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EFFICIENCY AND RESILIENCE TRADE-OFFS FOR ROADWAY INTERSECTION DESIGN IN THE U.S. VIRGIN ISLANDS

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THESIS

EFFICIENCY AND RESILIENCE TRADE-OFFS FOR ROADWAY INTERSECTION DESIGN IN THE U.S. VIRGIN ISLANDS

by

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EFFICIENCY AND RESILIENCE TRADE-OFFS FOR ROADWAY INTERSECTION DESIGN IN THE U.S. VIRGIN ISLANDS

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ABSTRACT

In September 2017, two Category-5 hurricanes made indirect hits on the U.S. Virgin Islands (USVI) and devastated local communities. The hurricanes acutely impacted surface roads and supply chains, where access to critical supplies was reduced due to impeded travel through intersections with broken and inoperable traffic signals. In this work, we assessed the possible post-disaster travel time benefits of converting intersections in the USVI into roundabouts. Roundabouts are more robust to disaster impacts because they do not require traffic signals to operate, but may be less efficient for normal traffic conditions. Thus, we studied efficiency-resilience tradeoffs for intersection design and developed a model to compare roundabouts to signalized intersections before and after a disaster. Our results show that roundabouts are unnecessary for intersections with low traffic flows on St. Croix and St. Thomas. However, roundabouts will be more efficient than traffic signals at intersections with high traffic and adjacent supply locations (e.g., shopping malls). At these intersections, roundabouts may also reduce vehicle travel times by upward of 25 minutes after a disaster when traffic lights are inoperable.

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List of Acronyms and Abbreviations

BPR Bureau of Public Roads

DTA Dynamic Traffic Assignment

DPW Department of Public Works

FHWA Federal Highway Administration

FEMA Federal Emergency Management Agency

FHA Federal Highway Administration

HCM Highway Capacity Manual

NPS Naval Postgraduate School

pcu Passenger Car Units

STJ St. John

STT St. Thomas

STX St. Croix

TRB Transportation Research Board

TRRL Transport and Road Research Laboratory

US United States.

USVI U.S. Virgin Islands

USD U.S. Dollars

USDOT U.S. Department of Transportation

UVI University of the Virgin Islands

VIDPW	Virgin Islands Department of Public Works
VIPA	Virgin Islands Port Authority
VITEMA	Virgin Islands Territorial Emergency Management Agency
VIWAPA	Virgin Islands Water and power authority
VPH	Vehicles per hour

Executive Summary

In September 2017, two Category-5 hurricanes (Irma and Maria) made indirect hits on the U.S. Virgin Islands (USVI) within a two-week period. As a result, all lifeline infrastructure systems were disrupted. In particular, local transportation and supply chain infrastructure was significantly damaged leading to blocked roads, a sevenfold increase in the car accident rate, and local communities living under a curfew to keep roads clear for recovery work. To reduce the impacts of future storms, it is important to make the transportation system resilient to disasters.

The goal of this thesis is to determine which mitigation actions can be taken to minimize the negative impact of future disasters on the USVI transportation system while supporting efficient day-to-day operation. We focus our analysis on roadway intersections, as intersections with traffic lights can cause significant delays after a storm when electricity is unavailable and the intersection operates as a stop sign. We assess the travel time benefits brought by changing intersections into roundabouts that do not require electricity to operate and are more robust to disasters.

While roundabouts may be more robust to storms, they may also lead to increased traffic during normal operations. Thus, we define a set of models to estimate travel time through roadway intersections and create efficiency and resilience metrics that determine travel time pre- and post-storm. We use these metrics to measure trade-offs between intersection designs and to determine the superior alternative for intersections in the USVI. Overall, the technical output of this thesis includes:

- a review of models useful to assess the time delay incurred when driving through roundabouts, traffic signals, and stop signs;
- a method to assess travel time through the same intersection with different designs that integrates into network flow models for dynamic traffic assignment;
- a parametric analysis of roundabout and signalized intersection delay times pre- and post-disaster to determine the effects of traffic flows on efficiency and resilience; and,
- case studies of four intersections on the islands on St. Croix and St. Thomas.

Our results measure the expected performance of intersection alternatives in both normal

and in disaster scenarios. In general, we conclude that roundabouts, traffic signals, and stop signs each have their own ideal operating range of traffic entering the intersection. Assuming traffic flows and intersection parameters are known, a superior alternative for an intersection can be identified based on delay time.

We also conclude that traffic signals are far less robust to disaster than roundabouts. Assuming traffic signals act as stop signs after a disaster, we find significant increases in delay time for traffic signals before and after a disaster. This additional delay time reflects the resilience of the intersection to failure. These results hold for both generic intersections and case studies within the USVI, and emphasize that roundabouts are generally better for the territory. We find roundabouts are more efficient and robust for all intersections studied on St. Croix and St. Thomas.

However, the benefits of roundabouts are more pronounced for intersections with high traffic flows approaching the intersection via few entries. Where traffic flows are large, roundabouts can reduce total travel time by several minutes in pre-storm traffic and by upwards of 25 minutes in post-storm traffic. Based on our analysis, we recommend that the Queen Mary Highway and Centerline Rd. intersection on St. Croix and the Tutu Choke Point intersection on St. Thomas should consider changing their design to a roundabout. In both cases, the traffic dynamics and intersection geometry lend themselves to be suitable for roundabouts.

This thesis is in support of a larger effort funded by the Federal Emergency Management Agency to recover lifeline infrastructure systems in the USVI. The results support recovery and future disaster mitigation by identifying which intersections should prioritize new designs when disrupted by future storms and how those new designs may mitigate losses of mobility. This thesis supports future work towards this purpose, with the intention to include intersection design and traffic delays in territorial recovery and mitigation plans led by the University of the Virgin Islands. Moreover, this work supports future efforts to assess interdependent infrastructure vulnerabilities, such as cascading failures across electricity and transportation systems.

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CHAPTER 1:
Introduction

In September 2017, two Category-5 hurricanes (Irma and Maria) made indirect hits on the U.S. Virgin Islands (USVI) within a two-week period. As a result, all lifeline infrastructure systems that provide critical services like electricity, water, mobility, and communications were disrupted. In particular, local transportation and supply chain infrastructure was significantly damaged leading to blocked roads, the car accident rate increasing seven times, and local communities living under a curfew to keep roads clear for recovery work (Alderson et al. 2018b). In response to these damages, the USVI initially received almost $1B in federal funding from Federal Emergency Management Agency (FEMA) to re-establish itself, with roughly $450 M dedicated for public assistance funding and reimburse territorial agencies for debris removal, power restoration, emergency protective measures and other eligible costs (FEMA 2018). The territory continues to recover from Irma and Maria three years after the storms and is receiving new federal funds to upgrade and adapt transportation systems to survive future disasters.

To reduce the impacts of future storms, it is important to make the transportation system *resilient* to disasters. We define resilience as the capacity for a system to sense, anticipate, adapt, and learn from surprising events (Eisenberg et al. 2019). In general, transportation systems are designed to provide safe and efficient mobility for day-to-day use. However, normal roadway designs for efficiency may not be resilient to surprising events like back-to-back Category-5 storms. The trade-offs between resilience and efficiency for transportation infrastructure upgrades and mitigation activities are not fully known. There is a risk of not employing federal funds in a manner that supports long-term resilience, or worse, recovered and upgraded infrastructure may make the USVI transportation system more vulnerable to the next storm.

The goal of this thesis is to determine which mitigation actions can be taken to minimize the negative impact of future disasters on the USVI transportation system while supporting efficient day-to-day operation. We define an efficiency-resilience space to determine trade-offs between various transportation system designs. We focus analysis on surface road networks and supply chain infrastructure systems.

We assess efficiency-resilience tradeoffs for a single roadway design question: What *intersections* are both efficient for day-to-day traffic and resilient to disasters? To answer this question, we compare the travel time and resilience effects of roundabouts, traffic lights, and four way stops. We analyze how changing from one intersection to another influences surface road mobility during normal operations and after a major disaster to provide recommendations for transportation hazard mitigation in the USVI. Taken together, we determine how choice of intersection design may impact system efficiency and resilience. Moreover, the measures and models developed in this work broadly inform efficiency-resilience tradeoffs for transportation and supply chain infrastructure.

This thesis is part of a broad, FEMA-funded effort by the Naval Postgraduate School (NPS) to improve infrastructure recovery and resilience in the USVI. This work is completed in collaboration with numerous federal, territorial, private, and community stakeholders and organizations, including the University of the Virgin Islands (UVI), Virgin Islands Department of Public Works (VIDPW), and U.S. Department of Transportation (USDOT) among others. While this work focuses on surface transportation and supply chain systems, additional studies were also completed on USVI transportation (Good 2019; Routley 2020), energy (Bunn 2018; Wille 2019), water (Wille 2019; Borgdorff 2020), and telecommunications systems (Wine 2020; Moeller 2020).

1.1 The U.S. Virgin Islands Territory

The USVI is an unincorporated U.S. territory in the Caribbean Sea. As Figure 1.1 shows, the territory is comprised of three main islands — St. Thomas (STT), St. Croix (STX), St. John (STJ) — and many smaller, less-inhabited islands. Prior to becoming part of the United States, the USVI was a European territory ruled by Denmark. The United States purchased the territory in March 1917 for $25M. The current territorial population is about 105,000 people, with roughly half of the population living on either STT or STX. STJ is the smallest of the three main islands and is primarily a national park (U.S. Census Bureau 2020).

STT is the main island which serves as the territory's capital and is located in the north-west of the territory. Most of the government offices and official organizations are on STT and located in the city of Charlotte Amelie. STT has a population of roughly 51,000 and a size of 23 squared miles (Encyclopedia Britannica 2020). The island has a large population density and a mountainous geography that leads to steeper terrain and significant commuter traffic.

STX is the geographically largest island in the USVI territory. It's located about 40 miles south of STT. STX is the center for territorial agriculture and large industries. It has a population that is roughly the same as STT (50,000), but its geographic area size is almost three times larger (82 squared miles). STX is also relatively flatter than STT, leading to roads with fewer turns and faster speeds. Thus, the travel time to cross STX might be longer, due to size, but is generally driven at faster speeds, due to geography (Encyclopedia Britannica 2020). In general, STX has sufficient roadway capacity to manage supply chain issues even during roadway failures (Good 2019).

STJ is the smallest of the three main islands by size and population. STJ is located about three miles east of STT. The island mostly consists a national park that covers around 70% of the island area. About 4,000 people live on STJ with most residents living in either Cruz Bay, Coral Bay, or in isolated wealthy homes or luxury hotels.

Tourism, trade, and other services are the primary economic drivers in the USVI, accounting for more than half of territorial GDP (USVI Economic Development Authority 2020). More than 2 million tourists visit per year, and tourism-related jobs employ more than 20% of the total wage and salary employment (U.S. Economic Development Administration 2015). However, this number varies from year-to-year, and specifically was impacted due to the 2017 storms (U.S. Bureau of Labor Statistics (BLS) 2020). The other main sectors in the USVI are the public sector, manufacturing (mostly rum production), agriculture, electronics, and pharmaceuticals (Central Intelligence Agency 2020; USVI Hurricane Recovery and Resilience Task Force 2018).

1.2 Transportation Infrastructure Systems in the USVI

Each main island has a transportation system that enables the movement of people and goods across roadways, sea, and air. In general, the USVI transportation infrastructure is different from other locations in the mainland United State due to the their remote location, weather conditions, and cultural features. These differences include transportation system services, stakeholders, characteristics, and traffic patterns. The following sections describe

Figure 1.1. USVI Map of Main Three Islands. Source: Encyclopedia Britannica (2010).

these characteristics with a focus on transportation via surface roads.

1.2.1 Infrastructure Services and Users

USVI transportation systems provide mobility for people, goods, and services throughout the territory. The purpose of mobility services differ for different user types. For example, local populations that live on the islands use transportation infrastructure for daily travel within and between islands. There is significant daily inter-island travel, especially between STJ and STT for food, education, and healthcare and between STX and STT for major shipping of supplies and coordinating business across the territory. Many other infrastructure systems also depend on the functioning of transportation infrastructure. For example, water, food, and energy (e.g., fuel) services are heavily dependent on the transportation systems for shipping across the territory. Governmental and healthcare systems are dependent on transportation infrastructure to connect islands to few, centralized facilities.

Overall, most of the USVI population work in tourism, trade, government, manufacturing, farming and other services (U.S. Economic Development Administration 2015; Congressional Research Service 2020). Hence, surface roads are needed to support daily usage. There are approximately 20,000 households in both STT and STJ, and the rate of unemployment in the USVI is less than 7% (Macrotrends 2020). The number of households with two cars or fewer increased by 24% from 2000 to 2010. Around 63% of all workers commute to work alone and less then 17% use public transportation according to the 2040 Comprehensive Transportation Master Plan Report (Parsons Brinckerhoff and Jaredian Design Group 2014).

Table 1.1 presents the availability of private cars per household in the territory. Most of the people drive alone to work, with fewer carpooling, and even less using public transit (*note:* this data is pre-COVID-19). In STT, the percentage of the people who use these three main mode of transportation are 50%, 24% and 16% respectively (Parsons Brinckerhoff and Jaredian Design Group 2014).

Table 1.1. Household Vehicles in the USVI (2000 vs. 2010)

Vehicle Availability	# (2000)	[%] (2000)	# (2010)	[%] (2010)	diff
None	4970	25.5%	5000	23.3%	-2.3%
1	8910	45.8%	9920	46%	0.2%
2	4300	22.1%	4860	22.5%	0.4%
3 (or more)	1280	6.8%	1770	8.2%	1.6%
sum	19460	100	21550	100%	-

In addition to providing mobility to populations and goods within the territory, there is also significant influx of tourists who visit the islands each week. There are about 2M tourists annually, where 25% arrive by air and stay overnight and 75% arrive by cruise ship and stay during the day and sleep on the ship at night. Accordingly, tourists are the second-largest user of USVI transportation services (Alderson et al. 2018b) and the road networks of the USVI are designed to support tourism by connecting major ports of entry to commercial hubs. For example, the cruise ship dock on STT is located near downtown

Charlotte Amelie and current roadway upgrades are meant to connect the cruise ship dock to shops in the central business district. The roads on STX are designed to have additional capacity for tourist travel between Fredericksted, where the cruise ship dock is located, and Christiansted, where there is a large downtown area.

1.2.2 Infrastructure Owners, Operators, and Stakeholders

Transportation infrastructure are owned and managed by two main organizations: the Virgin Islands Port Authority (VIPA) and VIDPW. VIPA is a non-profit, semi-autonomous, servic-revenue-based organization which is in charge of managing and maintaining the ports in the USVI, including the airports, harbors, and seaports. VIDPW is in charge of the surface transportation in the USVI. Together, both organizations own and operate the majority of all transportation infrastructure in the territory. VIDPW also manages construction, engineering, and operations for the local government. VIDPW consists of four primary offices: Highway Engineering, Transportation, Engineering, and Construction.

