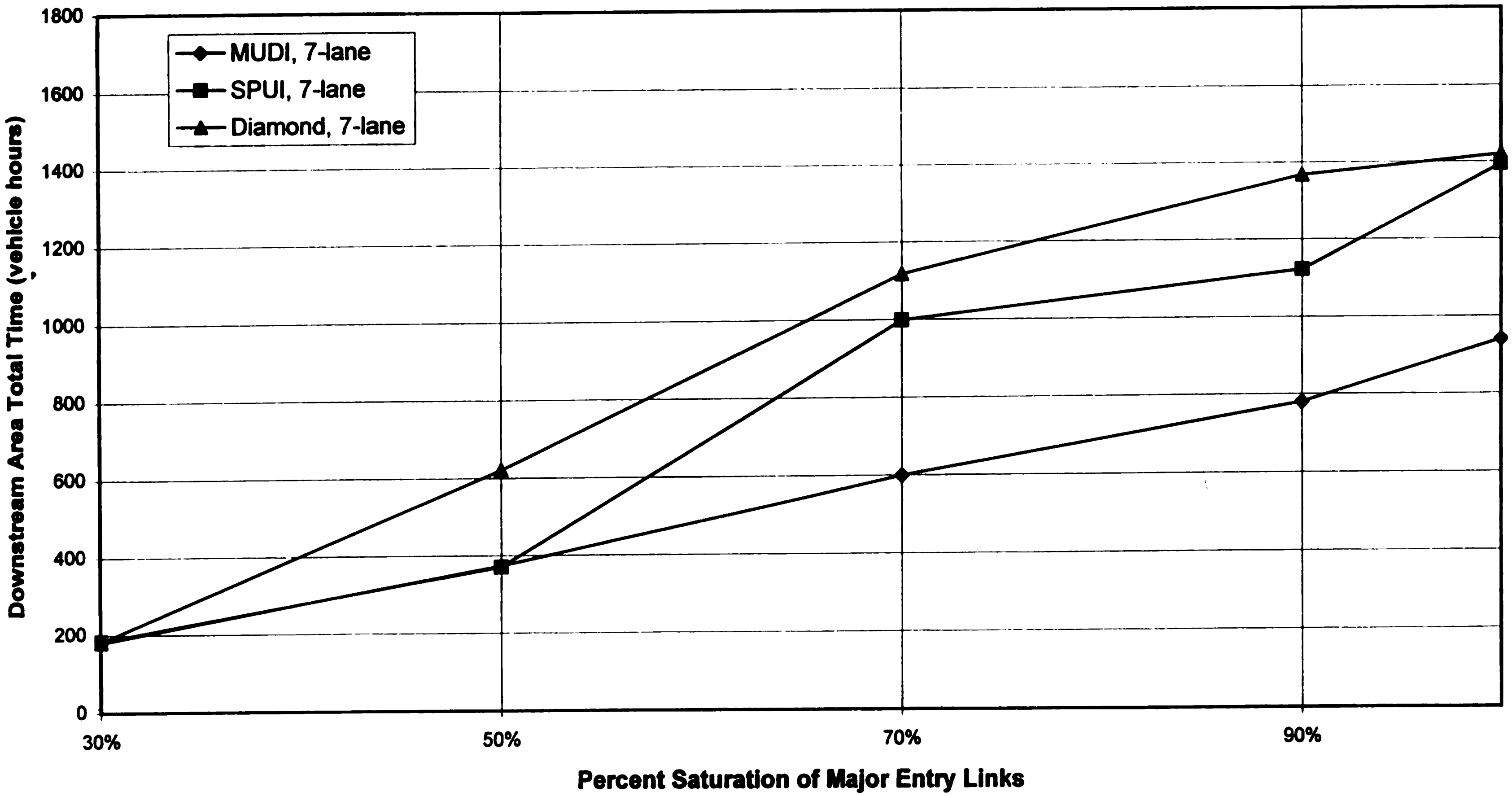


Figure B.11: Downstream Area Total Time For 50% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial

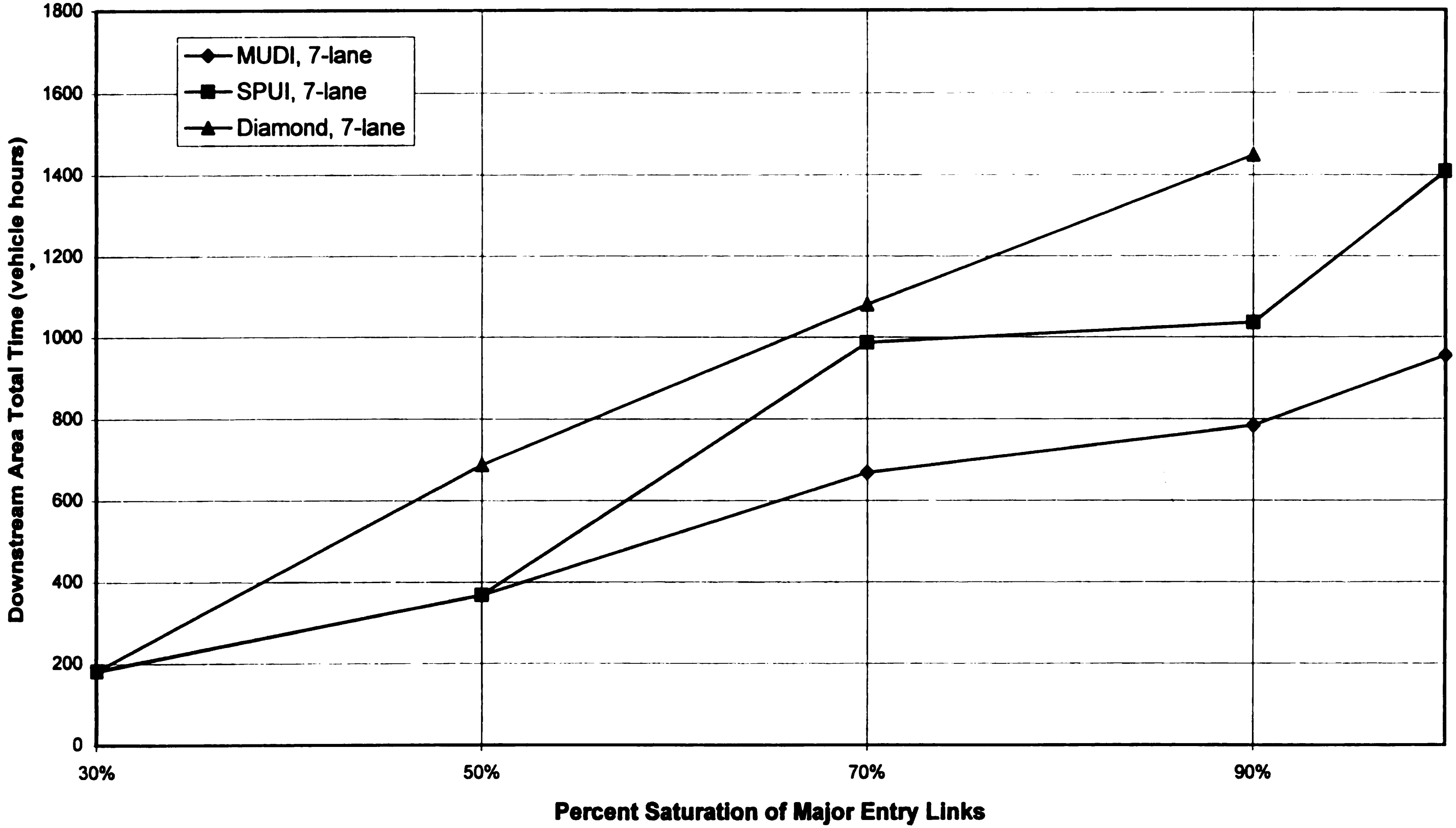


Figure B.12: Downstream Area Total Time For 30% Left Turns, w/out Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial

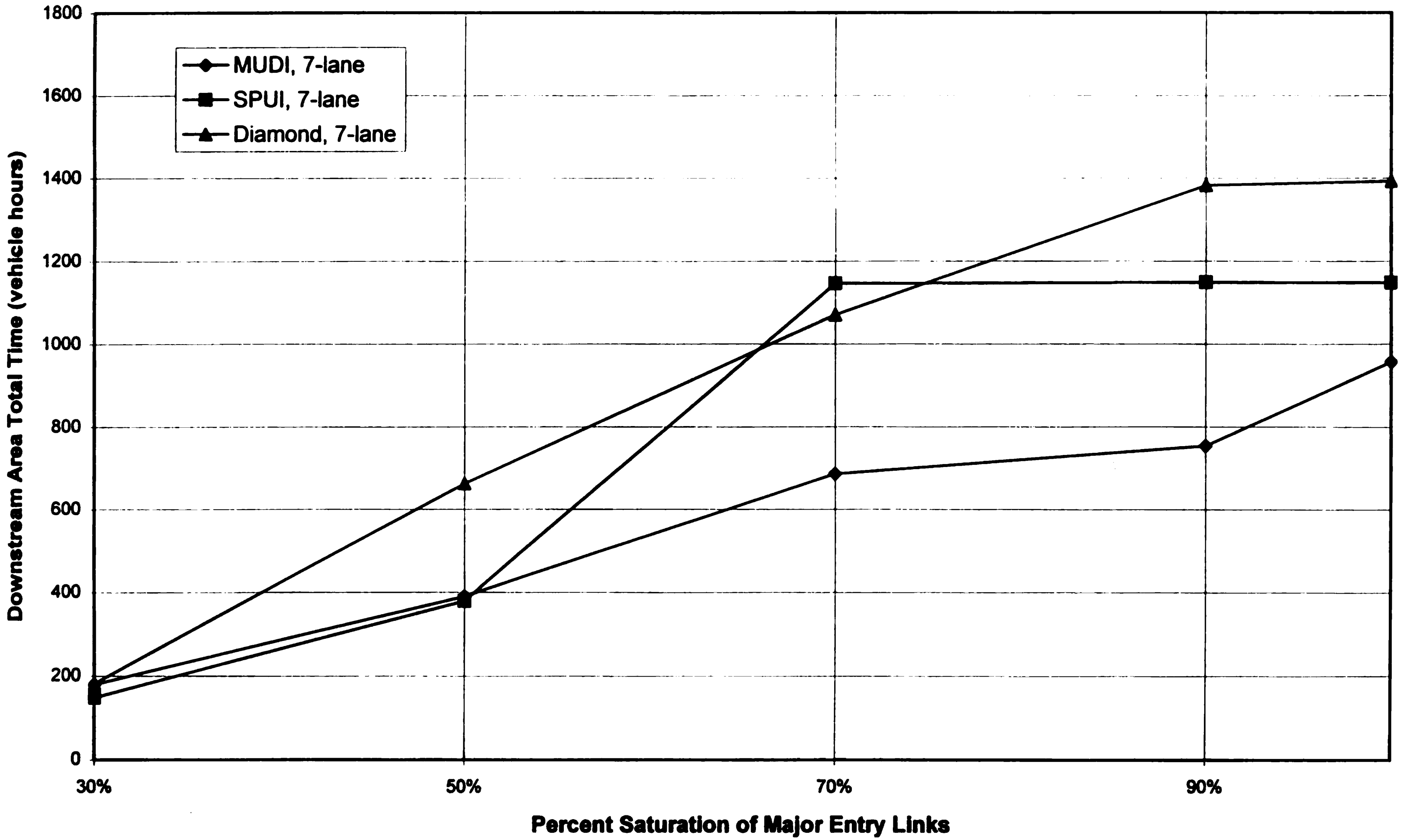


Figure B.13: Interchange Area Total Time For 70% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial

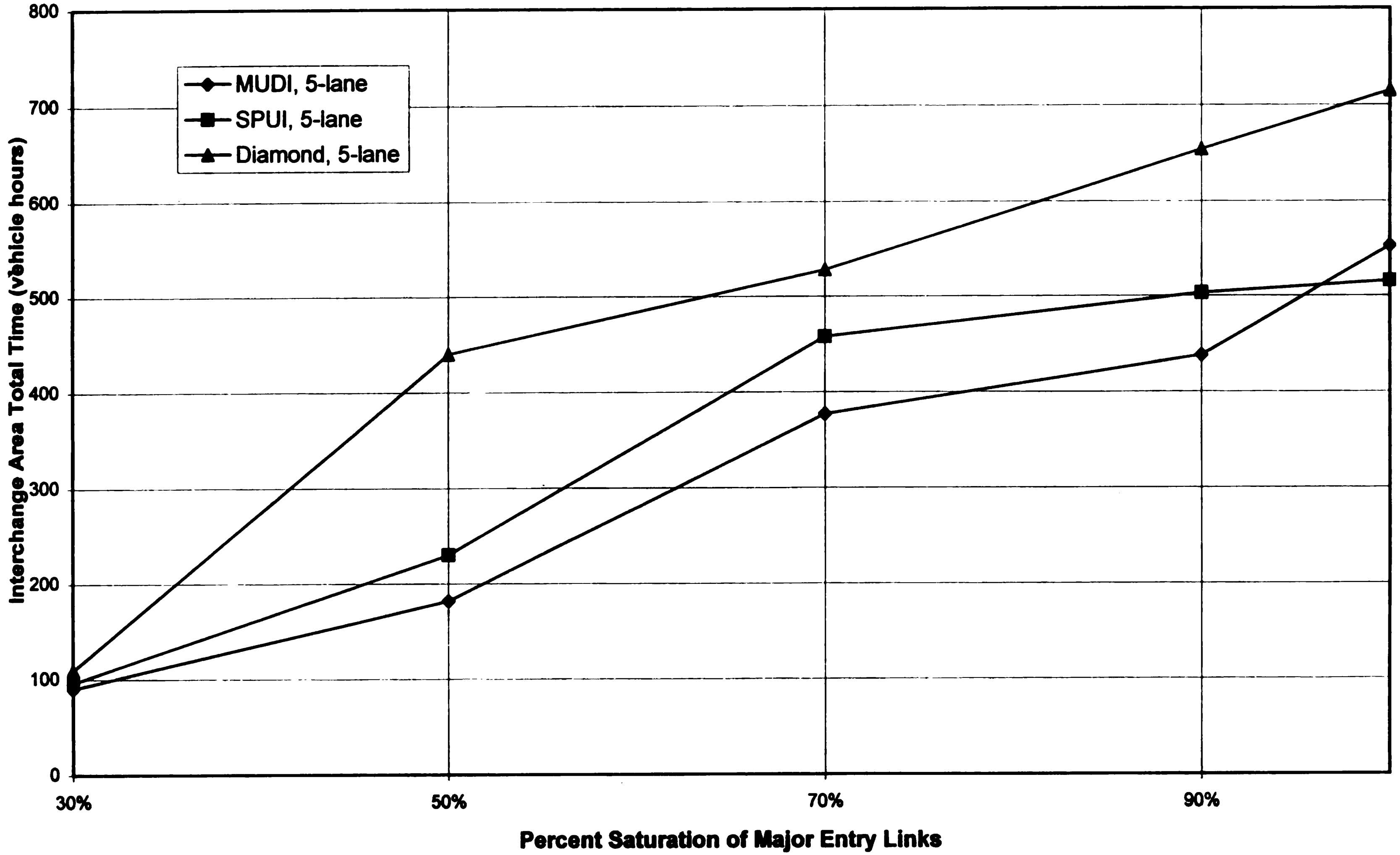


Figure B.14: Interchange Area Total Time For 50% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial

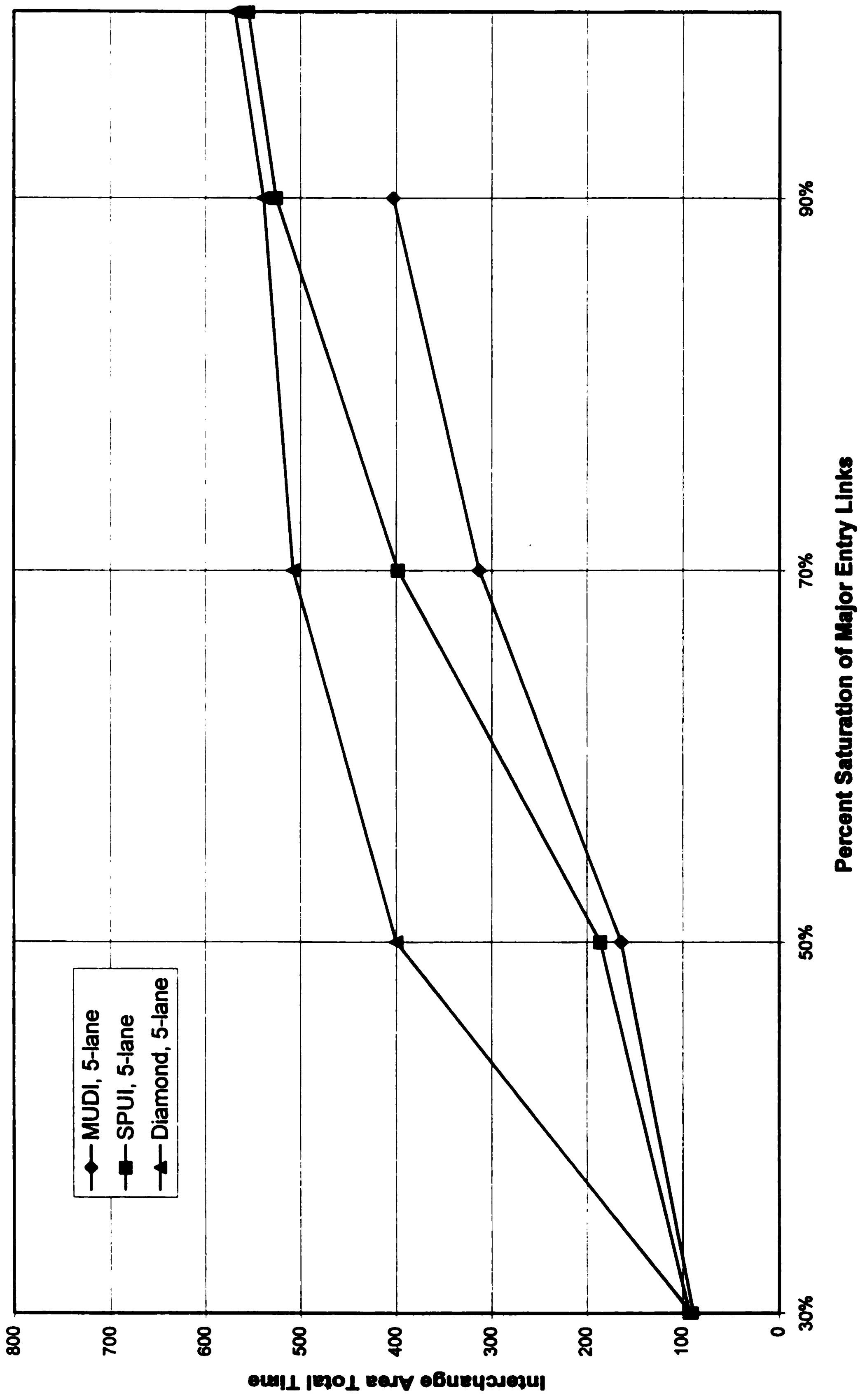


Figure B.15: Interchange Area Total Time For 30% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial

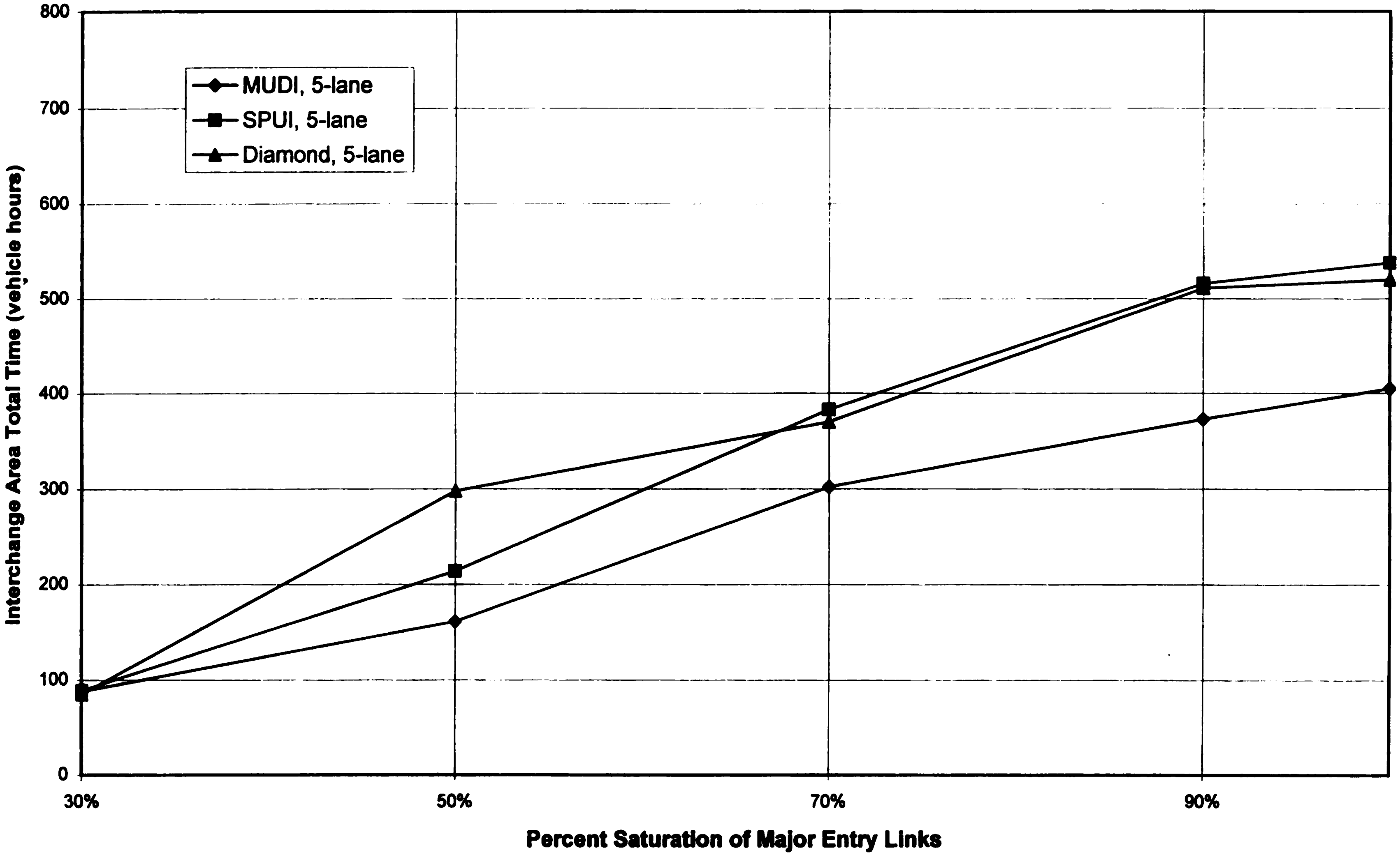


Figure B.16: Interchange Area Total Time For 70% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial

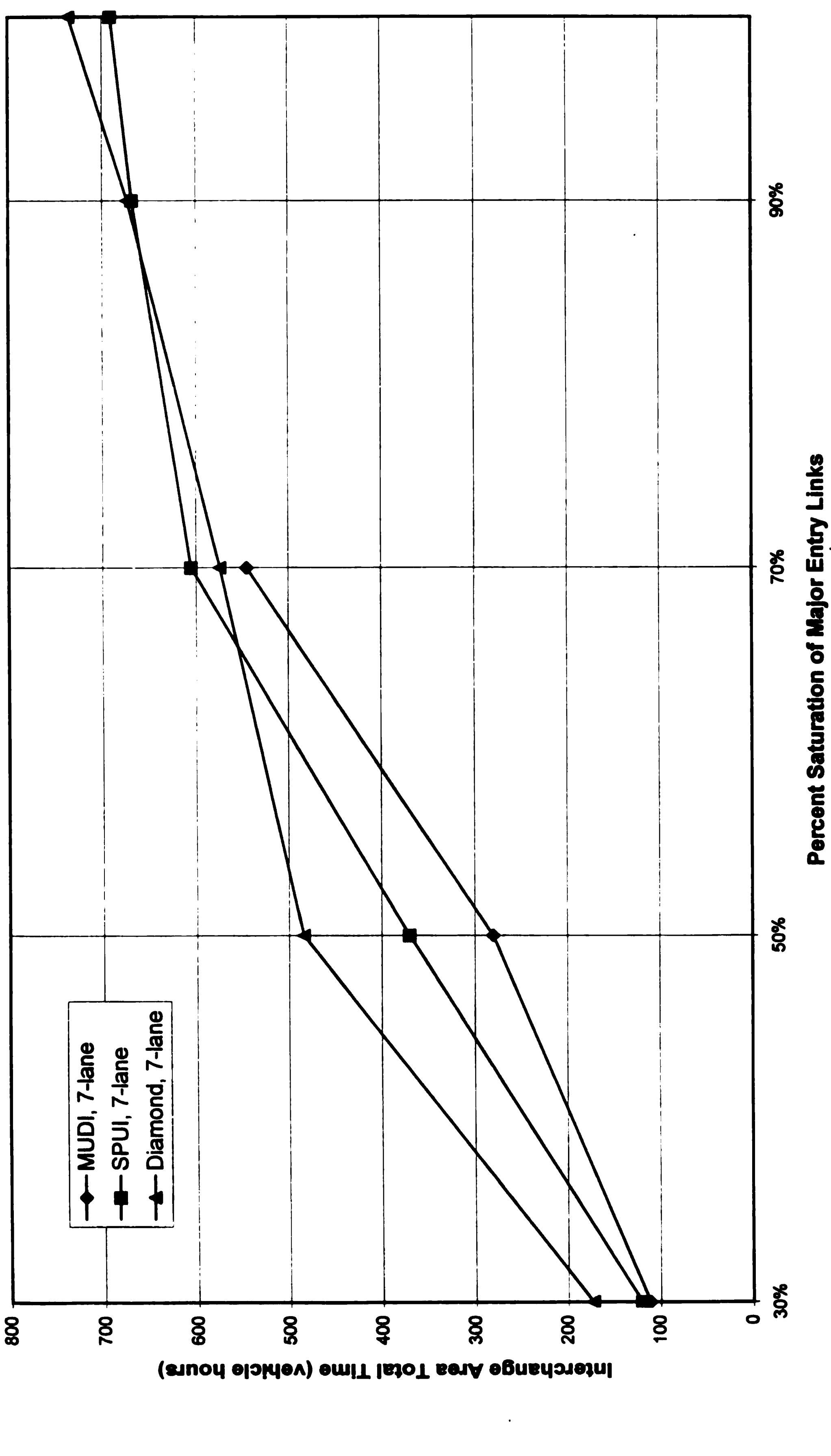


Figure B.17: Interchange Area Total Time For 50% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial

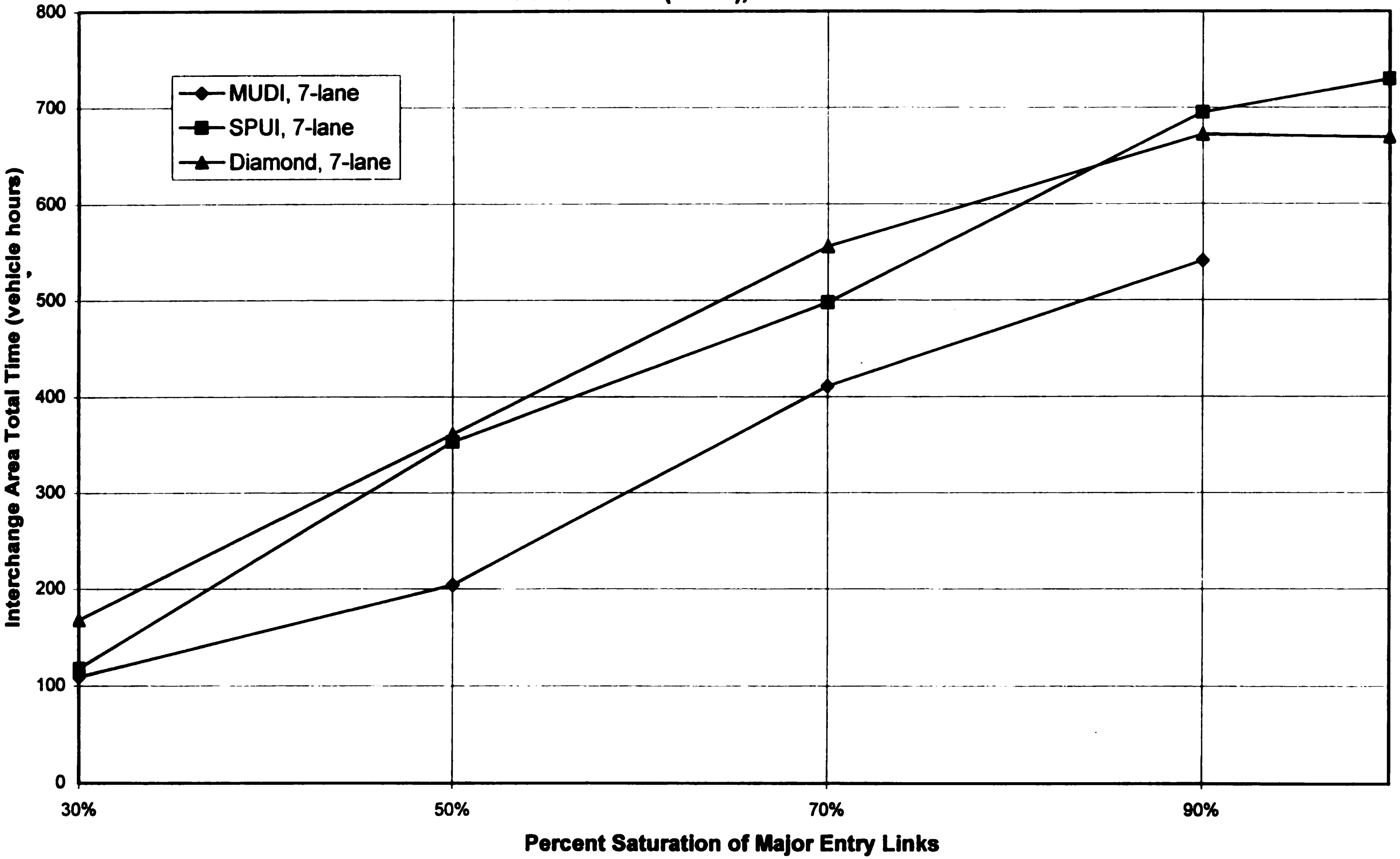


Figure B.18: Interchange Area Total Time For 30% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial

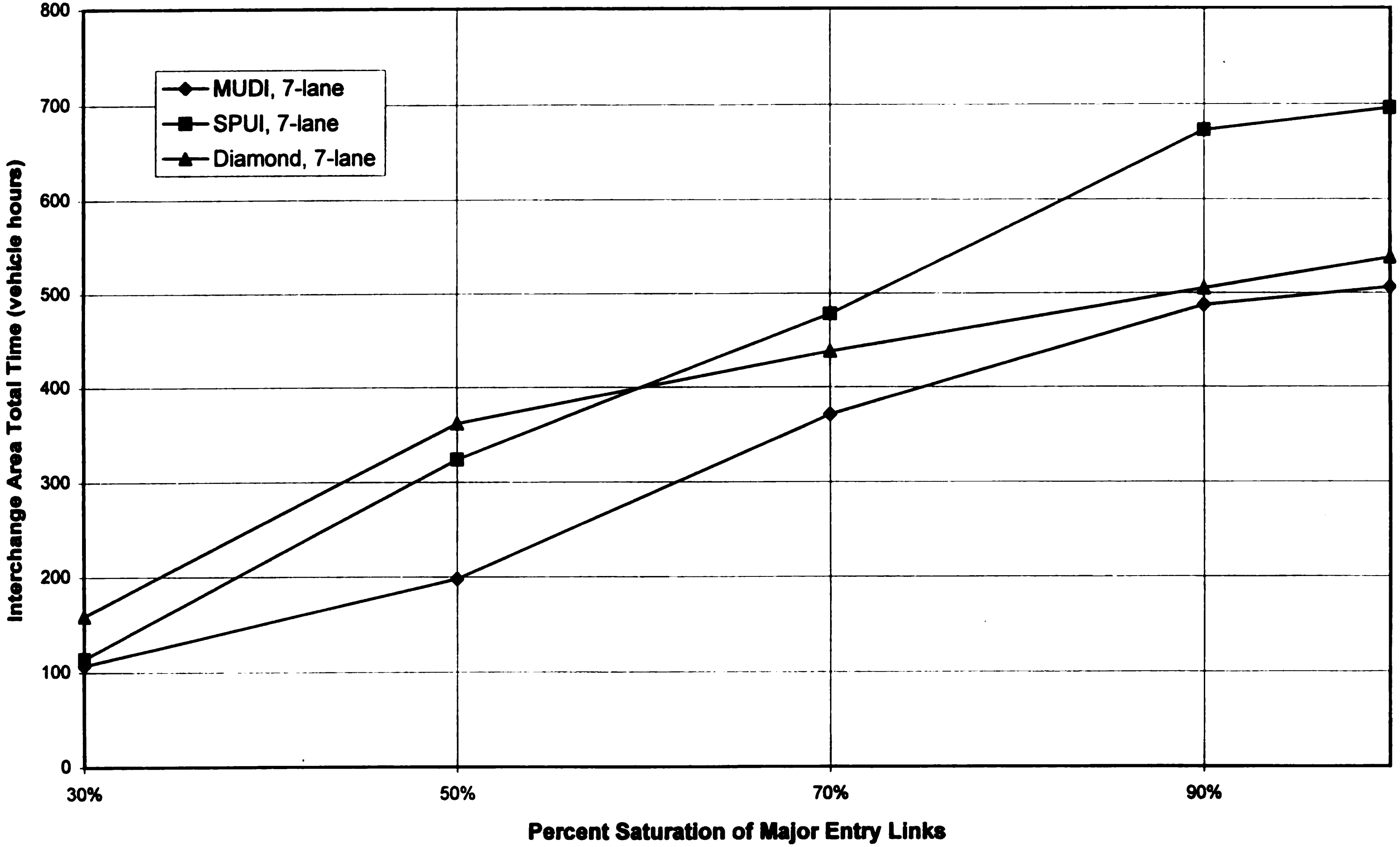


Figure B.19: Downstream Area Total Time For 70% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial

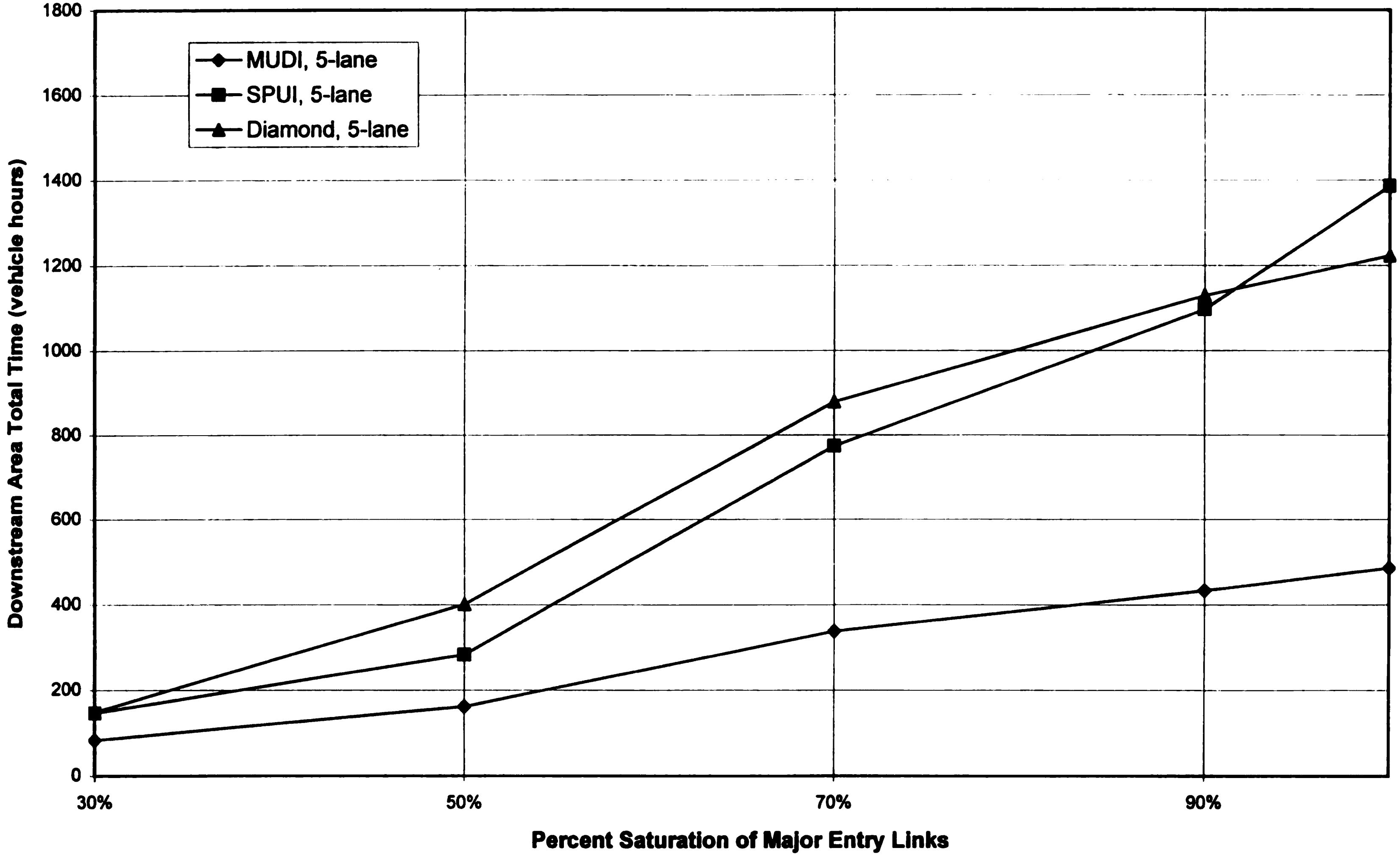


Figure B.20: Downstream Area Total Time For 50% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial

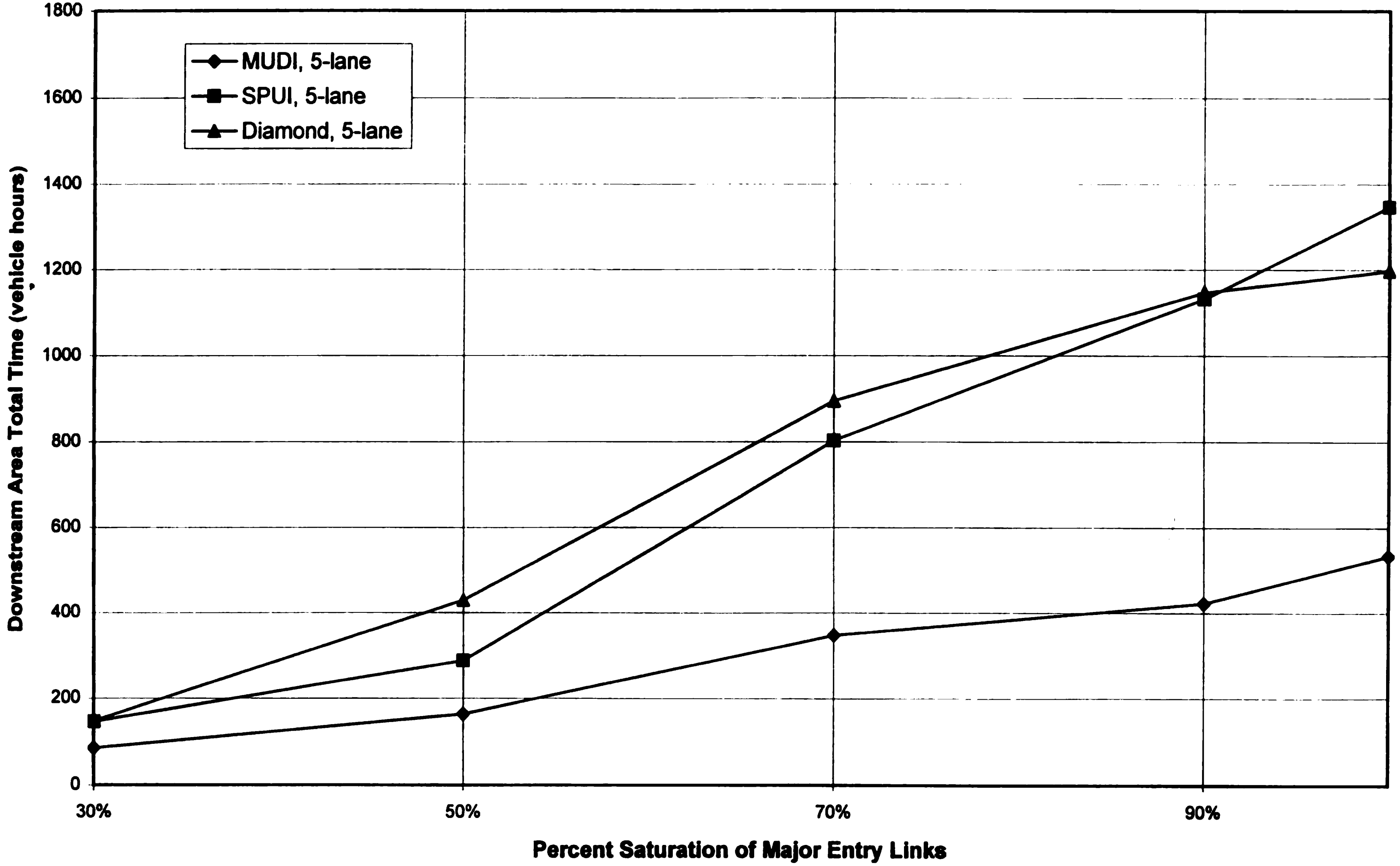


Figure B.21: Downstream Area Total Time For 30% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 5-lane Arterial

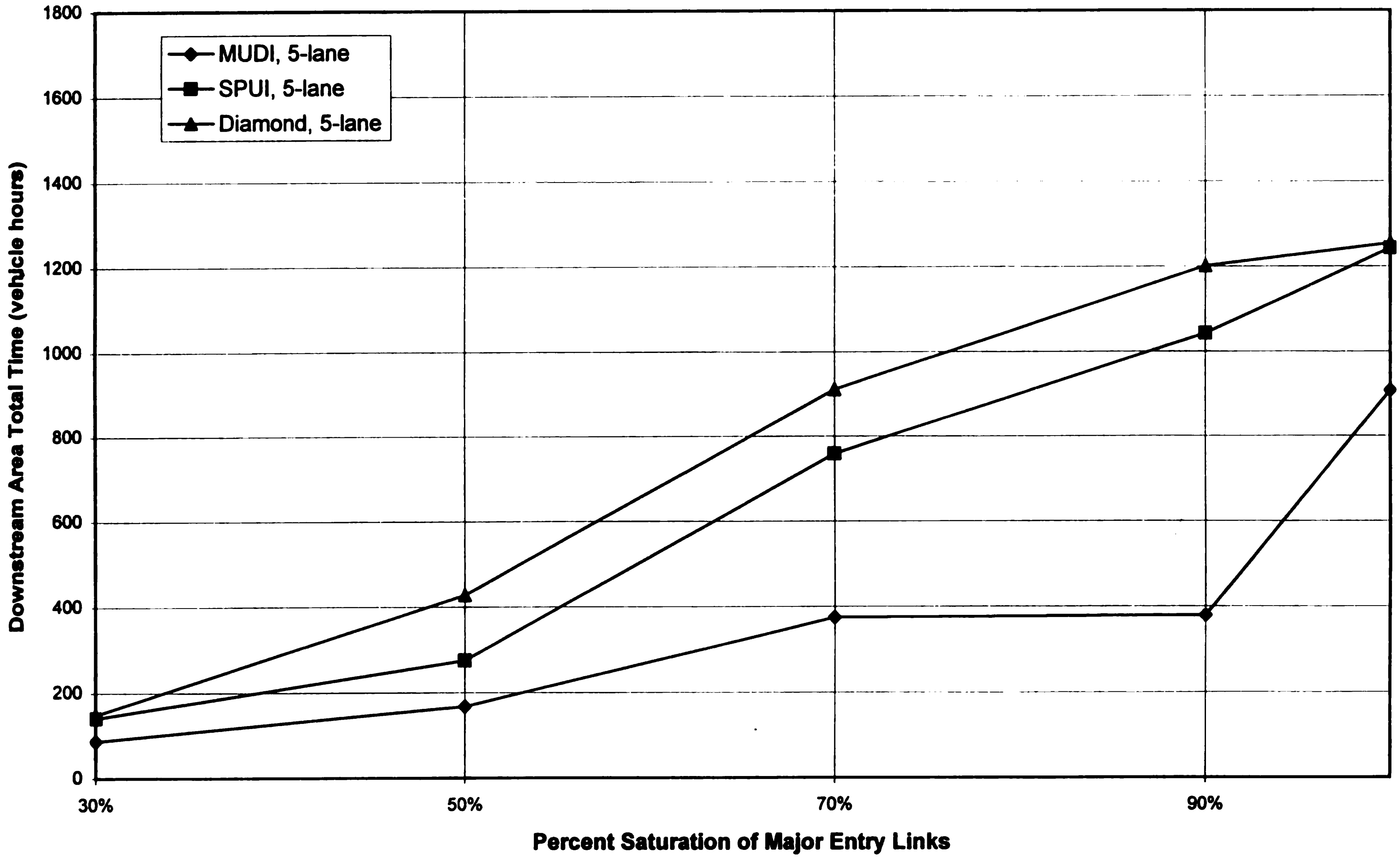


Figure B.22: Downstream Area Total Time For 70% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial

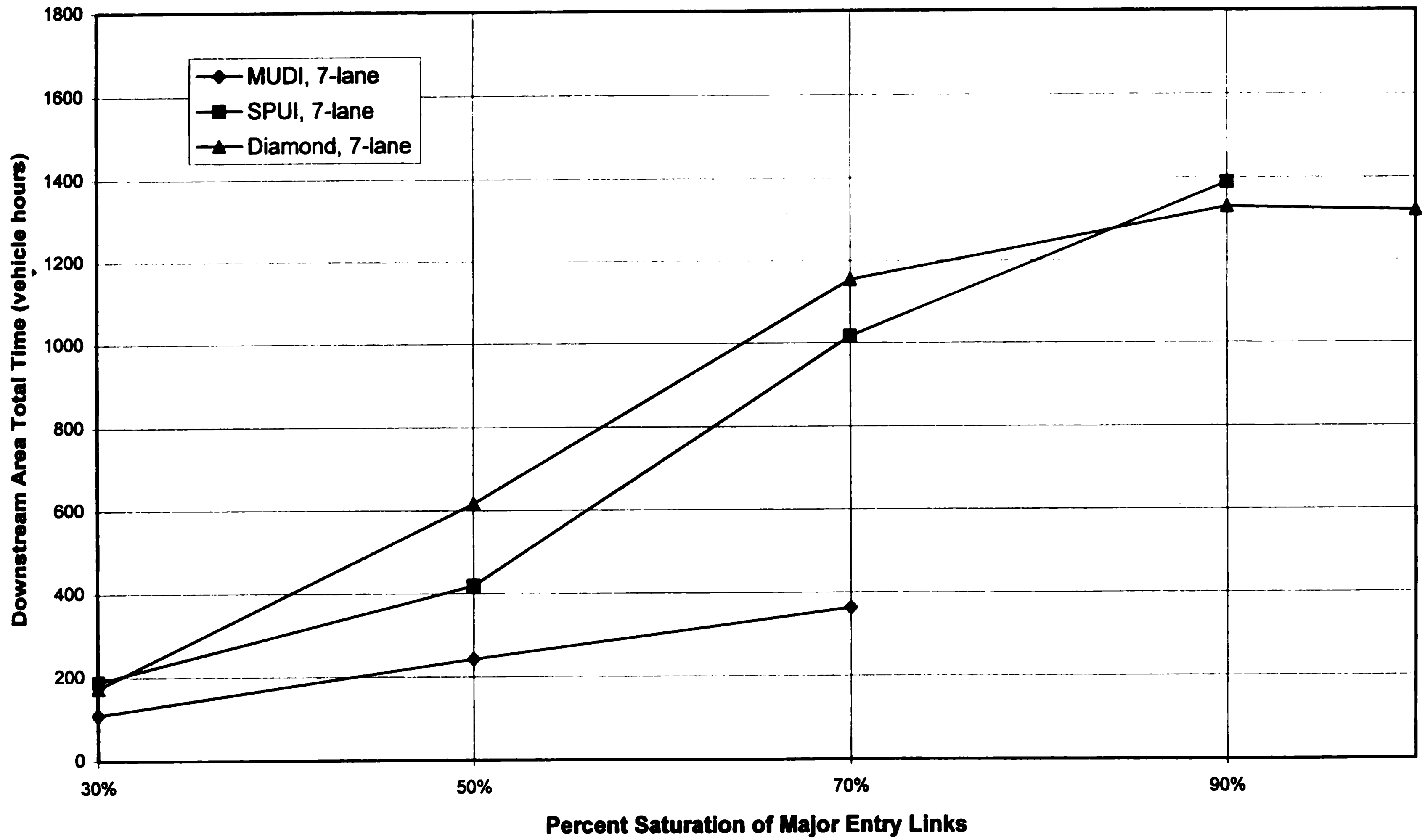


Figure B.23: Downstream Area Total Time For 50% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial

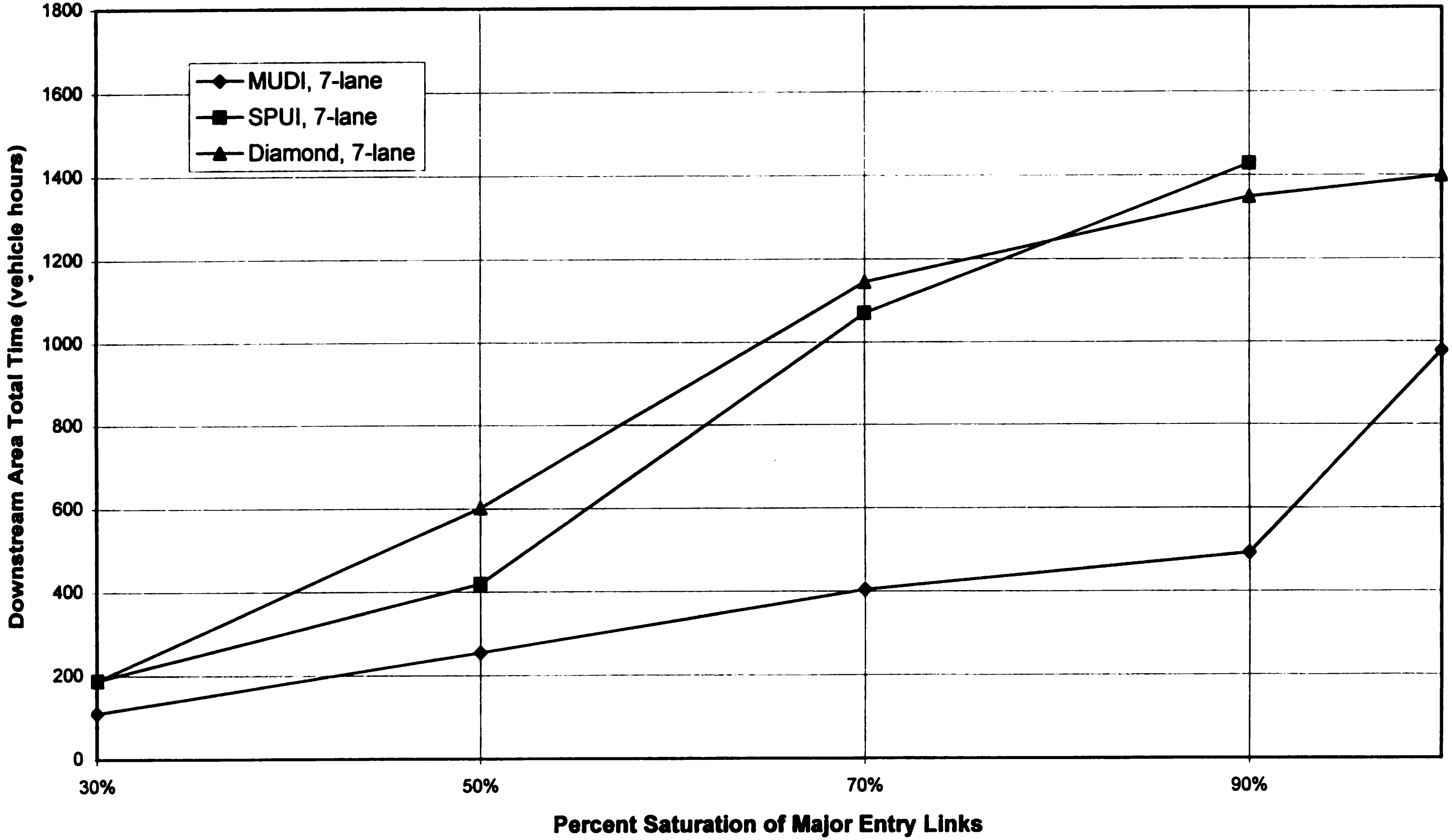


Figure B.24: Downstream Area Total Time For 30% Left Turns, with Frontage Roads, 1.6 kilometers (1 mile), 7-lane Arterial