There are several related agencies that support VIPA and VIDPW with infrastructure provision, design, maintenance, and upgrade. One key organization is the Virgin Islands Water and power authority (VIWAPA), an autonomous government public utility which is in charge of operating and maintaining the electric power and water distribution infrastructure in the USVI. VIWAPA is also a stakeholder in the territorial Internet provider, the Virgin Islands Next Generation Network. In these roles, VIWAPA is involved in the management of a significant portion of above and underground infrastructure nearby and under USVI roads. Moreover, electricity and Internet services are required for roadway and port operations across the islands.

Another significant organization is the Virgin Islands Territorial Emergency Management Agency (VITEMA). VITEMA is the territorial agency responsible for ensuring emergency response and territory's resilience to disasters. Efforts to protect recover surface roads after major disasters are organized by VITEMA and coordinated with federal agencies like FEMA.

1.2.3 Infrastructure Characteristics and Condition

Transportation infrastructure differ for surface, sea, and air transportation. Whereas sea and air transportation are primary means of accessing individual islands, surface transportation by roads dominates travel within a given island. When traveling in the USVI by road, there are several travel modes to choose from, such as personal car, a public bus system, shared-ride multi passenger taxis, open air safari and private taxis (USVI Hurricane Recovery and Resilience Task Force 2018; Parsons Brinckerhoff and Jaredian Design Group 2014; Alderson et al. 2018b). However, public transportation is not a dominant travel mode, and the majority of the population use private cars. For this reason, we focus our work on surface roads that support travel by personal car.

There are three main types of roads in the territory based on ownership and operation: federal, local, and private roads. The total mileage of each road type is 340 (federal), 410 (local) and 480 miles (private), respectively. Federal roads are usually paved with asphalt or concrete. In contrast, private roads, which serve mostly as an access to homes, are usually unpaved or semi-paved. Local roads are in between, with some paved and some unpaved. The responsibilities for road maintenance are divided between the VIDPW, which is in charge of federal roads and local roads, and the residents themselves who own the private roads (Alderson et al. 2018b)

Despite a relatively small transportation system, there is significant traffic within the territory due to roadway design and local geography. Most of the federal and public roads in the territory are two lane and without shoulders. The absence of shoulder increases the risk of traffic jam in the case of an accident, and having only two lanes (one in each direction) makes it easier for congestion and traffic to form on routes with high demand. For example, estimated travel time across STT can double depending on the traffic and the time of day.

In addition to limited roadway capacity, there are few routes available for daily traffic. Most of the major population centers are connected to the commercial centers through few primary roads. This increases the likelihood of traffic jams, especially during rush hours when populations travel from households to commercial centers, or vice versa. For example, estimated trip travel times in STX can increase in dozens of minutes (in extreme cases even more), due to single road segment blockage (Good 2019). Since STT is more dense, with roughly the same population size spread on an area that is three times smaller, there is a reason to believe that the impact of congestion or road segment blockage might be significant in STT as well. However, due to the small population and characteristic of the island, traffic does not tend to be an issue in STJ.

The topography of the territory is varied between the different islands and also affects driving conditions. STX is mostly flat, while STT and STJ are more curvy and hilly. The varied geography on STT and STJ leads to limited line-of-sight while driving and steep elevation changes that slow down routes between population and commercial centers.

Finally, some traffic issues are also caused by inexperienced drivers within the USVI. The USVI is the only place in the United States where vehicle traffic drives on the left or the "wrong" side of the road. However, USVI drivers use American-style left-hand drive cars (U.S. makes and models), which makes for a unique experience for drivers accustomed to using right-hand drive vehicles for driving on the left side of the road. With such a large influx of tourists, each week there are numerous drivers on the road that have never driven in these conditions.

Given these normal operating conditions, traffic within the USVI affects daily life across all islands. Improving the capacity and connectivity of USVI roads would reduce congestion, accidents, and promote economic development. These limitations become particularly acute during and after disasters, when roads are blocked and access to shelter and disaster relief supplies is critical for survival.

1.3 2017 Hurricanes Irma and Maria

In September 2017, two back-to-back Category-5 hurricanes (Irma and Maria) made indirect hits on the USVI within a two-week period. The first hurricane, Irma, made an indirect hit on STT and STJ on September 6, while Maria made an indirect hit on STX on September 20 (Alderson et al. 2018b; Good 2019). On the day Maria hit STX, a major disaster was declared in the territory. The hurricanes caused tremendous impacts on the territory. Measured in U.S. Dollars (USD), according the recovery and resilience task force, “Total damage is estimated at $10.7 billion: $6.9 billion to infrastructure, $2.3 billion to housing, and $1.5 billion to the economy” (USVI Hurricane Recovery and Resilience Task Force 2018).

The immediate effects on the territory were so devastating that many impacts remain up to the writing of this thesis. It caused major damage to the main infrastructure on the territory, such as blocking roads and causing blackouts (Good 2019). It affected the economy of the USVI by harming the tourism sector. The number of tourists decreased in about 30% from 2.65 million tourists in 2016 to about 1.9 million in 2018 (Travel Agent Central 2019) and the number of employees in the tourism sector decreased in by half (U.S. Bureau of Labor Statistics (BLS) 2020).

Hurricane Impact on Transportation Systems

The impact on USVI transportation systems was significant (Alderson et al. 2018b), including:

- **Shutting down airports:** territorial airports were shut down for two weeks and the number of incoming and outgoing flights has decreased dramatically after the hurricane. Within the first year after the hurricane there was a drop of 50% in booked flights seats into the USVI.
- **Car accidents and traffic control:** the hurricanes damaged many streetlights and traffic lights, leading to seven times more car accidents than usual. As for June 2020, 9 months after the hurricane hit the territory, half of the signalized intersection were still not back online, and as for August 2020, some of the signalized intersection are still yet to be fixed (Alderson et al. 2018b).
- **Road blockage and curfew:** the storms created over 600,000 tons of debris that blocked many roads including neighborhood and different supplies services that are consumed by the population (Good 2019). The storms also damaged the roads by creating mudslides or holes due to the massive rain that came with the storms. This damage led to 57 blocked intersections, mandatory curfews for 1.5 months. Overall, it took 6 months to clear debris from the roads (USVI Hurricane Recovery and Resilience Task Force 2018).

For these reasons, travel times and travel safety changed dramatically. The chance of car accidents and total travel times between population and commercial centers increased since roads were blocked, accidents made traffic slow, and traffic lights that stopped working.

One possible way to help manage these damages would be to change the transportation

Dependency	Signage/Polls	Electricity	Control System
Roundabout	V	V	V
Traffic lights	X	X	X

Figure 1.2. Signals vs. Roundabouts - Dependencies and Vulnerabilities

infrastructure to use more roundabouts. For example, there are numerous traffic signals across the territory for managing traffic flows at major intersections and there are no roundabouts as for August 2020 (at least not for managing a major intersection). Since traffic lights require above ground signage and electricity to operate, they are not robust to survive future disasters, and in particular, less robust than roundabouts. In Figure 1.2, there is an description of the different dependencies which signals have unlike roundabouts. A partial list of these dependencies would be: (1) **signage or polls** to hold physically the traffic lights; (2) **electricity** for the signals to operate; and (3) **control system** in charge of scheduling the light time cycle for each turn and entry.

1.4 Intersection Design and Disaster Impacts

Roadway intersections affect both normal system operations and disaster impacts. The two most common intersection designs use traffic signals and roundabouts. Both control traffic flow by adding rules that manage speed and dictate when and how a vehicle can change direction (i.e., turn). Generally speaking, a traffic signal uses timed lights whereas a roundabout relies on driver right of way and changing the "natural" driving path. There are many tradeoffs when choosing to design an intersection to use a roundabout or traffic signals, such as cost, feasibility, safety, and network effects. This thesis focuses on a single tradeoff for transportation systems — resilience and efficiency.

According to the hurricane recovery and resilience task force report, roundabouts might be an important alternative to traffic signals to reduce disaster impacts (USVI Hurricane Recovery and Resilience Task Force 2018). From a resilience point of view, roundabouts have the advantage of decreasing the vulnerability of failure since they do not depend on electricity, and might be less vulnerable to physical hits due to limited signage. However, from an efficiency point of view, traffic lights may be more efficient by supporting larger traffic flows across specific turns.

When examining disaster impacts, these two attributes — efficiency and resilience — combine together, since one affects the other. For example, roadway network design for efficient corridors can lead to vulnerable choke points that reduce island-wide mobility after a hurricane. Automated traffic signals can increase throughput of major intersections but also leave the intersection vulnerable to car crash when signals are broken or unavailable.

In contrast, roadway design for disaster mitigation can attenuate hurricane impacts but also reduce day-to-day efficiency. For example, a network with numerous parallel paths to re-route traffic during disaster can be difficult to operate and maintain and may have roadways that are never used on a day-to-day basis. Roundabouts may be less efficient for day-to-day traffic than automated signals, but do not require vulnerable signaling to function. Taken together, there is a trade-off between efficiency and resilience for different transportation system designs, including network connectivity, junctions,(roundabout/traffic light/ 4-sign-stop junction, etc.), and roadway materials among others that must be understood to support disaster mitigation in the USVI.

Thesis Goals

The goal of this thesis is to determine what mitigation actions can be taken to minimize the negative impact of future disasters on the USVI transportation system while supporting efficient day-to-day operation. This thesis addresses part of the broad question relevant to the USVI territory: is there a way to maximize transportation system efficiency while creating a more resilient transportation system? We answer part of this question by focusing attention on efficiency-resilience tradeoffs for intersection design (roundabout vs. traffic lights). In particular, this thesis completes the following tasks:

1. Identify methods to measure roadway operations with traffic lights and roundabouts;
2. Develop a model that allows the comparison of traffic lights and roundabouts;
3. Develop a measure of efficiency and resilience to assess intersection tradeoffs; and,
4. Assess intersection designs for a notional road network.

Completing these research tasks will help guide mitigation planning for future disasters like Hurricanes Irma and Maria. Moreover, there is limited work measuring the costs or benefits of resilience trade-offs in roadway network design. Answering this question for the USVI case will also further knowledge on the implications of efficient vs. resilient design trade-offs for interdependent critical infrastructure systems (e.g., electric power, water) that rely on transportation networks.

CHAPTER 2:
Literature Review

This chapter presents the engineering differences between roundabouts and traffic signals. There is significant literature on these types of infrastructures, and we present key models to describe the analysis process required to measure traffic in both intersections. Specifically, we review the literature on intersection design, transportation safety, infrastructure robustness, and traffic dynamics modeling for roundabouts and traffic lights. Together, this literature forms the basis of technical methods to model and assess intersection efficiency and resilience tradeoffs.

2.1 Roundabout Intersections

Roundabouts are an intersection traffic control structure designed to force vehicles to drive around a central island on a circular roadway and leave through an exit in order to make a turn (see Figure 2.1). Roundabouts have been used in the U.S. transportation system since the early 1900s. Historically, driving priority was given to vehicles that entered the roundabout. This design led to a high accident rate and led to a bad "first impression" and a lack of popularity of roundabouts in the United States.

Two main changes have made roundabouts safer since their first use. First, driving rules have changed to give priority to the vehicles inside the intersection, rather than those entering the intersection. Second, pedestrian crosswalk had moved from crossing the circulatory road into the roundabout's entries where the vehicles get inside the roundabouts (Nambisan and Parimi 2007). Around the 1990s, this "modern" and safer version of roundabout started to be implemented in the United States. Despite improved safety, roundabouts are still not popular in the United States compared to other countries, and account for only 1 in 1,200 intersections approximately (Bloomberg City Lab 2016).

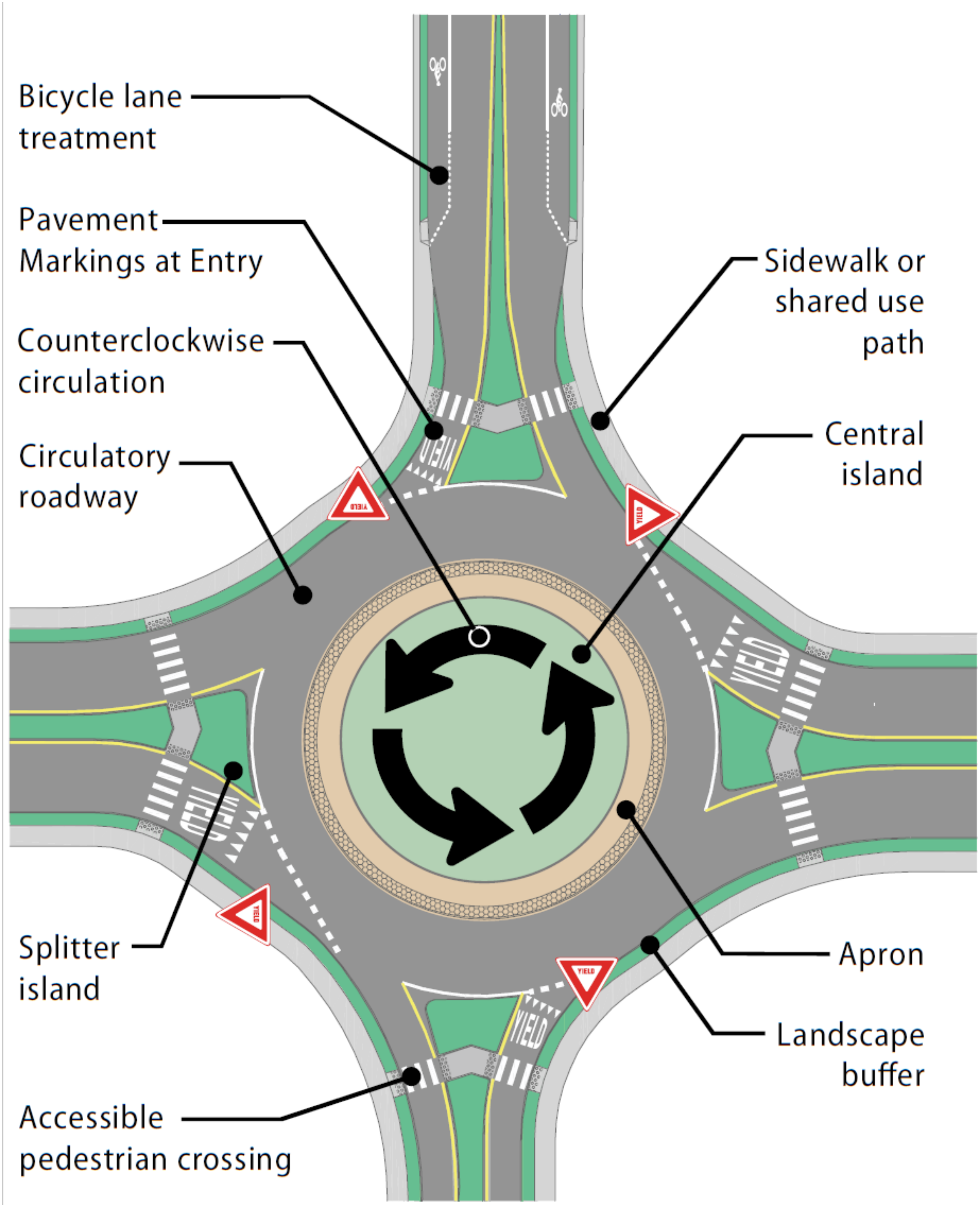

Figure 2.1. Example 4-Way Roundabout. Source: FHWA (2020).

2.1.1 Safety and Benefits

Many studies show evidence for the safety advantage of roundabouts compared to other intersections (Persaud et al. 2001; Saccomanno et al. 2008). The reasons roundabouts are safer than other traffic control structures include the number of conflict points, greater speed reductions, and ease of drive path (Nambisan and Parimi 2007).

First, there are fewer *conflict points* in a roundabout intersection when compared to signalized intersections (see Figure 2.2). Conflict points are locations in the intersection where two different paths cross each other (e.g., a vehicle and pedestrian path or two vehicle one with another). Conflict points cause accidents when drivers or pedestrians lack awareness and hit each other.

The number of conflict points in roundabout depends on its design (U.S Federal Highway Administration 2000, Chapter 5). For example, a one-lane roundabout has a conflict point for each entrance/exit (merging) into the circulatory road (four points for a four-way intersection) and another conflict point when exiting (diverging) the intersection (i.e., 8 conflict points for a one-lane roundabout in total). A roundabout would also have 8 pedestrian conflict points — two per entrance/exit. However, a two-lane roundabout would have 24 conflict points (Silva et al. 2013) and is less safe than one-lane roundabout (Eisenman et al. 2004).

Most signalized intersections have more conflict points than roundabouts (U.S Federal Highway Administration 2000, Chapter 5). A four-way traffic light intersection with three approaches — right turn, left-turn, and straight — would have 32 vehicle-to-vehicle conflict points (see Figure 2.2). Moreover, a signalized intersection has 24 pedestrian conflict points. Together, there are three to four times more conflict points when compared to a corresponding roundabout intersection.

The second way roundabouts increase safety is by forcing drivers to travel at lower absolute and relative speeds. Roundabouts decrease the speed of traffic by curving the roads on the entrance and on the exit from the roundabout. This curve slows driver approach even when there is limited traffic within the intersection. Second, roundabouts force cars to drive in the same direction, unlike traffic light where vehicles pass near each other driving opposite directions. The lower average speed combined with travel direction help drivers manage unexpected situations and decreases the severity of crashes when they occur (Nambisan and

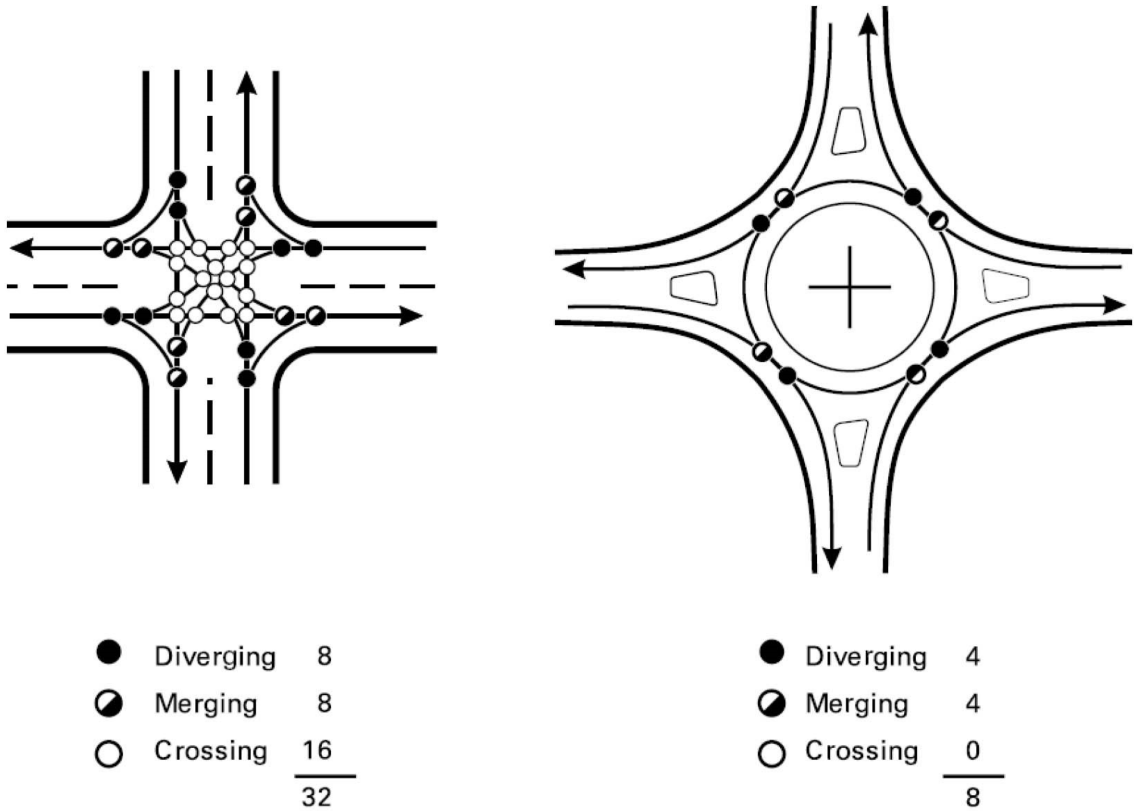

Figure 2.2. Conflict Points for Roundabouts and Signalized Intersections. Source U.S Federal Highway Administration (2000).

Parimi 2007).

Finally, roundabouts are considered easier to drive through (Nambisan and Parimi 2007). When driving through conflict points in a roundabout, a driver is only required to identify an acceptable gap for his vehicle when trying to merge in the intersection. In contrast, when driving through a traffic light, a driver must pay attention to all incoming traffic from all sides to ensure safety. This is necessary even though a green light implies priority.

Overall, roundabouts are generally considered the safest form of traffic intersection. Recently, there is a large push towards more roundabout use in the United States. For example, the Federal Highway Administration (FHWA) recommends roundabouts as an alternative transportation infrastructure design for improving safety (FHWA 2017). The growing popularity of roundabout as a safer alternative for traffic control has also improved its public opinion (Retting et al. 2002). Several U.S. cities now have policies requiring roundabouts to be considered as the first option for any new intersection design or upgrade (FHWA 2017).

2.1.2 Other Perceived Benefits of Roundabouts

Beyond safety, there are a number of other perceived benefits to roundabouts. According to the Insurance Institute for Highway Safety (2020), roundabouts provide additional benefits to traffic flow:

- "A study of three intersections in Kansas, Maryland and Nevada where roundabouts replaced stop signs found that vehicle delays were reduced 13-23 percent and the proportion of vehicles that stopped was reduced 14-37 percent (Retting et al. 2002)."
- "A study of three locations in New Hampshire, New York and Washington state where roundabouts replaced traffic signals or stop signs found an 89 percent average reduction in vehicle delays and a 56 percent average reduction in vehicle stops (Retting et al. 2006)."
- "A study of 11 intersections in Kansas found a 65 percent average reduction in delays and a 52 percent average reduction in vehicle stops after roundabouts were installed [(Russell and Mandavilli 2003)]."

These perceived benefits to traffic flow also suggest roundabouts yield improvement in fuel efficiency and emissions (Insurance Institute for Highway Safety 2020).

What remains largely unexplored is the potential benefit that roundabouts bring to robustness and resilience.

2.2 Infrastructure Resilience and Road Networks

Critical infrastructure resilience has become a topic of growing interest in the last two decades. There are several definitions of resilience across studies and disciplines (Seager et al. 2017; Park et al. 2013; Hosseini et al. 2016). Alderson et al. (2018a) describe resilience in terms of structural and operational views, where structural resilience refers to the physical ability of a system to recover or withstand a shock or a disturbance to the system like natural disaster, damage or any type of accident that might impact the system. Operational resilience refers to the plans, operations, and decisions that enable critical infrastructure systems to adapt to new operating conditions brought by large-scale disruptions. This perspective is similar to other popular definitions of resilience (Linkov et al. 2014; Ganin et al. 2017; Woods 2015). For example, Linkov et al. (2014) define resilience as "the ability of the system to maintain its demonstrated level of service or to restore itself to that level of service in a specified time frame" and Heaslip et al. (2009) tool a similar approach. However, Pant (2012) defines it as the "ability of transportation system to retain performance during and after disasters undergoing little to no loss and their ability to return to normal state." Similarly, Freckleton et al. (2012) defines it as the ability for a transportation network to absorb disruptive and restore the performance level (level of service) within reasonable time frame.

Resilience of transportation systems is broadly studied by considering the vulnerability of infrastructure to disasters, like floods, hurricanes, and earthquakes. Markolf et al. (2019) discuss transportation system vulnerabilities to natural disasters with a focus on future climate-related events. Transportation system resilience depends on failure response to a given hazard (e.g., pavement materials in extreme heat or flooding) in addition to interdependencies with other infrastructure systems that provide electricity, water, and fuel. Failures can cascade within transportation systems and across critical infrastructure systems to impact mobility and inhibit access to key resources like disaster supplies and infrastructure assets. These disruptions may become more frequent and larger in future climate scenarios. Thus, considering the resilience of transportation systems is an important aspect of design that impacts current and future infrastructure.

Resilience of road networks is often studied by measuring the total vehicle travel time for all vehicles across a metropolitan area given network disruptions and failures. For example, Alderson et al. (2018a) study operational resilience in large-scale transportation systems by modeling travel time across regional highway infrastructure in the San Francisco Bay Area. Alderson et al. (2018a) develop a multi-commodity flow model that determines traffic flows and roadway capacities during rush hour traffic. The model assesses resilience by measuring increased travel times given bridge and highway failures. The authors determine operational resilience by assessing system performance given worst-case failure of components, and identify key bridges and highway segments that, if failed, increase traffic the most. Alderson et al. (2018a) recommend system protection that prioritizes these critical road segments to reduce the impacts of worst-case failures.

2.2.1 Resilience and Efficiency tradeoff

As total travel time is a measure of road network efficiency, Ganin et al. (2017) show that one of the key issues with improving roadway resilience are tradeoffs in efficiency and robustness. Ganin et al. (2017) classify various metropolitan areas in the United States as either efficient or robust by measuring increased travel time given large-scale network failures (via network percolation). The authors identify several urban areas such as New York and San Francisco and analyze their vulnerability to failures in terms of increased travel time after loss of roads. The authors identify road networks that are robust to failures (resilient), but inefficient (e.g., Los Angeles, CA) and the converse which is road networks that are vulnerable to failure (e.g., San Francisco, CA). These measures provide a way to identify road networks that are both efficient and resilient (e.g., Cleveland, OH) and provide a means to provide recommendations for new network designs.

2.2.2 Intersections and Resilience

Despite significant popularity in infrastructure resilience research, there is limited work measuring the benefits or limitations of new system designs. Markolf et al. (2019), Alderson et al. (2018a), and Ganin et al. (2017) discuss transportation vulnerabilities, resilience, and tradeoffs, yet all studies center on the effects of roadway disruptions without measuring the effects of engineering design. For example, while Markolf et al. (2019) discuss the different effects of disasters on transportation infrastructure, recommendations for hardening systems are generic and not measured or applied in context. Alderson et al. (2018a) and Ganin et al. (2017) consider disruptions to road networks in real metro areas, but both only consider limited design options for managing failures (e.g., adding a new road, changing link capacity). Importantly, all studies tend to focus on the network structure via link connectivity, rather than the roadway control structures that impact driver actions at intersections and on ramps.

We focus attention on the effects of *intersection design* on roadway resilience. As described, roundabouts have characteristically different design than traffic lights that results in different driver behavior and increased safety. In addition, roundabouts may have resilience benefits as well by being more robust to disasters. Roundabouts are more physically robust than traffic lights as there are fewer signs, lights, or control structures required to operate a roundabout. This means that there are fewer components that can be destroyed in a disaster than affect mobility. Moreover, roundabouts have fewer interdependencies with others infrastructure systems, reducing the likelihood that failures in one system impact mobility (see Figure 1.2). For example, roundabouts do not require electricity to operate, but traffic lights do. In a place like the USVI where hurricane storms are likely to occur and cause blackouts, the ability of the intersection to function regardless power supply might become important. In 1.3, the impacts of hurricanes Irma and Maria on transportation systems included failure of intersections control devices, leading to increased accidents and reduced mobility.

While roundabouts may be safer and more robust to natural disasters, they may be less efficient depending on mobility needs. Roundabouts slow traffic in all directions, leading to decreased speeds even when there is limited merging traffic. Traffic lights may be more efficient in networks with few intersections with merging traffic or major turns (e.g., the majority of traffic is traveling in one direction). The travel time benefits of traffic lights suggest that there is an efficiency-resilience tradeoff in intersection design that has yet to be discussed in the literature. Moreover, this tradeoff is relevant when comparing different roundabouts to each other. For example, a one-lane roundabout is safer than two-lane roundabout (Eisenman et al. 2004), but the capacity of two-lane roundabout is greater. Measuring these tradeoffs requires detailed engineering models of each intersection type that enable comparison of roundabouts and traffic lights.

2.3 Roundabout Models and Traffic Flows

The following section describes technical approaches for modeling traffic and travel times for roundabout intersections. This section overviews the required parameters to model roundabout design and traffic flows, their physical relationships, and measures of travel time incurred by driving through the roundabout.

Before we present engineering models, we introduce terminology that informs intersection modeling. Some of these terms are also useful for modeling signalized intersections with traffic lights. We use the same notation for signalized intersections where terminology and physics remain the same.

In particular, roundabout modeling requires technical terminology for intersection structure and traffic flows. The key intersection structures that impact vehicle flows through roundabouts and their corresponding terms are the following.

- **Intersection Legs** – The approaches and exits the lead into the roundabout (usually three or four).
- **Central Islands** – A round traffic control structure which the vehicles drive around.
- **Circulatory Roads** – The curved road within a roundabout intersection that surrounds the central island.

These basic roundabout terms and other are described visually in Figure 2.1.

The key parameters that need to be measured or modeled to determine the effects of roundabouts on traffic flows are the following.

- **Entering Flow** – The flow of cars approaching the intersection. The entering flow rate is often different for different legs. The total flow of vehicles is the sum of entering flow over all intersection legs.
- **Circulating Flow** – The flow of cars driving on the circulatory road that block intersection entry. Circulating flow is calculated for each intersection leg separately and is a portion of total flow through the intersection.
- **Intersection Capacity** – Similar to road capacity, the intersection capacity is the maximum rate of vehicles per time that the intersection is designed to service. Once the number of vehicles trying to use the intersection exceeds the capacity, the delay time starts increasing non linearly.
- **Saturation** – An intersection is considered saturated if total entering flow is equal to designed capacity. An intersection is undersaturated if there is less flow than total capacity, or oversaturated if entry flows exceed capacity.
- **Passenger Car Units (pcu)** – A unit which is used to describe the flow of different vehicles (e.g., cars, trucks, motorcycles) on a normalized scale: pcu is measured as passenger vehicles per unit time, such as [pcu/h]. For example, a motorcycle is often measured as 0.75 pcu and a commercial truck is 3 pcu. In most cases, the units [pcu/h] and [veh/h] are used interchangeably.