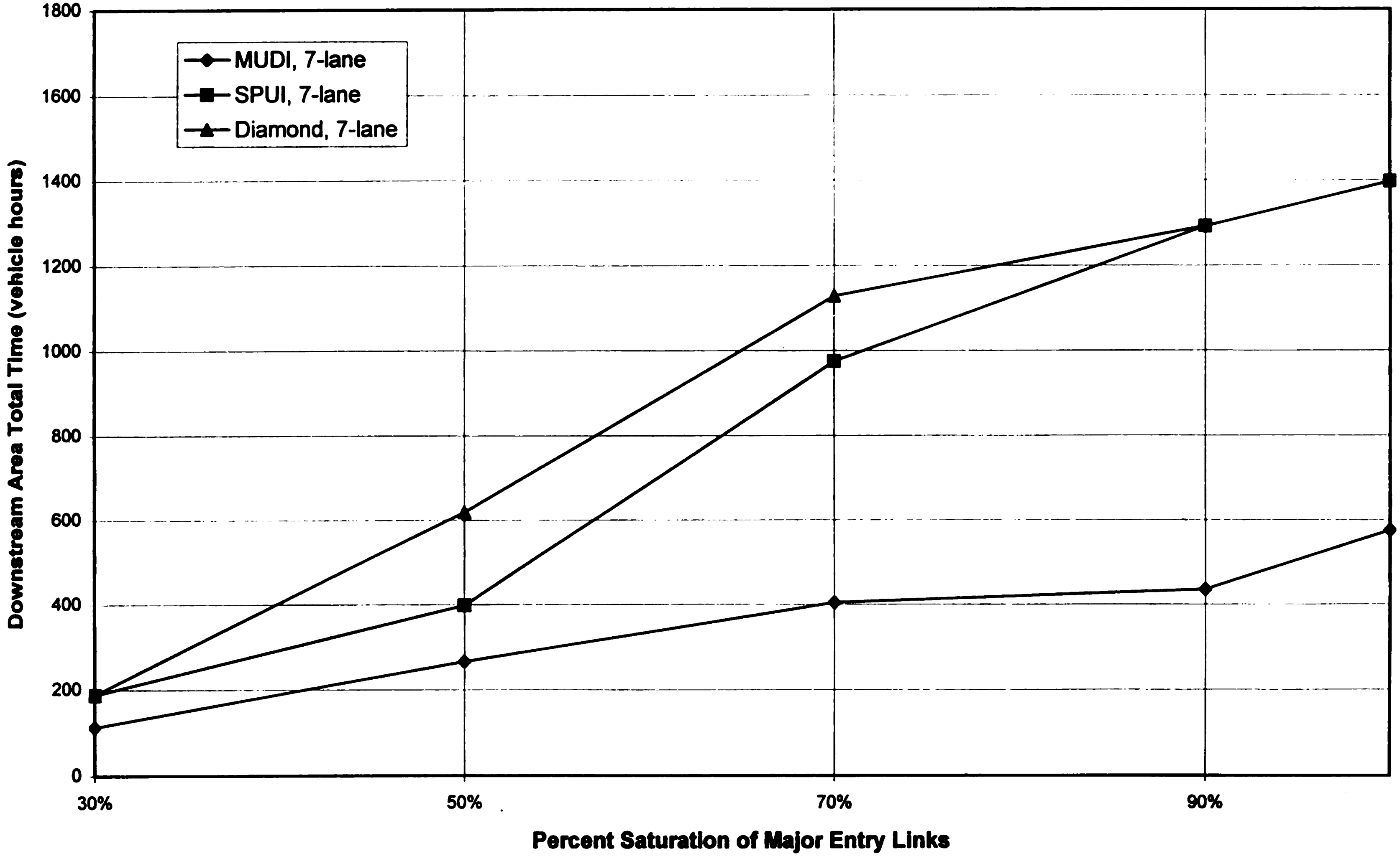


Figure B.25: Interchange Area Total Time for 70% Left Turns, w/out Frontage Roads, 5-lane Arterial, Varying Spacing Scenarios

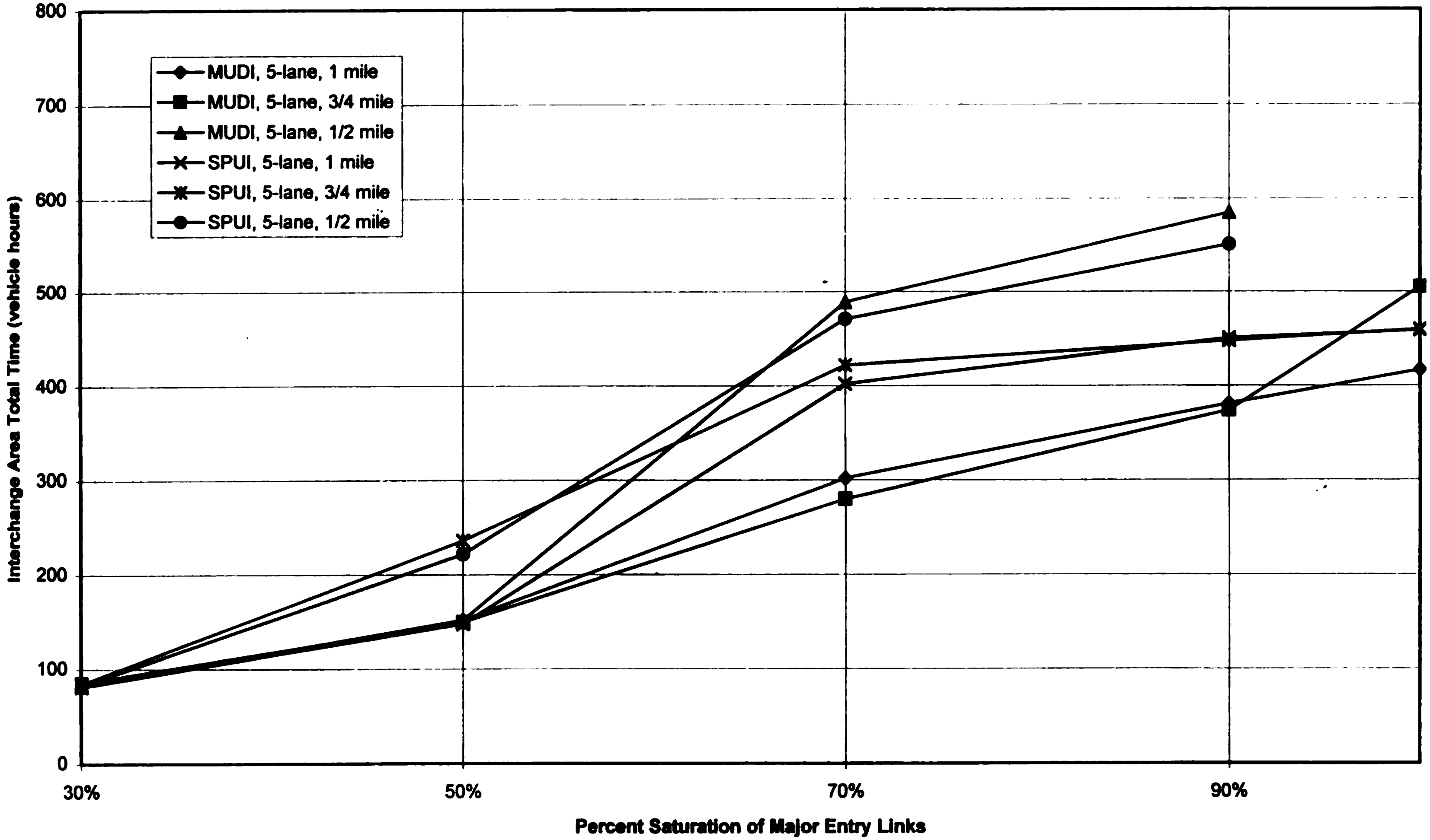


Figure B.26: Interchange Area Total Time for 50% Left Turns, w/out Frontage Roads, 5-lane Arterial, Varying Spacing Scenarios

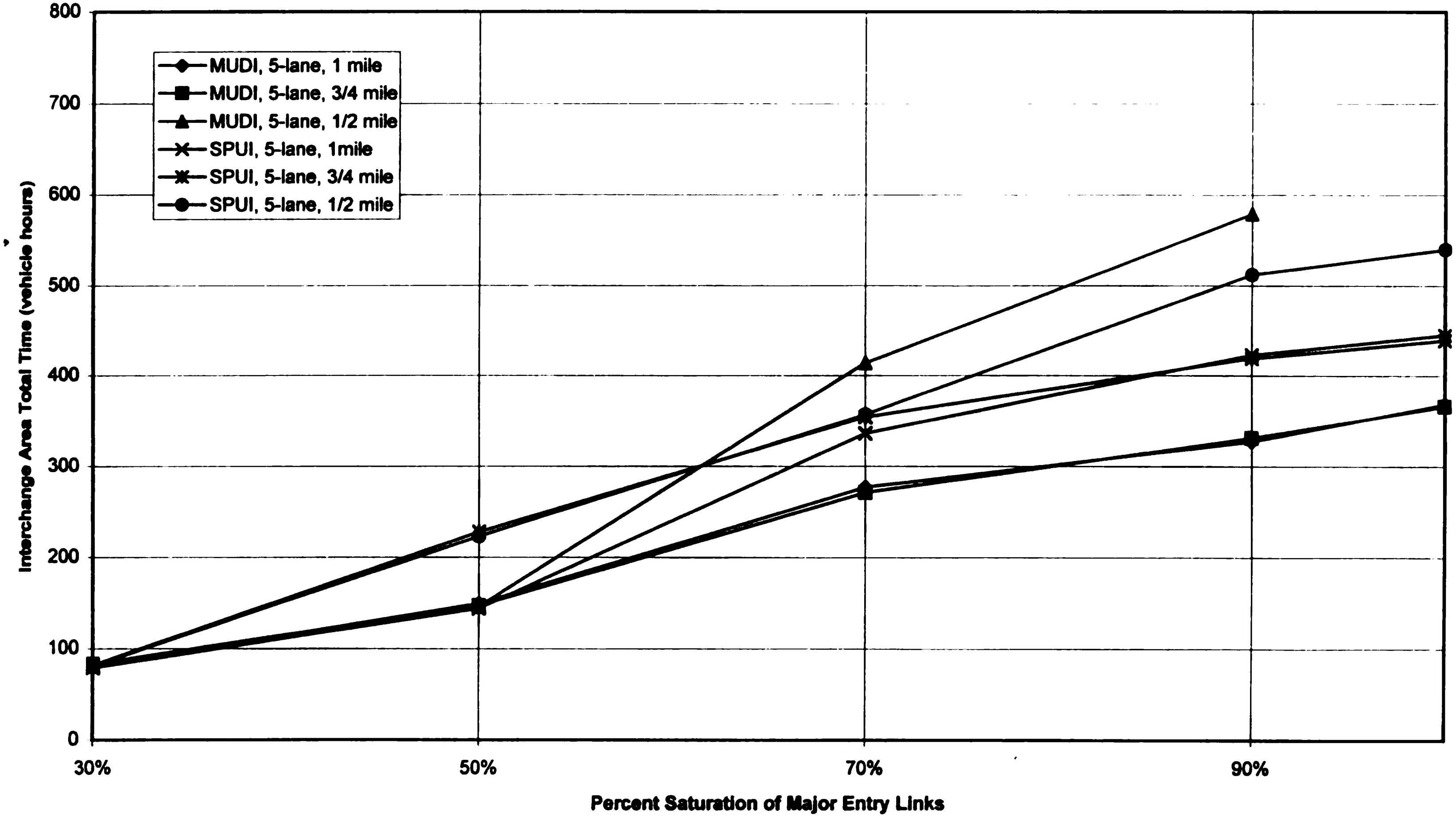


Figure B.27: Interchange Area Total Time for 30% Left Turns, w/out Frontage Roads, 5-lane Arterial, Varying Spacing Scenarios