2.3.1 Roundabout Traffic Flows

The goal of intersection modeling for roundabouts and traffic lights is to determine how the design supports flows of vehicles. The key variable determining how much time a vehicle will take to travel through an intersection is the total number of vehicles entering and exiting the intersection. For a roundabout, there are three flows that must be measured to determine travel time: (1) entry flow, (2) origin-destinations (O/D) over the intersection, and (3) circulating flows that block entry of more vehicles into the intersection.

Calculating these parameters is possible in three steps outlined in Mauro (2010). The first step is to define an entry flows vector Q_e and the origin-destination matrix $P_{O/D}$ for each flow within the intersection. Here, Q_e is a vector with an element $Q_{e,i} \forall i \in E$ where E is the set of all entries at the intersection and $Q_{e,i}$ is the flow of vehicles entering the intersection at entry i in pcu. $P_{O/D}$ is the matrix comprised of elements $P_{ij} \forall i \in E, \forall j \in L$ where X is the set of all exits from an intersection and element P_{ij} is the ratio of cars that enter at entry i that are exiting at exit j. An intersection where all legs accommodate two-way traffic (i.e., have entries and exits) has $E = X$ and $P_{O/D}$ is a square matrix. For example, Q_e and $P_{O/D}$ for a four-way intersection with two-way traffic are:

$$Q_e = \begin{bmatrix} Q_{e1} & Q_{e2} & Q_{e3} & Q_{e4} \end{bmatrix} \tag{2.1}$$

$$P_{O/D} = \begin{bmatrix} P_{11} & P_{12} & P_{13} & P_{14} \\ P_{21} & P_{22} & P_{23} & P_{24} \\ P_{31} & P_{32} & P_{33} & P_{34} \\ P_{41} & P_{42} & P_{43} & P_{44} \end{bmatrix}. \tag{2.2}$$

The second step outlined by Mauro (2010) is to convert $P_{O/D}$ ratios into flows and obtain the origin-destination flow matrix, $M_{O/D}$. $M_{O/D}$ is similar to $P_{O/D}$, but each element represents the $Q_{i,j}$ flow originating at entry i and exiting the intersection at j. $M_{O/D}$ is easily obtained by element-wise matrix multiplication, $M_{O/D} = Q_i \cdot P_{ij}$. For a four-way intersection with two-way traffic, the $M_{O/D}$ matrix would be:

$$M_{O/D} = \begin{bmatrix} Q_{11} & Q_{12} & Q_{13} & Q_{14} \\ Q_{21} & Q_{22} & Q_{23} & Q_{24} \\ Q_{31} & Q_{32} & Q_{33} & Q_{34} \\ Q_{41} & Q_{42} & Q_{43} & Q_{44} \end{bmatrix}. \tag{2.3}$$

The final step presented in Mauro (2010) is to calculate the circulating flow within the roundabout. Figure 2.3 provides a visual representation of circulating flow. As depicted in Figure 2.3, circulating flow for a given entry depends on the turns taken by cars at other entries. For example, the red line represents circulating flow in front of Leg 1 caused by a flow originating from Leg 3 and exiting Leg 4 (right turn, according to USVI left-hand driving rules). In contrast, flows originating from Leg 4 and exiting Leg 2 (blue line, driving straight) do not interfere with the depicted flow.

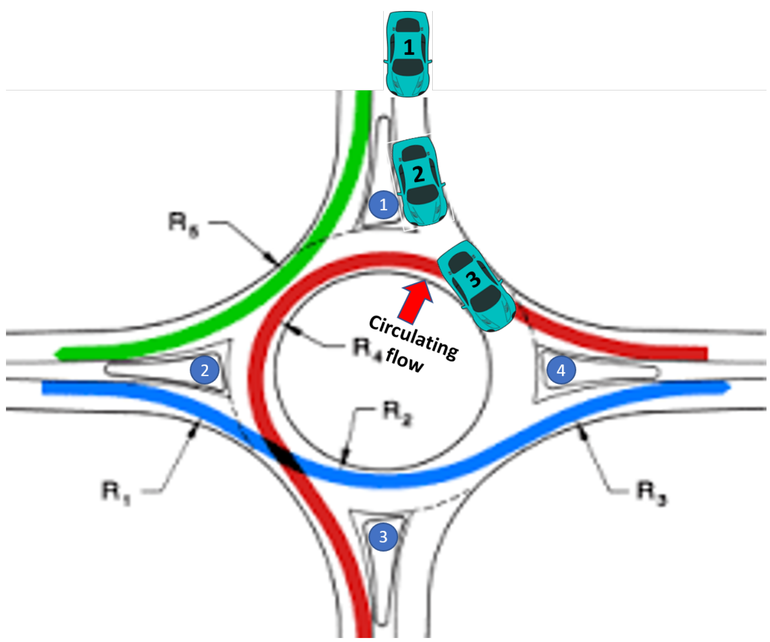

Figure 2.3. Roundabout Turns and Circulating Flows. One of the main factors for travel time in a roundabout is circulating flow that blocks entry into the intersection. This flow is a function of the number of cars entering other parts of the intersection that make turns and block a given entry. For example, using USVI driving and turning rules (similar side to the United Kingdom), cars entering from Leg 1 will be blocked by circulating flow originating from Leg 3 that exits at Leg 4 (red line, right turn). With U.S. driving rules, cars entering at Leg 1 and exiting at Leg 2 (green line) would be blocked by circulating flow originating at Leg 4 and existing at Leg 3 (red line, left turn). Thus, circulating flow is calculated for each entry separately and depends on driving rules. Adapted from Transportation Research Board (2010).

Thus, circulating flow is obtained separately for each entry Q_{ci} as the sum of vehicles that will block the entry. Each Q_{ci} is a function of the elements in the $M_{O/D}$ matrix and the legal turns within the intersection. For example, circulating flow for a the four-way intersection depicted in Figure 2.3 using USVI driving and turning directions would be:

$$Q_{c1} = Q_{24} + Q_{34} + Q_{23} \tag{2.4}$$
$$Q_{c2} = Q_{31} + Q_{41} + Q_{34} \tag{2.5}$$
$$Q_{c3} = Q_{42} + Q_{12} + Q_{41} \tag{2.6}$$
$$Q_{c4} = Q_{13} + Q_{23} + Q_{12} \tag{2.7}$$

Note: We use clockwise driving directions when calculating Q_c with Equations 2.4 to 2.7 to match left-hand traffic as practiced in the USVI. It is possible to convert Q_c for right-hand traffic as practiced in the rest of the United States by switching index order of ij entry-exit pairs.

2.3.2 Roundabout Capacity

There are several models for estimating roundabout capacity. Roundabout capacity, C_i, is total traffic that can travel through a given entry i over a given time period. Capacity is difficult to estimate because it is a function of intersection configuration, flows, geometric design, and user behavior. Different approaches to capacity measurement vary by the parameters they consider.

Table 2.1 presents an overview of several models used for estimating roundabout capacity. Roundabout capacity models have three important characteristics, including the countries that use them for transportation analysis, the model inputs required to assess capacity, and the relationships between inputs. For example, Mauro (2010) organize capacity models based on model inputs:

1. models that emphasize roundabout configuration (e.g., roundabout lanes and legs);
2. models that emphasize roundabout geometry (e.g., lane size); and
3. models that emphasize user behaviour (e.g., traffic flows).

To demonstrate the differences in each model, we provide detail on the Brilon-Bondzio, Transport and Road Research Laboratory (TRRL), and Highway Capacity Manual (HCM) 2000 models.

Models	Country Used	Inputs	Relationship	Reference
Brilon-Bondzio	Germany	Configuration	Linear	Brilon et al. (1997)
Bovy et al.	Switzerland	Configuration	Linear	Brilon (2012)
TRRL	United Kingdom	Configuration, Geometric Design	Linear	TRRL (1980)
CETEC	France	Configuration, Geometric Design, User Behavior	Non-linear	Louah (1992)
Brilon-Wu	Germany	Configuration, User Behavior	Non-linear	für Straßen et al. (2006)
HCM 2000	USA	Configuration, User Behavior	Exponential	TRB (2000)

Table 2.1. Common Roundabout Modeling Approaches. Adapted from Mauro (2010). TRRL: Transport and Road Research Laboratory; CETEC: Société d'études techniques et économiques; HCM: Highway Capacity Manual.

Brilon-Bondzio — Configuration Modeling

Brilon et al. (1997) developed a linear model for roundabout capacity which takes into account only the configuration of the roundabout. This model assumes the capacity is based on two constants A and B that result from a linear fit to real roundabout capacities when compared to the number of leg-lanes and circle lanes in the intersection. The model uses the circulating flow Q_c [pcu/hr] as an independent variable (Brilon et al. 1997). Unlike other models that take into account geometry and user behavior that measure capacity separately for each intersection leg, this model assumes the capacity is the same for all legs. The relationship between the capacity and these parameters is given in Equation 2.8, while the coefficient A and B are given in Table 2.2.

$$C = A - B \cdot Q_c \tag{2.8}$$

Table 2.2. Parameter Setting for Brilon-Bondzio Model

Circle lane number	Entry lane number	A	B	Sample size
3	2	1409	0.42	295
2	2	1380	0.5	4574
2-3	1	1250	0.53	879
1	1	1218	0.74	1504

TRRL — Geometry Modeling

The TRRL model was developed in United Kingdom in 1980 and takes as input both the roundabout configuration and geometry. When using the geometry parameters of the roundabout, detailed parameters are required. Table 2.3 presents some of the parameters considered by the TRRL model.

The parameters in Table 2.3 are presented to show the significant difference in detail between configuration and geometry models. However, for brevity, we do not reproduce the entire TRRL model here. We refer the reader to Kimber (1980) for details on the model and all geometric parameters necessary for measuring capacity.

Table 2.3. Parameters for the TRRL Model. Source: Mauro (2010).

Parameter	Description	Typical Values
e	Entry width	3-16 [m]
v	Lane width	2-12 [m]
e'	Previous entry width	3-15 [m]
v'	Previous lane width	3-12 [m]
u	circle width	5-22 [m]
l	Flare mean length	1 < x [m]
S	Sharpness of the flare	0-9
r	Entry bend radius	3 < x [m]
Φ	Entry angle	0-77 [deg]
D	Inscribed circle diameter	13-171 [m]
W	Exchange section width	7 - 26 [m]
L	Exchange section length	9 - 86 [m]

HCM 2000 — User Behavior Modeling

The HCM is a publication of the Transportation Research Board (TRB) of the U.S. National Academies of Science. The HCM provides details for modeling many different aspects of road networks in addition to roundabout capacity. The HCM roundabout capacity model is a non-linear function that relates circulating flow and the time required to merge into the intersection (Transportation Research Board 2000):

$$C_i = \frac{Q_{ci} \exp\left(\frac{-Q_{ci} T_c}{3600}\right)}{1 - \exp\left(\frac{-Q_{ci} T_f}{3600}\right)} \tag{2.9}$$

where Q_{ci} is the circulating flow blocking entry i, T_c is the critical time gap required between cars on the circular road to allow a car to merge (in seconds), and T_f is the follow-up time it takes for a car to get into the service line when merging is possible (in seconds). Although the HCM roundabout capacity model is widely adopted in the United States, it is limited to assess capacity for single lane entries and circular roads and for $Q_{ci} \leq 1200$ pcu/h (Transportation Research Board 2000; Mauro 2010).

Research has identified an appropriate range of values for T_c and T_f (Troutbeck 1992). Here, high values T_c and T_f estimate flows for more conservative driving patterns, where lower values reprsent more aggresive driving patterns. Troutbeck (1992) provides an upper bound for U.S. driving conditions as $T_c = 4.1$, and $T_f = 2.6$ s and a lower bound as $T_c = 4.6$ and $T_f = 3.1$ s.

2.3.3 Roundabout Time Delay

The ultimate goal of determining roundabout flows and capacities is to estimate the travel time required to traverse the intersection. Thus, the time delay of an intersection is a key measure of efficiency that determine its effect on driver mobility, routing, and safety.

The time it takes to traverse a roundabout is a result of three different driving speeds and travel times (U.S Federal Highway Administration 2000):

1. **Approaching Speed:** The first time delay created by a roundabout is due to cars slowing down to approach the intersection. This speed is usually slower than the unrestricted speed limit on the road leading to the roundabout. The factors that affect approaching speed are yielding requirements at the intersection, and the need to stop

in case there is a car inside the circular road, and meeting the negotiation speed of the circular road.

2. **System (Control) Delay Time:** The second time delay is due to waiting in a queue to enter the roundabout, including the time inside the queue line and waiting at the roundabout entrance merge onto the circular road (service time). The combined delay time for these two parts is also called *system* or *control* delay. This time delay depends on the circulating flow.
3. **Circulating Speed:** The third time delay is due to the geometry and curving shape of the circulatory road. This part includes driving the circulatory at a lower speed compared to the posted speed limits .

In general, transportation modeling assumes that time delays caused by approaching speed are included in models for travel time over road segments, and can be ignored when measuring the travel time over an intersection. Moreover, the circulating speed is assumed constant for all cars within the roundabout for a given flow condition. Thus, the most important factor dictating intersection effects on traffic is the system delay time.

The U.S Federal Highway Administration (2000, Chapter 4) and Akcelik and Troutbeck (1991) developed an equation to estimate the system delay time by relating entry flow rates and capacities.

$$D_i = \frac{3600}{C_i} + 900T \cdot \left(\frac{Q_{ei}}{C_i} - 1 + \sqrt{\left(\frac{Q_{ei}}{C_i} - 1\right)^2 + \frac{\left(\frac{3600}{C_i}\right) \cdot \left(\frac{Q_{ei}}{C_i}\right)}{450T}} \right), \tag{2.10}$$

where:

D_i = average control delay per vehicle which goes through the roundabout at entry i [s/pcu];
Q_{ei} = flow rate for entry i of the roundabout [pcu/h];
C_i = capacity of entry i [pcu/h]; and,
T = analysis time period [hour].

Time Delay and Saturation

Equation 2.10 is used to measure travel time for each entry separately. Thus, it is most appropriate to use Equation 2.10 with the HCM 2000 model for roundabout capacity and when the intersection is under or near saturation (i.e., $Q_{ei} < C_i; \forall i \in E$). In general, it is recommended to use Equation 2.10 only when,

$$Q_{ei} \leq 0.85 \cdot C_i \quad \forall i \in E. \tag{2.11}$$

However, the equation might hold in the near and oversaturation situations, but would be less accurate. In oversaturation situations, it is recommend to include residual queues, metering effects and upstream and downstream entries (U.S Federal Highway Administration 2000, Chapter 4). The metering effect occurs due to smaller circulating flow in front of the downstream entries and therefore increased capacity of these legs.

2.3.4 Steady State Flow Conditions and Saturation

Real traffic flows over intersections are intermittent with punctuated times of high or low flow over a given entry. Since all roundabout models are based on flow rates in pcu/hr, calculation of entering and circulating flow requires the identification of *steady state flow conditions* (Morse 1982). Steady state conditions are based on a minimum time interval that is long enough so traffic over the interval is assumed constant. Thus, with steady state flows, all flows across all entries can be assumed to occur simultaneously and related flow rates Q_{ei} become linear additive across all entries.

A roundabout is considered to be in a steady state when its operation and design meets two criteria presented in Mauro (2010): (1) The entering traffic flow into each of the roundabout legs is fixed and doesn't change with time and (2) the average queue lengths and waiting times when approaching to the roundabout are stable and not indefinitely increasing with time (i.e., the intersection is not oversaturated).

Using these criteria, we define three steady state conditions that must be met to analyze roundabout flows. Specifically, Morse (1982) define a characteristic time interval, Δt, for which flows Q_e and Origin-Destination ratios $P_{O/D}$ do not change.

$$Q_e(t+\Delta t) = Q_e(t) \qquad \Delta t > T_{ss} \tag{2.12}$$

$$P_{O/D}(t+\Delta t) = P_{O/D}(t) \quad \Delta t > T_{ss} \tag{2.13}$$

$$Q_{ei}(t) < C_i(t) \qquad \forall\, i; i \in \text{Roundabout entries} \tag{2.14}$$

$T_{s.s}$ is the minimum time required for steady state conditions to hold for a given intersection and $C_i(t)$ is the time-dependent capacity for entry i. It is important to note that these conditions only hold for undersaturated roundabouts.

In order to satisfy Equations 2.12 and 2.13, one must first calculate T_{ss}. Morse (1982) presents a way to calculate the minimum value for T_{ss} as a function of entry capacity, C_i, and entering flow Q_{ei}.

$$T_{ss}[sec] \; > \; \max\left(\frac{1}{\left(\sqrt{\frac{C_i}{3600}} - \sqrt{\frac{Q_{ei}}{3600}}\right)^2}\right) \quad \forall i \in E \tag{2.15}$$

Equation 2.15 indicates that the smaller the difference between the entry capacity and the entry flow, the greater T_{ss} and the greater Δt must be for steady state conditions to hold. For example, Table 2.4 presents the steady state times for entry to a roundabout with capacity $C_1 = 1000$ pcu/hr with different demands. Here, the roundabout with entry flow $Q_{e1} = 900$ pcu/hr has a $T_{ss} \approx 22.8$ min. In contrast, with $Q_{e1} = 600$ with the same capacity, $T_{ss} \approx 1.2$ min.

Table 2.4. Time Intervals for Steady State Conditions (C_i = 1000 pcu/hr)

	1	2	3	4	5	6	7	8	9
Entry Flow (Q_e)	100	200	300	400	50	600	700	800	900
$T_{ss}[min]$	0.12	0.20	0.30	0.44	0.70	1.2	2.25	5.38	22.78

2.4 Signalized Intersection Models and Traffic Flows

Similar to roundabout modeling, there are many approaches for modeling signalized intersections. In this section we describe one popular approach described in the Transportation Research Board (2000). We divide the process of assessing the efficiency of a signalized intersection into two parts: (1) measuring intersection capacity, and (2) calculating the estimated average time delay created by the intersection. Unless otherwise stated, all factors for traffic, flows, capacities, and time described in this section use the same units as in Section 2.3.

2.4.1 Signalized Intersection Capacity

Unlike roundabouts that have circular flow, signalized intersections rely on setting lane priority via green lights. This approach to traffic control simplifies the analysis of traffic within the intersection such that traffic flows Q_{ei} are independent of each other and there is no need to calculate an equivalent Q_{ci}.

Instead, signalized intersection capacity depends on independent lane groups and signal scheduling, rather than flows within the intersection. For example, if the left turn and the straight lane have different signal timing, their capacity should be calculated separately since they have different green light cycles. Thus, the two key factors for measuring intersection capacity and time delay are *saturation flow rate* s_i for a given lane group $i \in G$, and the *effective green ratio* for the lane group ($\frac{g_i}{t_i}$).

Saturation Flow Rate

The saturation flow rate is the vehicle flow rate in pcu/h that the intersection can serve assuming that the light is green all the time. Another way to think about the saturation flow rate is the capacity of the road ignoring the traffic light. The saturation rate has a basic value of s_0 (basic saturation) which is usually set to be 1900 pcu/h (Transportation Research Board 2000, Ch. 16). Equation 2.16 describes the relationship between the basic saturation and the saturation flow rate of the lane group given a large number of geometric factors (see Table 2.5.

$$s_i = s_0 N f_w f_{HV} f_g f_p f_{bb} f_a f_{LU} f_{LT} f_{RT} f_{Lpb} f_{Rpb} \tag{2.16}$$

Table 2.5. Parameters for Saturation Flow Calculation

Factor	Description
s_i	saturation flow rate for lane group i
s_0	base saturation flow rate pcu/h ; default - 1900
N	number of lanes in lane group
f_w	adjustment factor for lane width
F_{HV}	adjustment factor for heavy vehicle
f_g	adjustment factor for approach grade
f_p	adjustment factor for existence of a parking
f_{bb}	adjustment factor for blocking affect of local buses
f_a	adjustment factor for area type
LU	adjustment factor for lane utilization
f_{LT}	adjustment factor for left turn
f_{RT}	adjustment factor for right turn
f_{Lpb}	adjustment factor for pedestrians-bicycle for left turn movements
f_{Rpb}	adjustment factor for pedestrians-bicycle for right turn movements

As shown in Equation 2.16 and Table 2.5, there are many parameters to consider when determine the saturation flow rate of an lane group feeding a signalized intersection. However, for simplicity we do not provide a full description of each parameter. For a detailed explanation of each parameter and a range of normal values, we direct the reader to Transportation Research Board (2000, Ch. 16).

Capacity Calculation

We can obtain the lane group capacity by relating the saturation flow rate to the green light ratio out of the signal cycle (Transportation Research Board 2000, Ch. 16):

$$C_i = s_i \cdot \frac{g_i}{t_i} \tag{2.17}$$

where:
C_i = capacity of lane group i;
s_i = saturation flow rate for lane group i from Equation 2.16;
g_i = the green light time for lane group i a signal cycle (in seconds);
t_i = Time duration of signal cycle (in seconds).

2.4.2 Signalized Intersection Time Delay

We can use lane group capacity and green light timing with other road and adjustment factors to estimate the time delay incurred for traversing a signalized intersection (Transportation Research Board 2000). This time delay is related to the efficiency of the intersection, where lower delays relate to faster mobility and more efficient road networks. Also similar to roundabouts, there are three delay factors that determine how long it takes for a vehicle to travel through the intersection (Transportation Research Board 2000):

$$d_i = d_i^1 \cdot PR + d_i^2 + d_i^3 \tag{2.18}$$

where:
d_i = total delay time per vehicle [s/pcu] for a given lane group i,
d_i^1 = uniform delay for intersection controls (assuming uniform arrivals) [s/pcu],
PR = progression factor based on the arrival type of the vehicles,
d_i^2 = incremental delay for random arrivals and over-saturation scenarios; and,
d_i^3 = queue delay in case the analyzed case starts with queue.

We present methods developed by the Transportation Research Board (2000) to determine each factor for Equation 2.18

The **uniform delay** d_i^1 for a given lane group i is the time delay relating the type of the vehicles approaching the intersection and on the lane group signal parameters:

$$d_i^1 = \frac{0.5t_i\left(1 - \frac{g_i}{t_i}\right)^2}{1 - \left[min(1, X_i) \cdot \frac{g_i}{t_i}\right]} \tag{2.19}$$

where $X_i = \frac{Q_{ei}}{C_i}$ is the flow capacity ratio for a given lane group i.

The **incremental delay** d_i^2 is the time delay caused by random arrivals due to fluctuations in the uniform arrivals with uniform delay. It also takes into account the impact of over-saturation impact on the delay time if that is the examined case.

$$d_i^2 = 900T\left[(X_i - 1) + \sqrt{(X_i - 1)^2 + \frac{8klX_i}{C_iT}}\right] [s/\text{pcu}] \tag{2.20}$$

where:
$X_i = \frac{Q_{ei}}{C_i}$ = flow capacity ratio for lane group i;
k = incremental delay factor that is dependent on signal controller settings. For pre-timed signals, a value of $k = 0.5$ is used;
l = upstream filtering/metering adjustment factor. For isolated intersections, $l = 1$;
C_i = lane group capacity [pcu/h];
T = duration of analysis [hours]; and,
Q_{ei} = vehicle flow for lane group i [pcu/hour].

More details on how to assign k and l, the incremental delay factor and upstream filtering adjustment factor, are in provided in Transportation Research Board (2000, Chapter 16).

The queue delay, d_i^3 is for queues left over from previous time periods. d_i^3 is only used for analysis for multiple time periods and is not used further in this work. In addition, the process of obtaining the adjustment factor PR requires significant analysis. For simplicity purposes, we direct the reader to Transportation Research Board (2000) for more detail on d_i^3 and PR.

2.5 Literature Summary

This literature review reveals tradeoffs for intersection design and resilience that have yet to be studied. After review of literature on roundabouts, resilience, and intersection modeling, it appears that roundabouts may be more safe and resilient than traffic lights, but less efficient with respect to travel time. In contrast, traffic lights may be more efficient than roundabouts with respect to travel time, but may lack resilience. This suggests that there is an efficiency-resilience tradeoff in intersection design that has yet to be discussed in the literature. We identify well-established literature for modeling roundabouts and signalized intersections that can support assessing this tradeoff. In the next chapter, we present a structured method for integrating these perspectives and models for road network analysis that might help in intersection design process and serve as basis for future analysis.

CHAPTER 3:
Methods

The goal of this thesis is to explore efficiency-resilience tradeoffs for road network design in the USVI. In this chapter we develop methods for measuring the advantages of different road intersections during normal traffic and after a disaster (Alderson et al. 2018a). Based on the effects of Hurricanes Irma and Maria presented in Chapter 1 and the literature review in Chapter 2, we hypothesize that there is a tradeoff between efficient intersections that reduce travel times and resilient intersections that ensure driver safety and mobility after a disaster. Our proposed method to explore these tradeoffs involves the following computational and analysis steps.

1. **Implement Existing Network Model:** We present an existing transportation model for the USVI developed by Good (2019) to provide a technical basis for managing non-linear equations in road network analysis.
2. **Intersection Modeling:** We determine how to implement existing roundabout and signalized intersection models in a way that extends the Good (2019) model.
3. **Define Efficiency-Resilience Measure:** We define measures for intersection efficiency and resilience to assess tradeoffs.
4. **Obtain Measures performance of Examined Networks:** We compare intersections and their effects on efficiency and resilience.

3.1 USVI Transportation Model from Good (2019)

We build upon past models of the USVI surface road networks used to assess mobility and network capacity after disasters. Specifically, we build on a multi-commodity network flow model developed by Good (2019) and extended by Routley (2020).

We use this model to show how to represent road network model primitives and determine the best way to model intersections that have non-linear capacities and time delays in flow networks. As shown in Chapter 2, the equations for both roundabout and signalized intersections are non-linear and have multiple parameter inputs. These equations are similar to those used by Good (2019) to assess congestion and traffic delays over road segments in-between intersections. By presenting this existing model, we demonstrate methods for estimating the effects of non-linear traffic delays in network flow models and determine ways to include intersection time delays in multi-commodity flow models.

3.1.1 Model Purpose

The purpose of the model in Good (2019) is to measure the capacity of surface roads in the USVI to support access to critical supplies after a major disaster. The model is a multi-commodity network flow model that measures travel time for all communities in the USVI to reach locations of critical supplies (e.g., food, water, ice, gasoline, etc.). The network performance is measured as the total travel time of all the vehicles in (vehicle-hours), which serves as the model objective function we want to minimize. By minimizing travel time, the model develops a Dynamic Traffic Assignment (DTA) that simultaneously determines driver routes and travel times. This model was recently updated to consider new road types and the effects of flooding disasters (Routley 2020).

The model in Good (2019) was developed to consider disaster scenarios and their effects on traffic. Scenarios are chosen to assess the failure of major roads during 6-hour curfew periods similar to those USVI communities experienced after Hurricanes Irma and Maria. The inclusion of a travel window acts as an added penalty to the objective function for drivers that cannot reach critical supplies within the curfew window and become dropped flow. Thus, the model balances whether USVI communities can reach supplies, or if travel times will be so large that they would be better off staying home. Good (2019) developed scenarios and applied this model to STX, where Routley (2020) conducts similar assessments of STT and STJ.

3.1.2 Model Inputs

The model in Good (2019) uses three primary inputs to perform DTA:

1. network structure and component composed of arcs and nodes,
2. route choice for origin-destination pairs, and
3. road parameters that dictate road capacities and congestion-based travel time.

The first model input is the road network. The network is composed of arcs and nodes.

While all the arcs represent road segments, there are three different types of nodes: *origin nodes* that either populations or supplies originate at such as population centers and ports, *destination nodes* that provide essential daily services, and *transshipment nodes* which act as intersections connecting road segments.

Network flow is driven by allocating origin-destination requirements at the corresponding nodes. According to objective function, the model minimizes the total vehicle-hours given routing processes for assigning origin-destination pairs. Good (2019) considers two methods to assign origin-destination pairs: (1) *selfish routing* where households travel to the destination nearest to their community based on geographic distance, and (2) *coordinated routing* where households will travel to the destination that optimally reduces travel time for the entire island.

The final input is the road parameters assigned to each network arc. These parameters are the capacity, unrestricted speed, and the length of each arc in the network.

Altogether, network structure, routing, and road capacities enable optimal travel time calculation for each origin-destination pair, community, and roadway. Building on this model, the methods developed herein apply most appropriately to the intersections as to transshipment nodes in Good (2019). However, the model structure does not yet afford intersection nodes to include travel time calculations. We build on methods applied to road segments to develop an approach that can be integrated into the model.

3.1.3 Travel Time Calculation

Similar to roundabout and signalized intersection models, roadway traffic has non-linear relationships that determine total travel time. Good (2019) and Routley (2020) consider only these effects on the road segments and not the transshipment nodes (intersections). Specifically, the model in Good (2019) uses the Bureau of Public Roads (BPR) function to obtain the relationship between the travel time across a road segment as a function of flow and road capacity ratio (Peeta et al. 2015). The BPR function used in both Good (2019) and Routley (2020) is

$$f_{ij} = \frac{d_{ij}}{s_{ij}} \left(1 + 0.15 \left(\frac{Y_{ij}}{u_{ij}}\right)^4\right) \quad \forall (i,j) \in A \subseteq N \times N, \tag{3.1}$$

where:
A is the set of (i, j) arc pairs;
N is the set of all nodes;
f_{ij} = travel time per vehicle to traverse road arc ij [h];
d_{ij} = road length [miles];
s_{ij} = road unrestricted speed [mi/h];
Y_{ij} = road car flow [veh/h]; and,
u_{ij} = road capacity [veh/h].