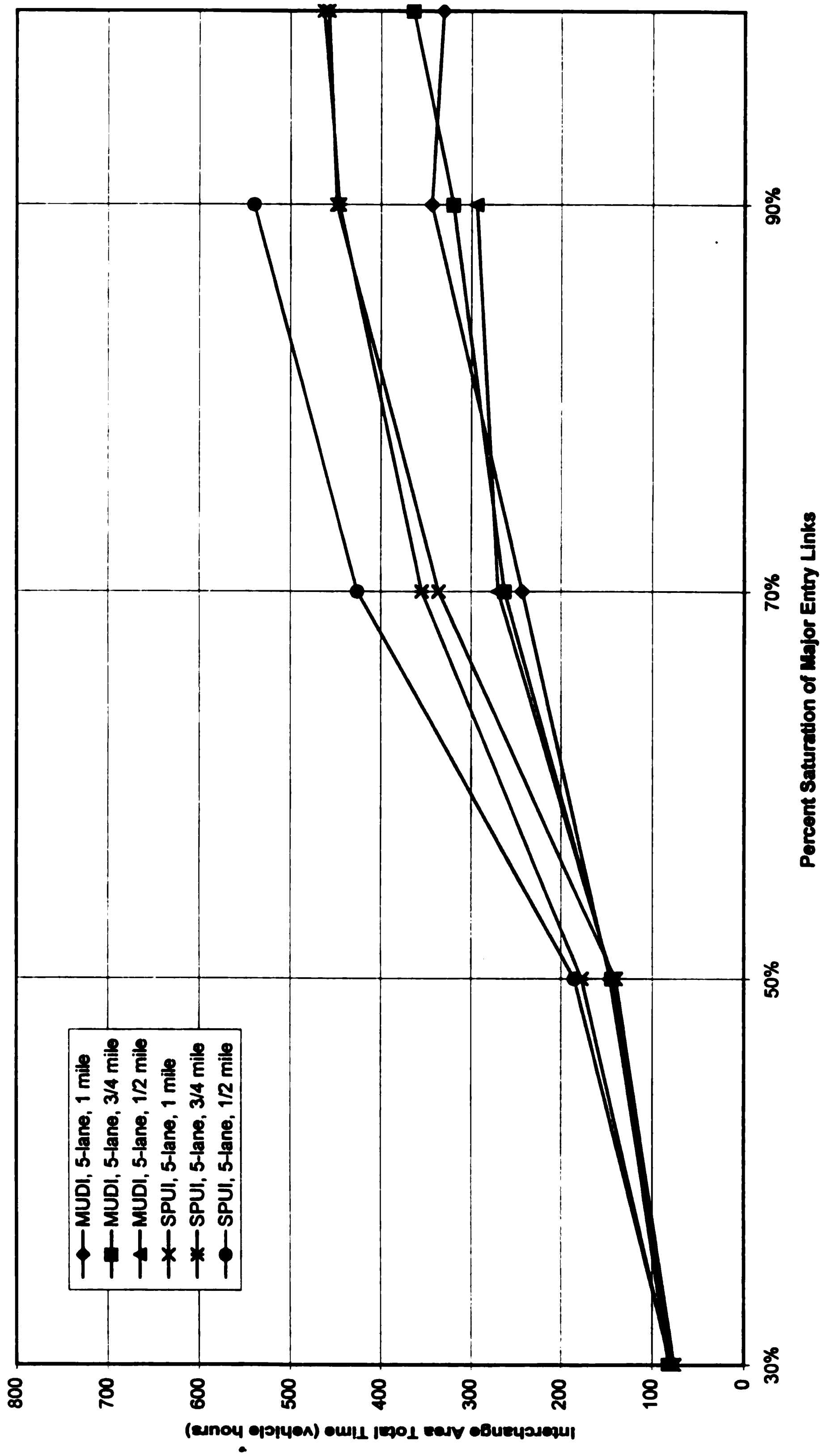


Figure B.28: Interchange Area Total Time for 70% Left Turns, w/out Frontage Roads, 7-lane Arterial, Varying Spacing Scenarios

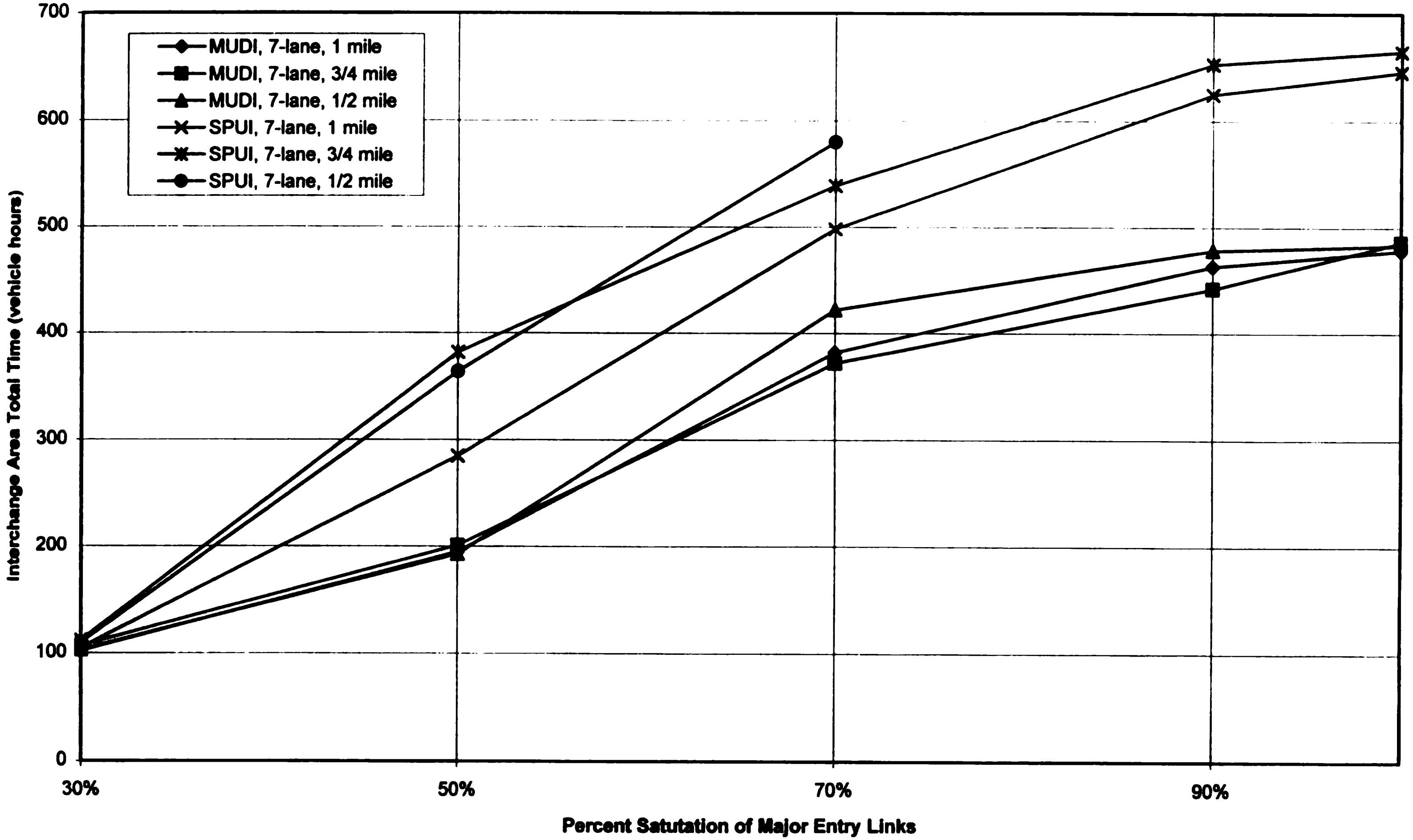


Figure B.29: Interchange Area Total Time for 50% Left Turns, w/out Frontage Roads, 7-lane Arterial, Varying Spacing Scenarios

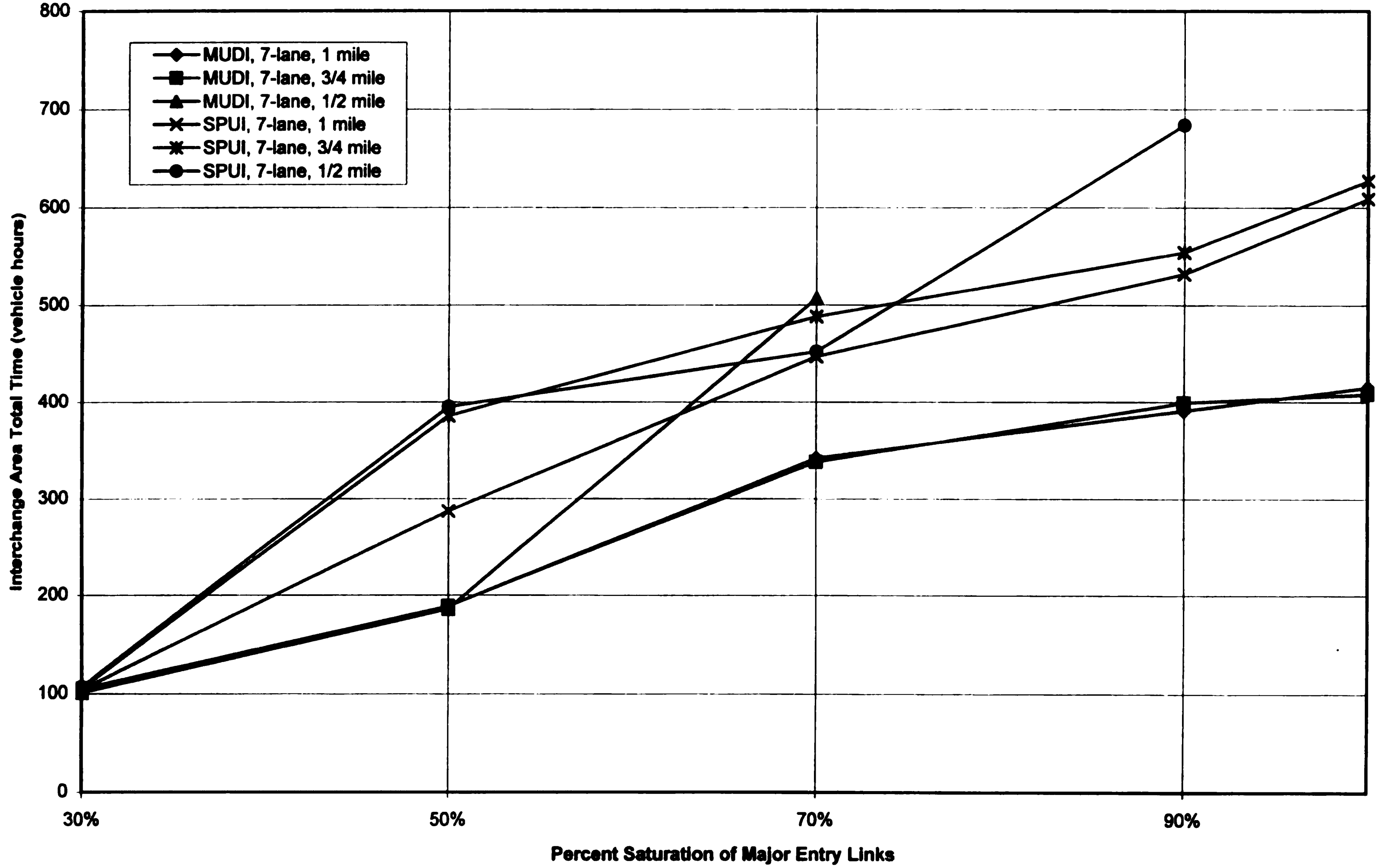


Figure B.30: Interchange Area Total Time for 30% Left Turns, w/out Frontage Roads, 7-lane Arterial, Varying Spacing Scenarios