We plot Equation 3.1 in Figure 3.1 to show its non-linear form and its relationship to travel time. Shown in Figure 3.1, there is a non-linear relationship between the vehicle flow, road capacity, and travel time. Since the model in Good (2019) seeks to minimize the total travel time across a network, a given road arc (i, j) can be chosen over another for DTA when increasing Y_{ij} keeps travel time close to the roadway unrestricted travel time (i.e., $Y_{ij} << u_{ij}$). Once the number of vehicles reaches the arc capacity u_{ij}, the travel time begins to increase non-linearly with each additional vehicle.

In both Equation 3.1 and Figure 3.1, we use the variable and parameter names from Good (2019) which are different than those used for roundabout and signalized intersection modeling. For example, the model from Good (2019) uses d_{ij} for road distance and s_{ij} for speed, where signalized intersection models use d_i^k for delay and s_i for saturation flow rate. However, some parameters and variables do correspond with those defined in Chapter 2. For example, Y_{ij} of a given road segment is the total number of vehicles traveling on that road segment from node i to j. Assuming that node j is a transshipment node, Y_{ij} is equal to the entering flow into intersection j, such that $Y_{ij} = Q_{ek}$ $\forall (i, j) \in A$, $\forall k \in E$. Moreover, u_{ij}, the capacity of a road segment, is qualitatively similar to intersection capacity C, but calculated differently. Here, u_{ij} is a parameter given to the model as an input $\forall (ij) \in A$, where C for roundabouts and signalized intersections is a function of the flows and design of entries and lane groups.

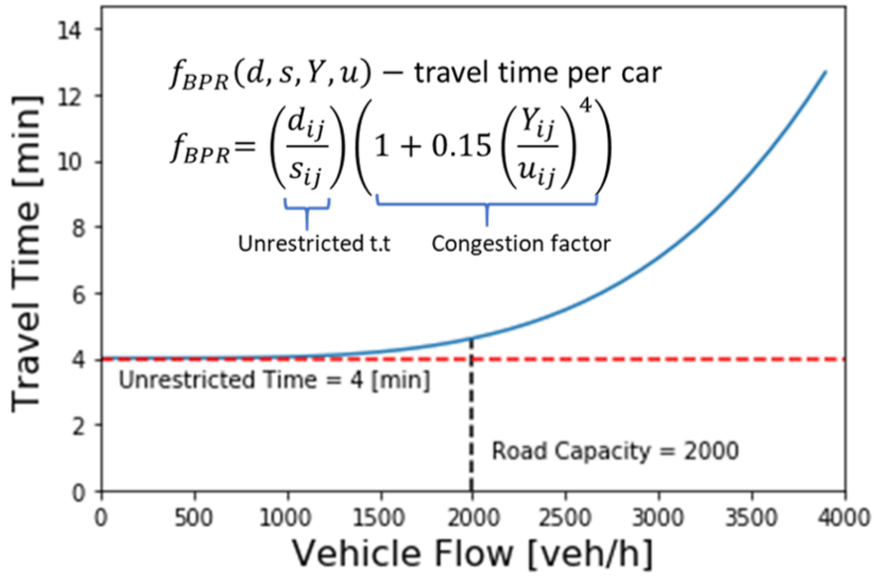

Figure 3.1. The Bureau of Public Roads Function. We present the BPR function based on (Peeta et al. 2015) and parameterized using values from Good (2019) and Routley (2020). Cars traveling across road a segment from noe i to node j have an unrestricted travel time of ≈ 4 min. The function is non-linear with respect to vehicle flow, and total travel time increases quadratically once $Y_{ij} \geq u_{ij}$

To avoid using the non-linear function during optimization solve, Good (2019) and Routley (2020) use a piecewise linear approximation to BPR function based on methods developed in Alderson et al. (2018a). Specifically, Alderson et al. (2018a) draw $\bar{r} = 40$ linear constraints that approximate the BPR function:

$$f_{ij} = \frac{d_{ij}}{s_{ij}}\left(1 + 0.15\left(\frac{r\lambda_{ij}}{u_{ij}}\right)^4\right) \quad \forall r \in R \tag{3.2}$$

where:

$\bar{r}$ = the number of linear approximations used;

$r \in R$ is the ordinal set of intervals between 0 and $\bar{r}$; and,

$\lambda_{ij} = \frac{u_{ij}}{\bar{r}}$ determines where each linear approximation is drawn for f_{ij}.

We plot Equation 3.2 in Figure 3.2 to show how the linear constraints relate to the non-linear BPR function. Parameters chosen for Figure 3.2 are $d_{ij} = 4$ mi $s_{ij} = 60$ mi/h $u_{ij} = 2000$

veh/hr, $\bar{r} = 20$, and Y_{ij} is bounded between $0 \leq Y_{max} \leq 2u_{ij}$. As we can see, the non-linear original function is divided into $\bar{r}$ sections, where the width of each segment is λ_{ij}. Using that technique, we turn non-linear constraint into $\bar{r}$ linear constraints.

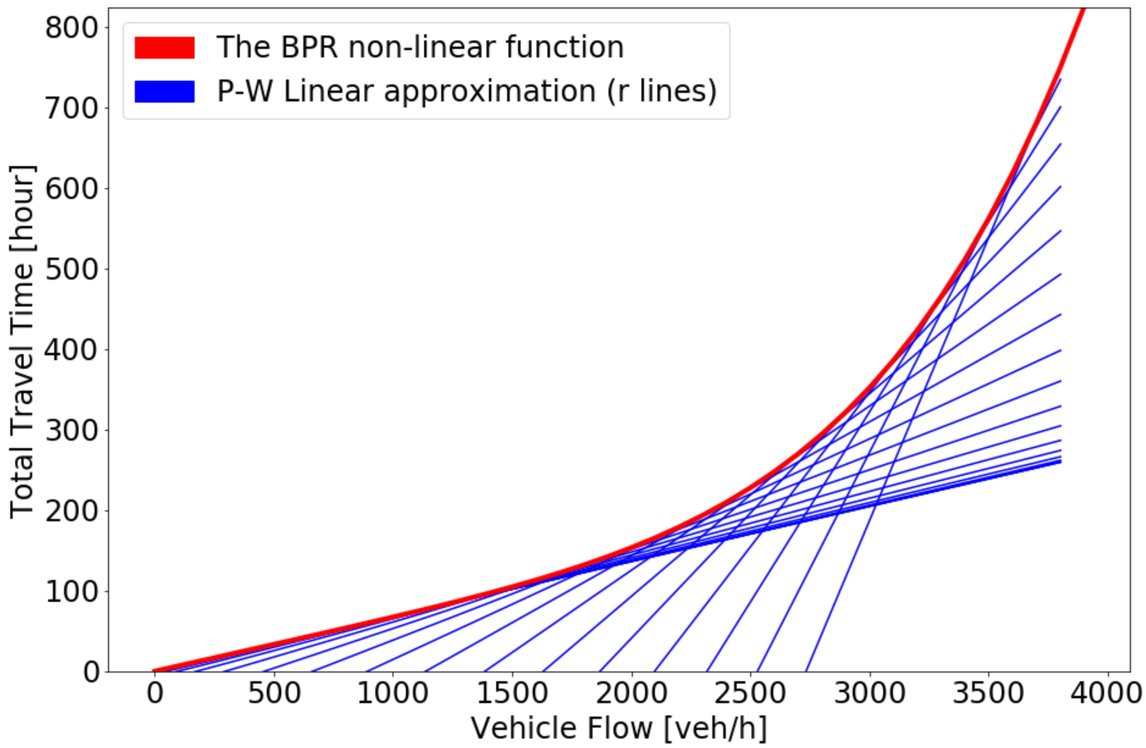

Figure 3.2. Piecewise Linear Approximation of the Bureau of Public Roads Function. Twenty linear blue lines ($\bar{r} = 20$) approximate the curvature of the red BPR function. For multi-commodity network flow optimization, the set of linear constraints that correspond to the blue lines are used to approximate non-linear congestion measured using the red line. Parameters are $d_{ij} = 4$ mi $s_{ij} = 60$ mi/h $u_{ij} = 2000$ veh/hr, $\bar{r} = 20$, and Y_{ij} is bounded between $0 \leq Y_{max} \leq 2u_{ij}$.

3.1.4 Model Formulation

Using the inputs and linear approximations described in section 3.1.3, the model in Good (2019) conducts DTA to minimize the travel time for all origin-destination pairs across a road network. We present the full mathematical model for traffic flow from Good (2019) to show how network primitives and linear approximations are implemented in the model:

Indices and Sets

$i \in N$	nodes (alias j,s,t)
$(i,j) \in A \subseteq N \times N$	arcs
$(s,t) \in D \subseteq N \times N$	set of all origin and destination pairs
$r \in R$	section for piece-wise linear approximation
	($\bar{r}$ = total number of sections)
$Out_i \subset A$	set of all outbound arcs from node i
$In_i \subset A$	set of all inbound arcs from node i

Data [units]

b_{st}	supply rate at node s destined for node t [VPH]
	($b_{st} < 0$ represents demands)
u_{ij}	nominal capacity of arc (i,j) [miles per hour [VPH]
s_{ij}	unrestricted speed of arc (i,j) [MPH]
d_{ij}	length of arc (i,j) [miles]
$avail_{ij}$	1 if arc (i,j) is available for use, 0 otherwise
q	travel window or time of analysis [hours]

Calculated Data [units]

λ_{ij} — interval width on arc (i,j) for calculating piece-wise linear congestion

$$\lambda_{ij} = \frac{2u_{ij}}{\bar{r}}$$

h_{ijr} — total travel time for all vehicles traversing segment r on arc (i,j) [hours]

$$h_{ijr} = (r\lambda ij)\left(\frac{d_{ij}}{s_{ij}}\right)\left(1 + 0.15\left(\frac{r\lambda_{ij}}{u_{ij}}\right)^4\right)$$

$slope_{ijr}$ — slope of segment r for arc (i,j)

$$slope_{ijr} = \frac{h_{jir} - h_{ij(r-1)}}{\lambda_{ij}}$$

$intercept_{ijr}$	y axis intercept of line section r for arc (i,j)
	$intercept_{ijr} = h_{ij(r-1)} - slope_{ijr} \cdot (r\lambda ij)$

Decision Variables [units]

Y_{stij}	flow rate of supply (s,t) transiting arc (i,j) [VPH]
Y_{ij}	total flow rate transiting arc (i,j) [VPH]
Z_{ij}	total vehicle-time spent on arc (i,j) [vehicle-hours]
$Dropped_{st}$	dropped quantity of supply (s,t) [vehicles]
$Excess_{st}$	excess quantity of supply (s,t) [vehicles]

Formulation

$$\min_{Y,Z,Dropped,Excess} \sum_{(i,j)\in A} Z_{ij} + \sum_{\substack{(s,t)\in D \\ s\neq t}} \frac{q}{2} \cdot Dropped_{st} \tag{3.3}$$

$$\sum_{(i,j)\in Out_i} Y_{stij} - \sum_{(j,i)\in In_i} Y_{stij} + Dropped_{st} = b_{st} \qquad \forall i \in N, (s,t) \in D, i = s \tag{3.4}$$

$$\sum_{(i,j)\in Out_i} Y_{stij} - \sum_{(j,i)\in In_i} Y_{stij} - Excess_{st} = -b_{st} \qquad \forall i \in N, (s,t) \in D, i = t \tag{3.5}$$

$$\sum_{(i,j)\in Out_i} Y_{stij} - \sum_{(j,i)\in In_i} Y_{stij} = 0 \qquad \forall i \in N, (s,t) \in D, i \neq s, i \neq t \tag{3.6}$$

$$Y_{stij} \leq b_{st} \qquad \forall (s,t) \in D, (i,j) \in A \tag{3.7}$$

$$Y_{ij} = \sum_{(s,t)\in D} Y_{stij} \qquad \forall (i,j) \in A \tag{3.8}$$

$$Y_{ij} \leq 2u_{ij}avail_{ij} \qquad \forall (i,j) \in A \tag{3.9}$$

$$Z_{ij} \geq intercept_{ijr} + slope_{ijr} \cdot Y_{ij} \qquad \forall (i,j) \in A, \forall rin\bar{r} \tag{3.10}$$

$$Excess_{st} = Excess_{ts} \qquad \forall (s,t) \in D \tag{3.11}$$

$$Dropped_{st} = Dropped_{ts} \qquad \forall (s,t) \in D \tag{3.12}$$

$$Y_{stij}, Y_{ij}, Z_{ij}, Dropped_{st}, Excess_{st} \geq 0 \qquad \forall (i,j) \in A, (s,t) \in D \tag{3.13}$$

We present a full description of the objective and constraints. First, Equation 3.3 is the objective function to minimize the time vehicles spend on roads [vehicle-hours], including any penalties for cars that cannot reach their intended destination within the travel window q. Equations 3.4, 3.5, and 3.6 are the flow constraints according to the different types of nodes — origin, destination, and transshipment. Equations 3.7 represent the supply constraint for the flow from origin nodes, and Equations 3.8 represent the relationship between the total flow on each arc as the sum of the flow of all commodities on this arc. Equations 3.9 set the upper bound on flow decision variables set to be $\leq$ two times the road capacity, and Equations 3.10 set the lower bound on the total vehicle time on each arc Z_{ij} by implementing a piece-wise linear function on the BPR function. Equations 3.11 and 3.12 provide symmetric constraints for elastic variables, and 3.13 is non-negativity constraint for the decision variables.

3.1.5 Simple Example: A Single Origin-Destination Pair and Transshipment Node

We use the technique from Good (2019) to develop a road network representation that can relate to intersection models. In Figure 3.3, we present a simple road network that could be modeled using the methods from Good (2019). This network has one origin (node 1), one destination destination (node 6), and one transshipment node (node 2). Using the established notation, we represent a single vehicle originating at (node 1) as $b_{16} = 1$ and the corresponding return trip as $b_{61} = -1$. The model in Good (2019) finds the route with the shortest travel time over this network. The shortest (and only) route in this simple example is to travel through transshipment node (node 2) with flow $Y_{1216} = Y_{2616} = 1$.

Although this example is trivial, it provides a good foundation for intersection modeling. In particular, to build upon the work of Good (2019), it makes sense to consider a node-splitting strategy that allows non-linear delays to be assessed for traversing intersections. This strategy works for both roundabouts and signalized intersections.

3.2 Roundabout Modeling

We develop a method for measuring roundabout delay times that integrates into the Good (2019) model. The main delay factors for roundabouts are described in Chapter 2 Sec-

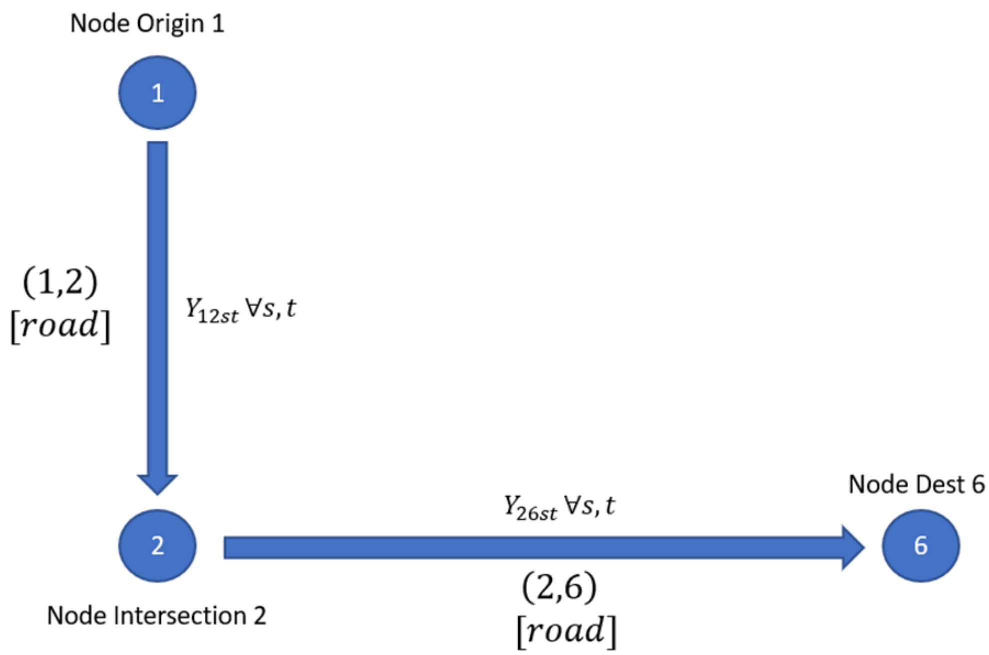

Figure 3.3. Road Network Represented as a Simple Flow Network. This simple model has a single origin (1), destination (6), and transshipment (2) node connected by two roads (1,2) and (2,6). Cars originating at origin (1) traveling to destination (6) follow the path shown. Associated decision variables Y_{ijst} from the Good (2019) and Routley (2020) multi-commodity flow models are shown.

tion 2.3.3. We focus analysis on D_i, the control delay time as an approximation for total delay time. We make this assumption because the model developed by Good (2019) already measures travel time, which we assume include effects of approaching speed. We also assume circulating speed is equal for all vehicles within a roundabout intersection and is small relate to D_i.

3.2.1 Roundabout Representation for Flow Networks

We use node splitting strategy to link roundabout delays to DTA and multi-commodity flow networks. Our node splitting strategy takes transshipment nodes and "splits" them into intersection nodes representing roundabout legs. Intersection nodes are then connected by arcs $(i, j) \in T \in A$ representing the different turns cars can make when traveling between

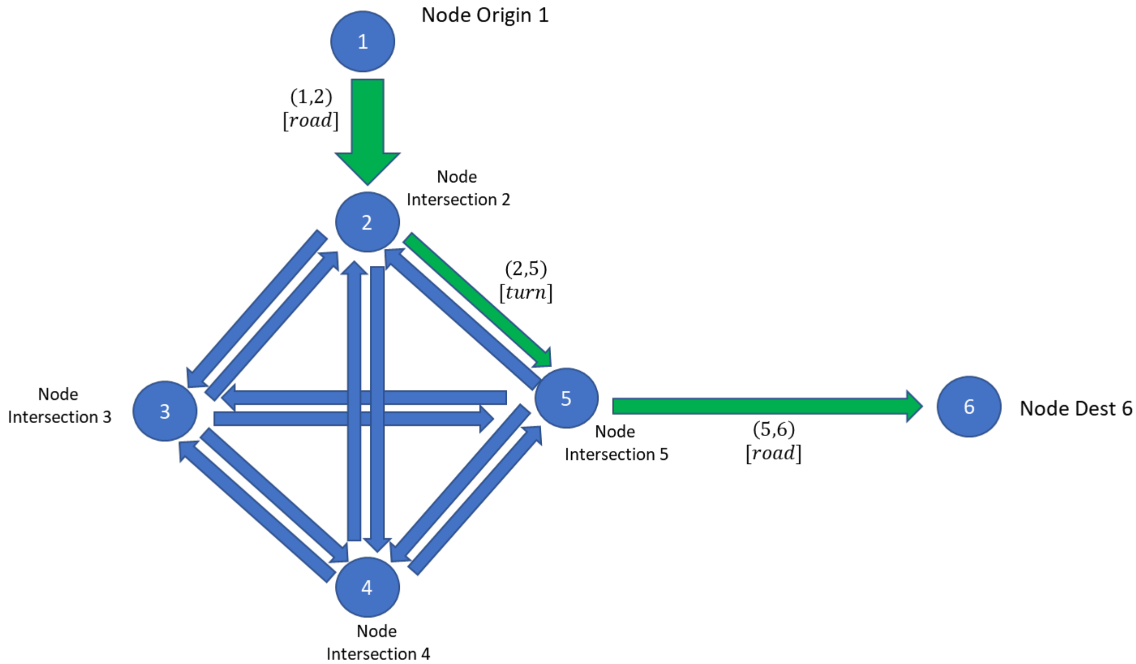

Figure 3.4. Node Splitting Strategy to Incorporate Roundabouts in Road Network Models. We update the simple flow network from Figure 3.3 to consider traffic delay calculations associated with roundabout intersections. Here, transshipment node (2) is 'split' into four nodes (2-5) connected with arcs $(i, j) \in 2, 3, 4, 5$ representing all possible intersection entries, exists, and turns. The new flow path for cars originating at origin (1) traveling to destination (6) is shown in green. Travel time calculation over turn (2,5) is based on delay functions for roundabouts, where travel time calculation over roads (1,2) and (2,6) is based on the BPR function.

legs in a roundabout (i.e., left, right, and straight).

Figure 3.4 presents this connectivity for the simple network presented in Figure 3.3. Here, we assume transshipment node (2) is a four way roundabout. Accordingly, we split node 2 into 4 intersection nodes (2-5) where each node represents roundabout leg $i \in E$. Each turn within the intersection is now represented by arcs $(i, j) \in 2, 3, 4, 5$. Thus, the same route described in Figure 3.3 from origin (1) into destination (6) is now: road (1,2) $\rightarrow$ turn (2,5) $\rightarrow$ road (5,6).

3.2.2 Roundabout Delay Time Calculation

We use the method presented in 2 to calculate the expected delay in roundabout. The roundabout delay time estimation is based on two main parameters - the circulating flow and the capacity which are derived per entry.

Circulating Flow

The key difference between traffic calculations in Good (2019) and the proposed model, is that the total travel time for $Y_{1,2,s,t}$ in Figure 3.4 at node 2 is affected by the circulating flow within the roundabout. Circulating flow can also be captured using our node splitting strategy, represented by flows on arcs $(3,5)$, $(3,4)$, and, $(4,5)$ in Figure 3.5. These three flows represent all turns that may block entry into the intersection at (2) according to left-hand-side driving in the USVI. The approach enables roundabout circulating flow for entry (2) using decision variables from the Good (2019):

$$Q_{c2} = \sum_{s} \sum_{t} Y_{34st} + Y_{35st} + Y_{45st} \quad \forall (s,t) \in D. \tag{3.14}$$

Capacity and Delay Time

With Equation 3.14 relating flow variables Y_{ijst} to Q_{ci}, we can use Equations 2.9 and 2.10 from the HCM to calculate roundabout capacity and traffic delay time. Capacity calculation with Equation 2.9 (Chapter 2, Section 2.3.2) does not change for given new notation and use, where Equation 2.10 (Chapter 2, Section 2.3.3) replaces entry flows Q_{ei} with model decision variables. We reproduce Equation 2.9 here for completeness.

$$C_j = \frac{Q_c \exp\left(\frac{-Q_{cj}T_c}{3600}\right)}{1 - \exp\left(\frac{-Q_{cj}T_f}{3600}\right)} \left[\frac{pcu}{h}\right]$$

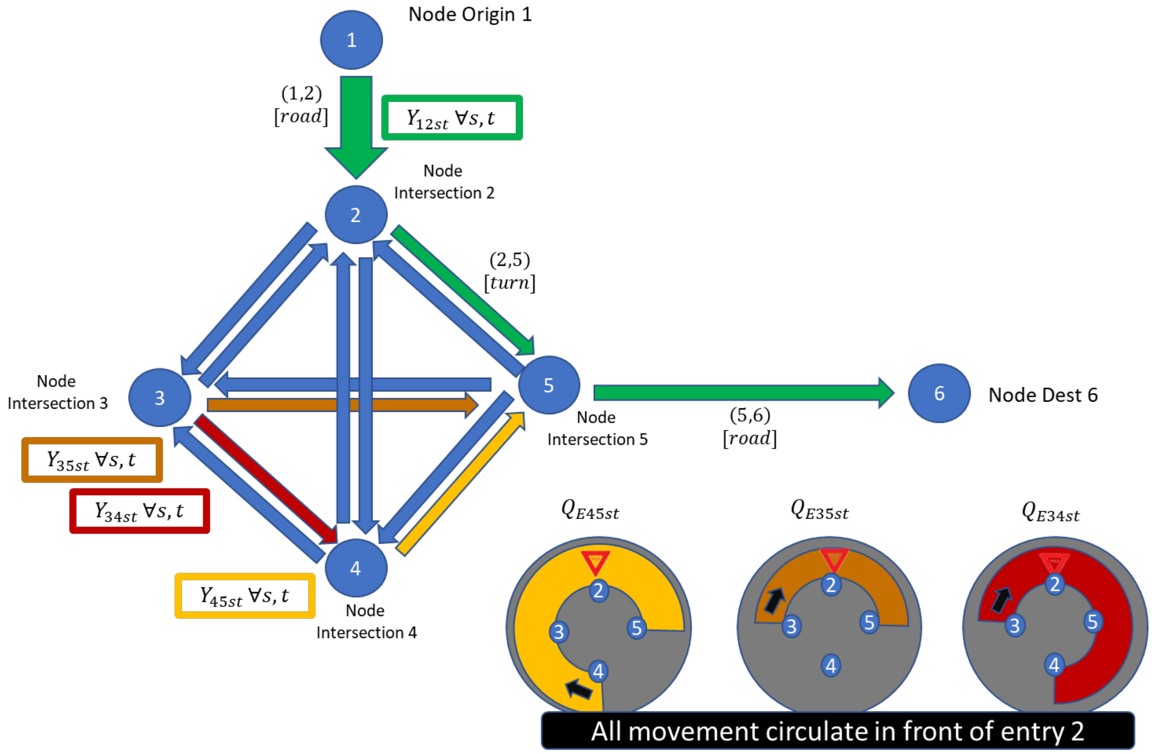

Figure 3.5. Example of Circulating Flow in Road Networks. Flow variables developed using our node splitting strategy enable circulating flow and capacity calculation for roundabouts with road network models. Here, we show which arcs are involved in determining circulating flow for intersection node (2). Specifically, turns (3,4), (3,5), and (4,5) generate traffic that blocks the entry of vehicles from node (2) into the roundabout. The sum of the Y_{ijst} over these three turns determines Q_{c2} and C_2 needed for roundabout time delay calculation.

$$D_j = \frac{3600}{C_j} + 900T \cdot \left(\frac{Y_{ij}}{C_j} - 1 + \sqrt{\left(\frac{Y_{ij}}{C_j} - 1\right)^2 + \frac{\left(\frac{3600}{C_j}\right) \cdot \left(\frac{Y_{ij}}{C_j}\right)}{450T}} \right) \tag{3.15}$$

where:

C_j = capacity for roundabout entry (intersection node) j;

Q_{cj} = circulating flow for entry j;

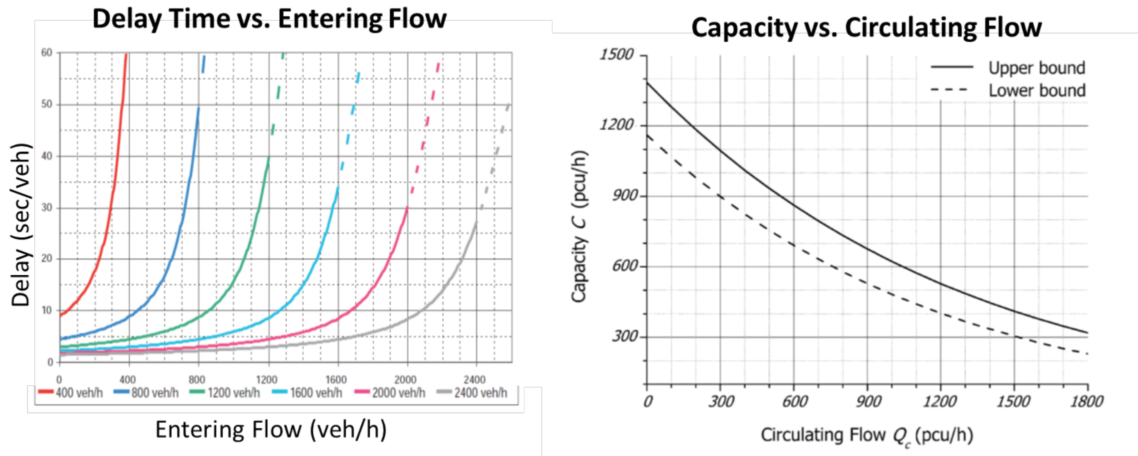

Figure 3.6. Roundabout Capacity and Delay Time. Source: U.S Federal Highway Administration (2000), Mauro (2010).

T_c = time required between cars for merging into the circular road [sec];
T_f = the time it takes to enter the service line when its cleared [sec];
D_j = delay time per vehicle entering the roundabout at entry j [sec/veh];
Y_{ij}= total flow of vehicles from node i towards intersection node j [pcu/h];
T= analysis time period [hour].

The resulting model uses standard measures for roundabout flows, capacities, and delay times in a manner that can be integrated into multi-commodity flow models. Figure 3.6 presents examples of entry capacities and related delay times using our methods.

3.3 Signalized Intersection Modeling

We use the same node splitting strategy developed for roundabouts to model signalized intersections. However, there are several key differences between time delay calculations for roundabouts and traffic signals that require additional assumptions and parameters.