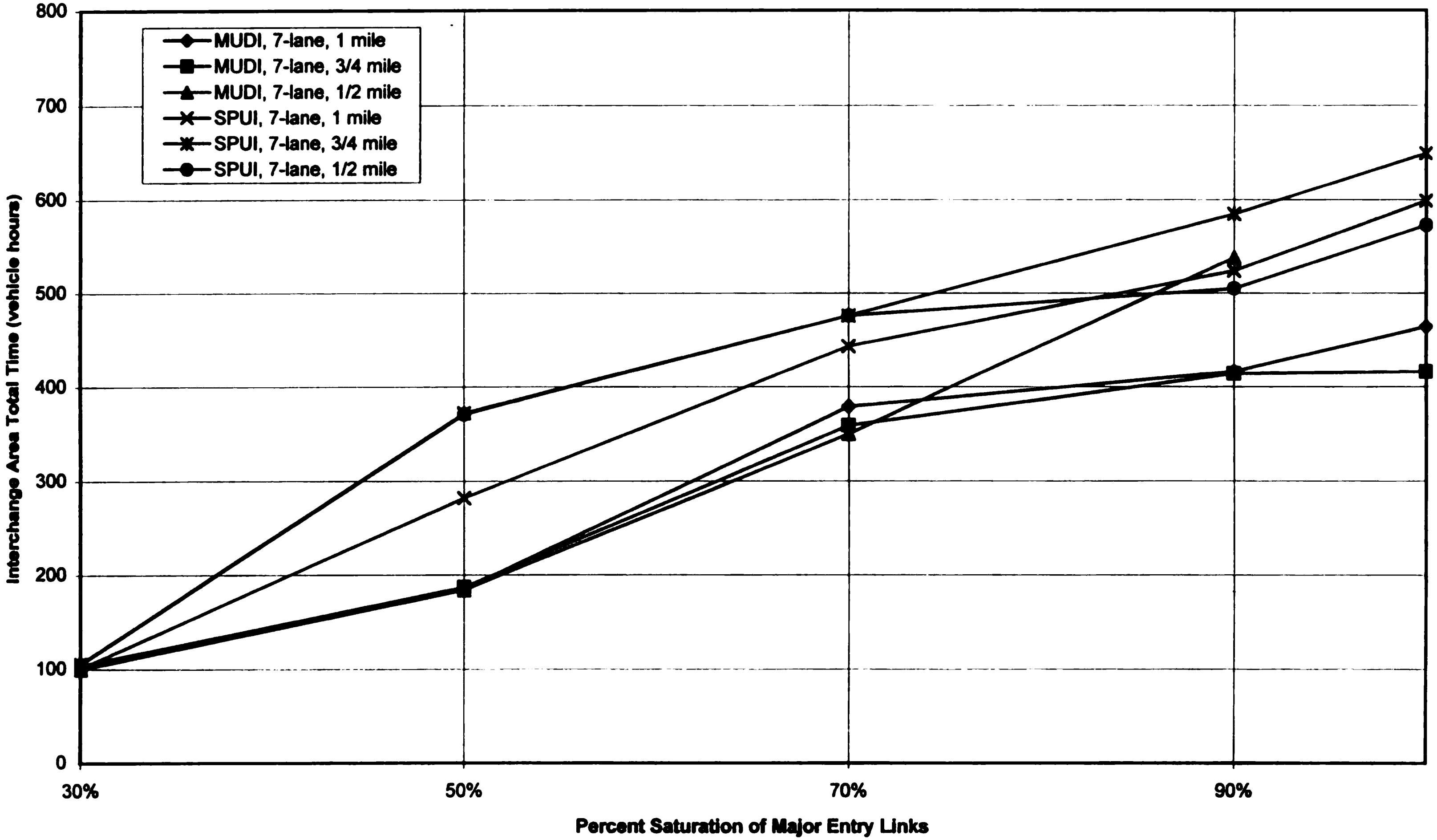


Figure B.31: Downstream Area Total Time for 70% Left Turns, w/out Frontage Roads, 5-lane Arterial, Varying Spacing Scenarios

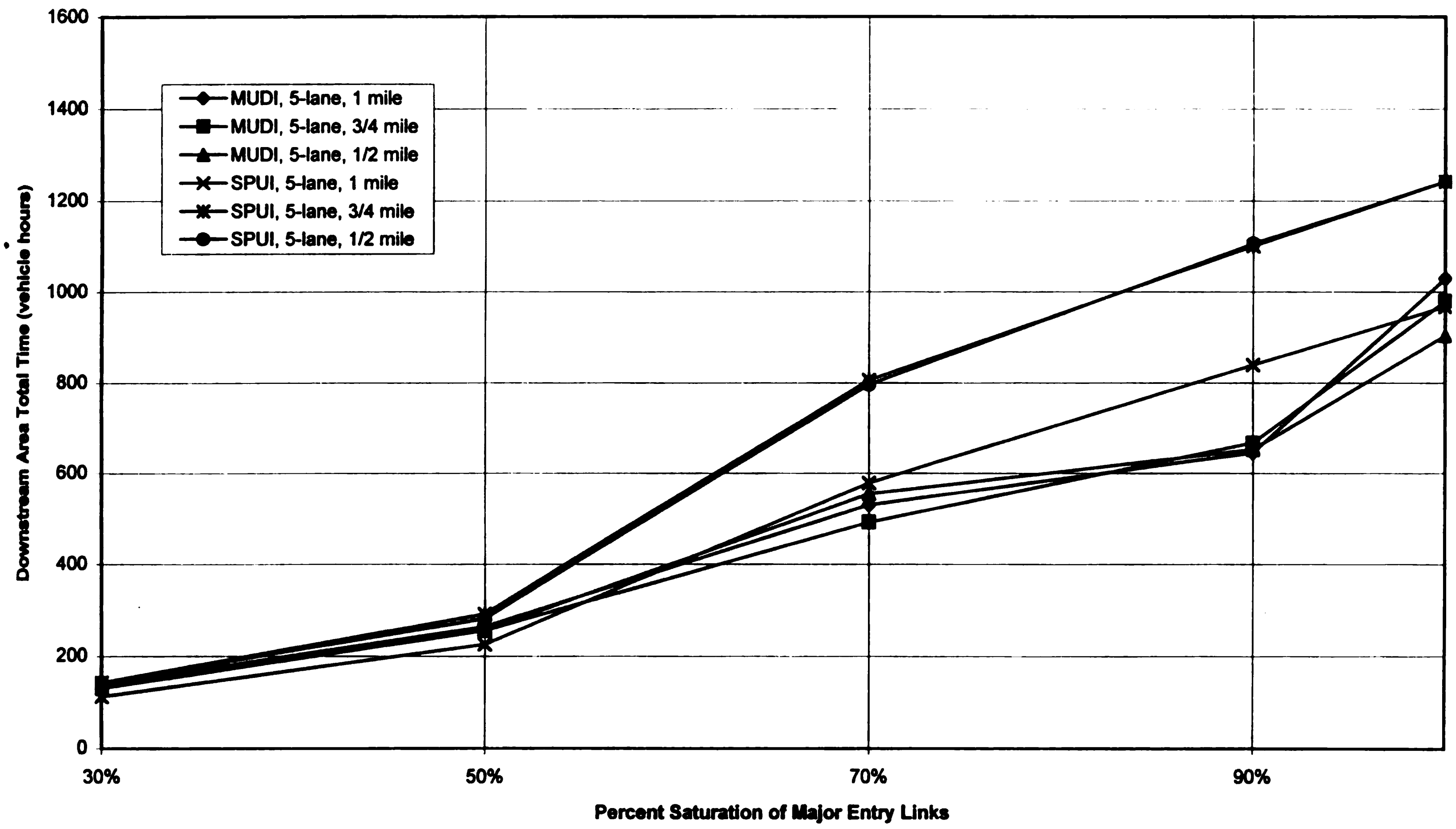


Figure B.32: Downstream Area Total Time for 50% Left Turns, w/out Frontage Roads, 5-lane Arterial, Varying Spacing Scenarios

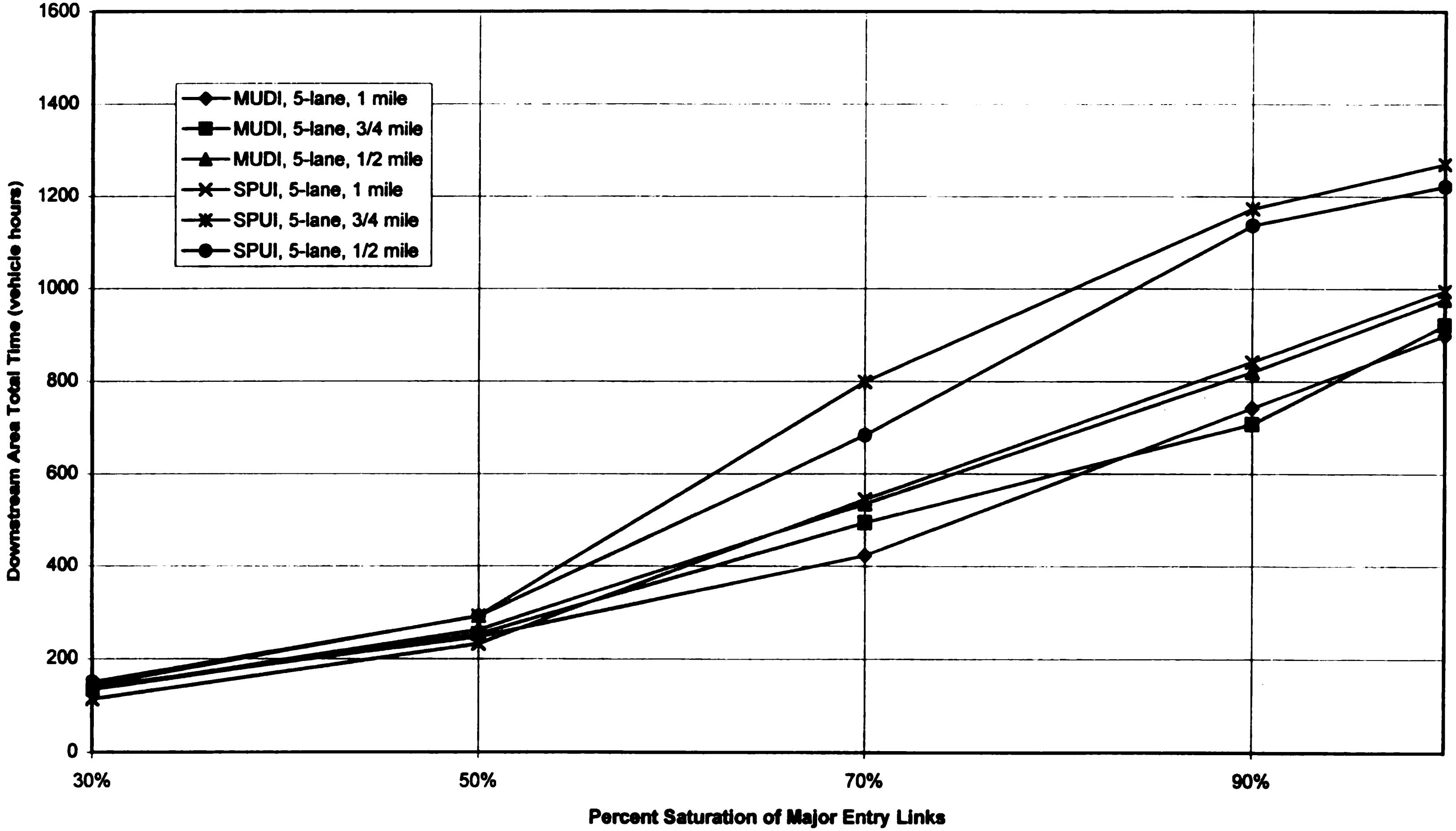


Figure B.33: Downstream Area Total Time for 30% Left Turns, w/out Frontage Roads, 5-lane Arterial, Varying Spacing Scenarios

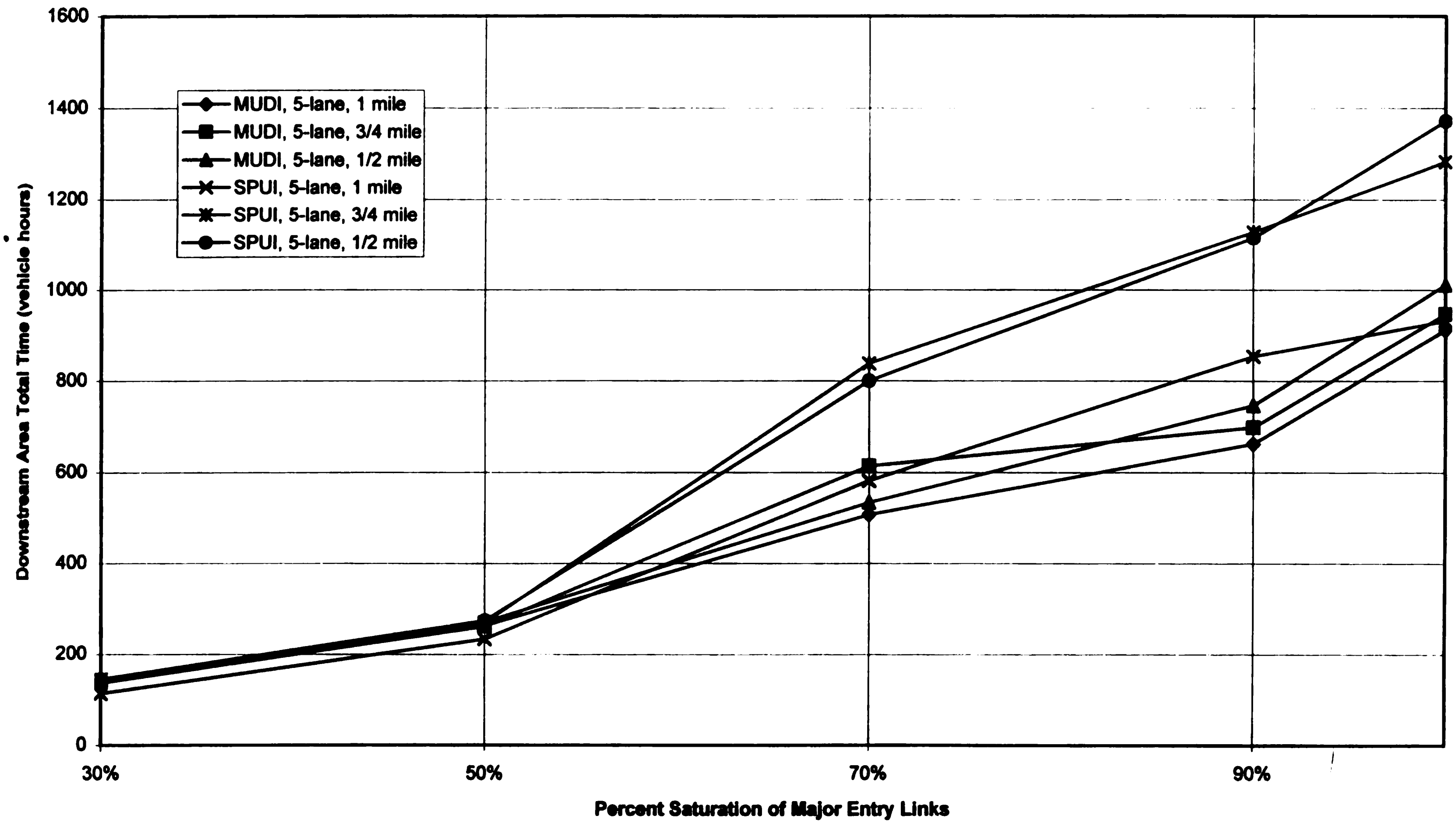


Figure B.34: Downstream Area Total Time for 70% Left Turns, w/out Frontage Roads, 7-lane Arterial, Varying Spacing Scenarios

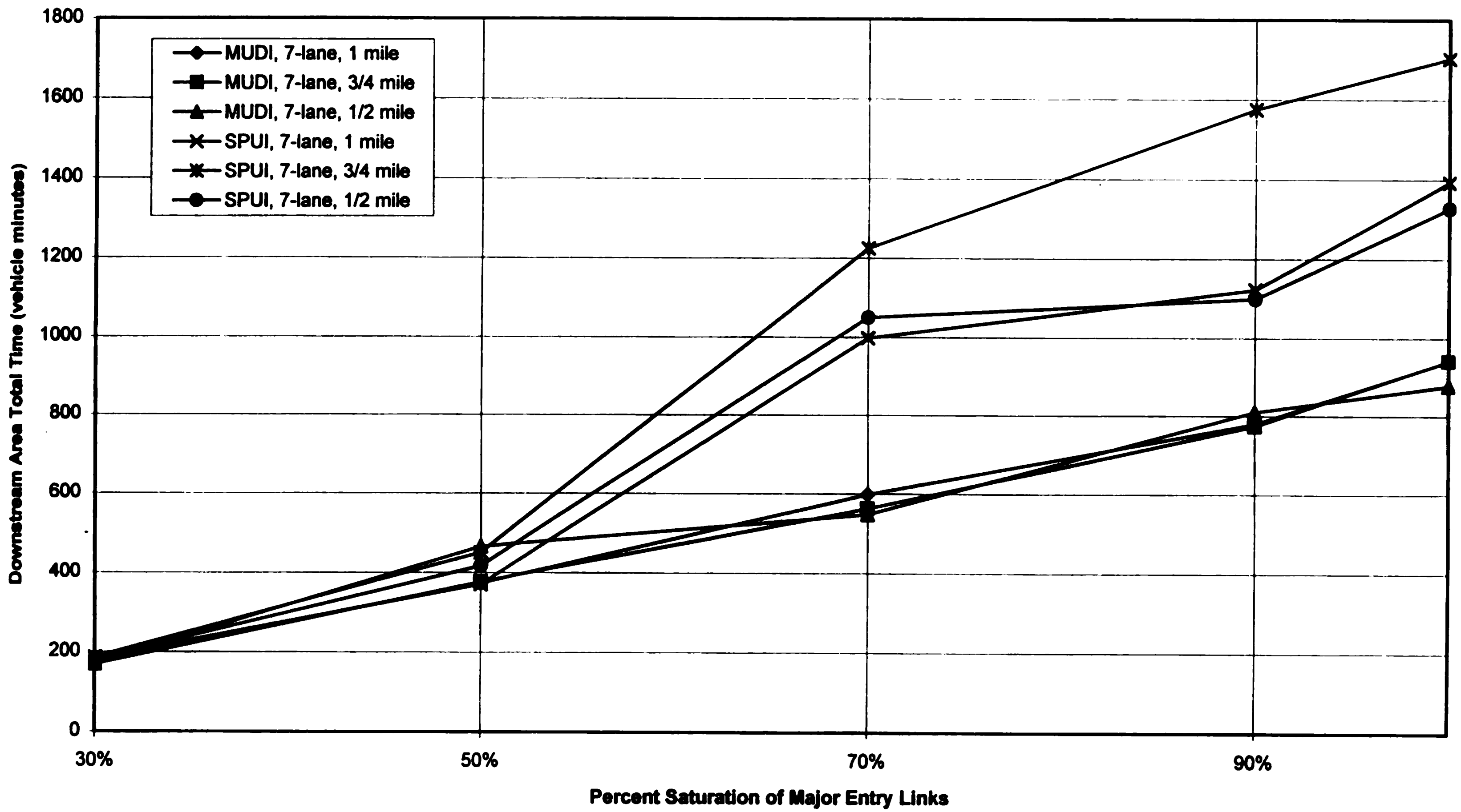


Figure B.35: Downstream Area Total Time for 50% Left Turns, w/out Frontage Roads, 7-lane Arterial, Varying Spacing Scenarios

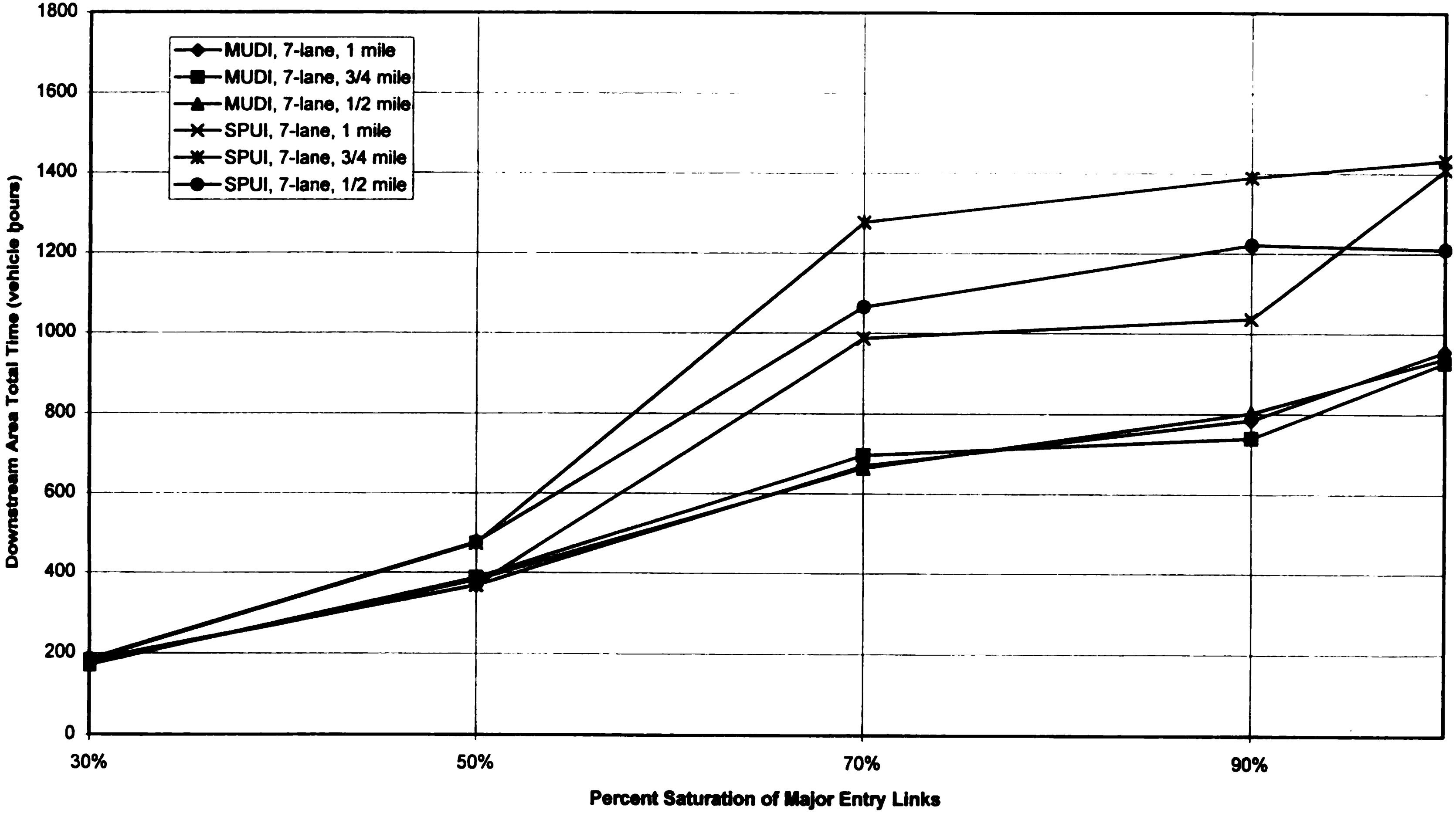
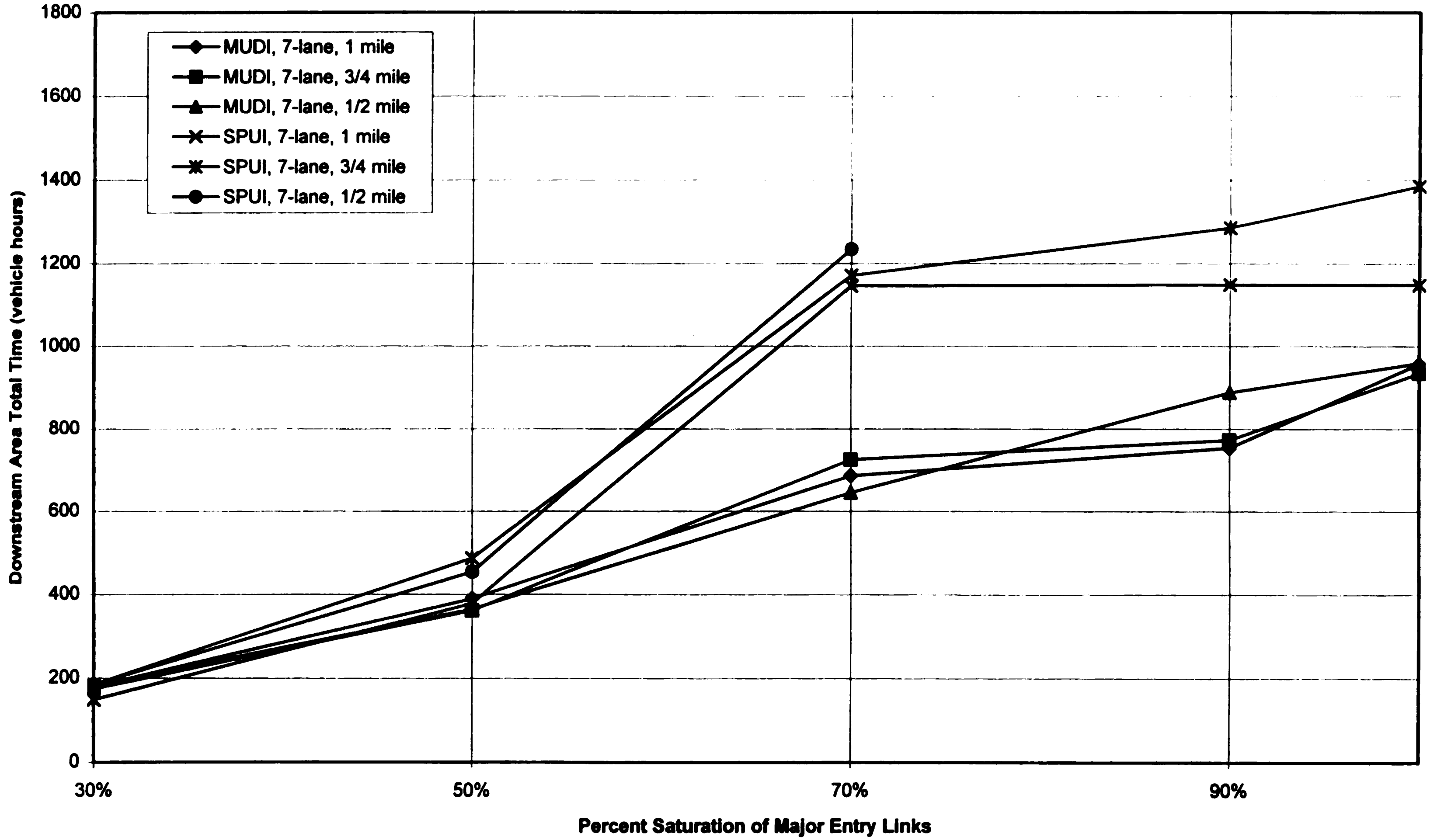


Figure B.36: Downstream Area Total Time for 30% Left Turns, w/out Frontage Roads, 7-lane Arterial, Varying Spacing Scenarios



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