3.3.1 Signalized Intersections and Node Splitting

Travel time through a given entry to a signalized intersection depends on lane group (see Figure 3.7). Lane groups combine multiple turning arcs (ij) that all receive simultaneous green lights or priority. A common lane group in the mainland U.S. combines vehicles making a right turn with those going straight. This is the opposite for the USVI, i.e., the

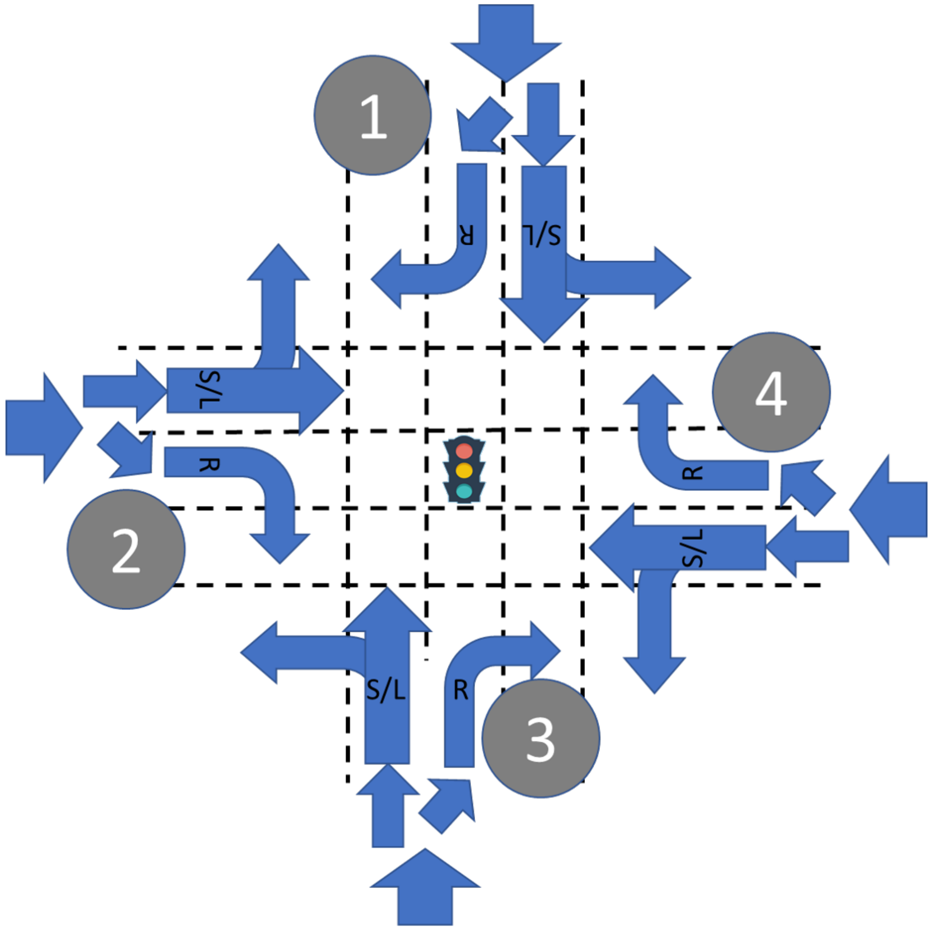

Figure 3.7. USVI Signal Intersection Illustration

most common lane group in the territory combines left turns with cars traveling straight. For example, using entry labels from Figure 3.7, we calculate the total travel time for turn $(1,4)$ (left) and $(1,3)$ (straight) for entry (1) as a single lane group.

Signalized intersection capacity is not dependent on flows within the intersection. Whereas roundabout capacity is a function of circulating flow Q_c, we assume that signal priority deals with any possible blocking or clearing time that would affect cars entering an intersection with a traffic light. There is no need to calculate an equivalent Q_c for travel time through signalized intersections. This means the capacity of the each lane group is independent for a signalized intersection does not depend on the traffic flow for other entries. This means the capacity of each turning arc (i,j) for a signalized intersection is dependent only on parameters for road geometry and design.

While entry capacities are independent of other entries, the travel time for a given lane group depends on entry approach and signal timing. In a single lane roundabout, each approach has the same waiting time regardless what turn a driver would take at the intersection. However, with signals, the intersection capacity depends on the signal parameters, and specifically on the green cycle and the total signal time cycles for each lane group. Therefore, if a signal would have different timing for different turns (right left or through), a different turns would have a different waiting time.

For all calculations in this thesis, we assume that lane groups each have independent lanes for queuing and waiting for the signal. This is shown in Figure 3.7 for entry (1) as right turns are given a protected lane for queuing separated from the lane group for left/straight. We further assume that the queue length for each lane group does not affect travel time within the other, and vice versa. Finally, we assume that simultaneous counter flow (e.g., lane group $(1, 3)$ and $(3, 1)$) is possible within a given intersection, and does not impede one another.

3.3.2 Signal Parameters

There are several key parameters for signal timing that impact total time delays through intersections. The first key parameters are the green light duration for each lane group for each entry g_i and the signal cycle time for the intersection c. In general, the ratio of green light time to the signal cycle ($\frac{g}{c}$) sets the effective capacity for each lane group in the intersection and determines *cycle flow*. For higher traffic flows, a higher cycle flow is needed (Urbanik et al. 2015).

We choose a signal cycle $c = 90$ for traffic signals with normal traffic conditions and $c = 100$ for the more congested intersections. The delay time of an intersection depends on the cycle length whereas too short or too long signal cycles are inefficient. When the signal cycle is too short, vehicles cannot cross the intersection. Conversely, if the signal cycle is too long, there is wasted green time. Optimal signal cycle times for a one-lane four-way intersection are presented in Figure 3.8. According to National Association of City Transportation Officials (2013), a general guideline for the signal cycle length in an urban environment is between 60-90 seconds.

We match the green light time for each lane group to the expected entering flow to reduce

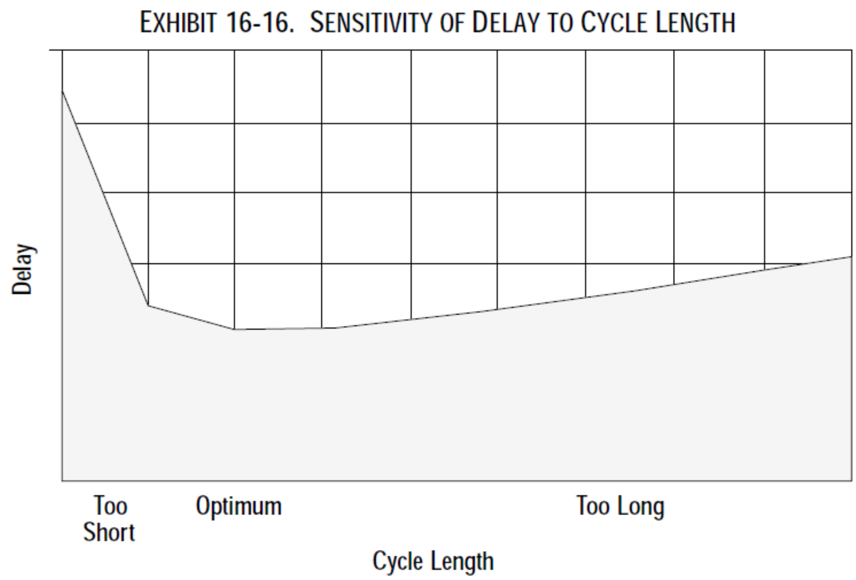

Figure 3.8. Illustration of the tradeoff between signal cycle length and travel time through an intersection. Source: Transportation Research Board (2000).

total delay (Transportation Research Board 2000). For a given cycle time c, g_i must be chosen for each lane group with simultaneous flow. Figure 3.9 presents general guidelines for the green light timing depends on the type of the road (facility) and the type of turn. We apply these same rules for choosing g_i relative to left/straight and right turns for each entry of an intersection.

We choose different value of g_i depending on intersection design. For example, In some cases when one street ("major") is busier than the other ("minor"), or when one lane group has higher entering flow than another, it might be implemented in different green timings, as illustrated in Figure 3.9. In general, when an intersection has major and minor streets, the recommended ratio signal times between the two is 3:2 (National Association of City Transportation Officials 2013).

The effective green time g_i also depends on the signal lost time, i.e., the time when the intersection is effectively not being used. According to Urbanik et al. (2015), there are two main components of lost time. First, the clearance interval t_{clear} is the time between one signal turning red and another turning green to ensure the intersection is empty prior

Phase Type	Facility Type	Maximum Green (Seconds)
Through	Major Arterial (> 40 mph)	50 to 70
	Major Arterial (≤ 40 mph)	40 to 60
	Minor Arterial	30 to 50
	Collector, Local, or Driveway	20 to 40
Left Turn	Any	15 to 30

Phase Type	Facility Type	Minimum Green (Seconds)
Through	Major Arterial (> 40 mph)	10 to 15
	Major Arterial (≤ 40 mph)	7 to 15
	Minor Arterial	4 to 10
	Collector, Local, or Driveway	2 to 10
Left Turn	Any	2 to 5

Figure 3.9. Typical values for minimum and maximum green based on queue clearance. Source: Urbanik et al. (2015).

to additional traffic. Table 3.1 presents clearance times for various intersection approach speeds. Second, the start up t_{start} time is the time waiting for the first cars to move once a signal becomes green. Several empirical studies show start up time tends to be about two seconds in normal weather conditions (Urbanik et al. 2015). The effective green time for an entry is $g_i - (t_{clear} + t_{start})$. We use the values from Table 3.1 and Urbanik et al. (2015) to determine the effective green time per lane group.

Approach Speed (MPH)	Red Clearance [Sec]		
	Width of Intersection [Feet]		
	70	90	110
25	2.5	3.0	3.5
30	2.0	2.5	3.0
35	1.8	2.1	2.5
40	1.5	1.9	2.2
45	1.4	1.7	2.0
50	1.2	1.5	1.8
55	1.1	1.4	1.6
60	1.0	1.2	1.5

Table 3.1. Red Light Clearance Times. Red clearance values in seconds as function of the approaching speed to the intersection and the width of the intersection. Adapted from Urbanik et al. (2015) and based on McGee et al. (2012).

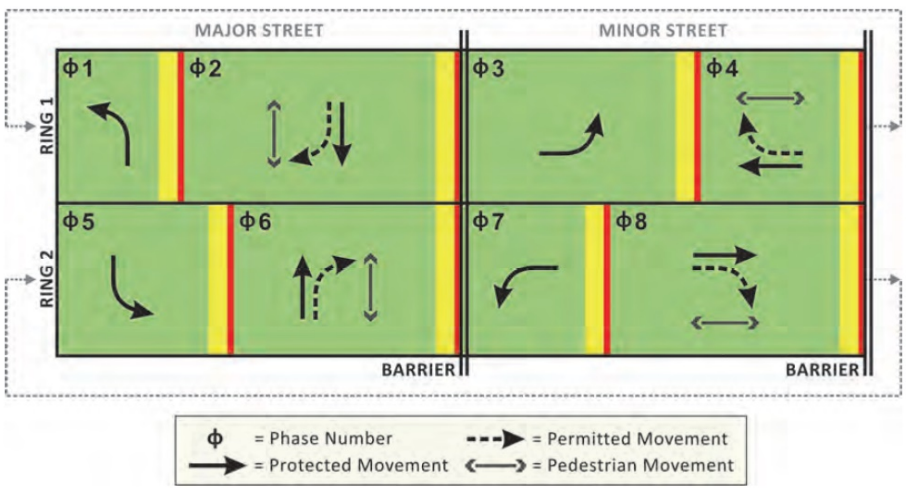

Figure 3.10. In the example there are 8 different phases - 4 for the major street and 4 for the minor street (major and minor street are getting roughly similar duration of green time but it is not always the case). Phases 1,2,5 and 6 serve the major street (North-South) when phase 1 and 5 are a protected left turn and phases 2 and 6 are driving through and right turn. Phases 3,4,7 and 8 serve the minor street (East-West) when phase 3 and 7 are a protected left turn and phases 4 and 8 are driving through and right turn. The phases in the pictures are corresponding to right-side-driving as practiced in the United States. Source: Urbanik et al. (2015).

3.3.3 Signalized Delay Time Calculation

As described in Chapter 2, the delay time for each lane group depends on signal parameters and an adjustment factor for traffic patterns 2.4.2. We build on the methods presented in Chapter 2 from the Transportation Research Board (2000) for delay time to determine the functions necessary for calculating delay. Specifically, we use Equation (2.18) to caluclate delay time with the following assumptions:

- **Random Arrivals** - We assume random arrivals assumption, which treats the progression adjustment factor as equal to one ($PR = 1.0$).
- **No Initial Delay** - We assume no cars initially queued at any intersection prior to analysis, such that $d_i^3 = 0$.
- **Under/Near Saturation Condition** - we assume traffic does not exceed saturation conditions, such that $min(1, X) = X$ where X is the car flow rate to road capacity ratio. Note: In our model we allow X to take on values between $0 \leq X \leq 2$, but in case when $X \geq 1$, the incremental delay d_i^2 will grow faster than the uniform delay d_i^1. Thus, our total delay will be accurate even as flows near saturation.

With these assumptions, the total delay time for a lane group at entry i, d_i, is the sum of the uniform delay d_i^1 and incremental delay d_i^2:

$$d_i = \frac{0.5c\left(1 - \frac{g_i}{c}\right)^2}{1 - (X) \cdot \frac{g_i}{c}} + 900T\left[(X-1) + \sqrt{(X-1)^2 + \frac{8klX}{C_i T}}\right] \tag{3.16}$$

Where:

C_i = lane group capacity calculation [veh/h];

s = saturation flow [veh/h] ;

g_i = each phase green light [sec];

c = signal cycle [sec] ;

Y_{ij} = vehicle flow [veh/hour] ;

$X = \frac{Y_{ij}}{C}$ lane group flow-capacity ratio, i.e., degree of saturation;

T = duration of analysis [hours];

k = incremental delay factor that is dependent on controller settings. For pre-timed signals, a value of $k = 0.5$ is used; and,

l = upstream filtering/metering adjustment factor. For isolated intersections $l = 1$.

3.3.4 Signal Delay During Disasters

As discussed in Chapter 2, a signal has a high chance to lose power and stop working due to the damage caused by a disaster like a hurricane. Moreover, if signal controls fail during a disaster, then the timing for each lane group is also lost. Given the high possibility that either lighting or control systems are damaged during a disaster, we assume that all signalized intersections fail to function and act as stop signs after a disaster.

We measure the travel time through a signalized intersection post-disaster as a stop sign using Equation 3.16. Specifically, we assume that controls fail and there is no way to have different green light timing for each entry. Thus, the total effective green time is divided equally between all entries, $g_i = \frac{c}{|E|}; \forall i \in E$. Moreover, we assume that each time a vehicle crosses a stop sign intersection, there is additional t_{clear}. We assume it takes 4 seconds for a vehicle to cross the intersection and for the next car to enter — $t_{clear} = 2$ and $t_{start} = 2$ for a stop sign (Urbanik et al. 2015).

3.4 Efficiency and Resilience Measures

The goal of this thesis is to analyse the tradeoffs between different intersection designs such as roundabout and signals. We propose measures to compare the efficiency and robustness of a given intersection to others. These measures focus attention on travel time during normal network operations and after disasters.

3.4.1 Intersection Efficiency

In general, the efficiency of a road network relates to the total travel time for vehicles traveling between origins and destinations. As described in Chapter 2, intersections control traffic to allow for safe and efficient turns, and ideal intersection would have either no effect on total travel time for a vehicles or reduce total travel time using specialized equipment. In practice, all intersections increase the travel time of vehicles driving through them. The added travel time depends on several parameters such as intersection type, characteristics, and traffic flows. Thus, the added travel time for a given intersection is a key measure of intersection efficiency. We define the *intersection efficiency score* as the added travel time for a given origin-destination pair traversing an intersection:

$$\eta^i_{st} = TT^i_{st} - TT^0_{st}; \tag{3.17}$$

where:
TT^0_{st} = the travel time for origin-destination pair st without intersection i;
$TT^{intersection}_{st}$ = the travel time for origin-destination pair st with intersection i.

Using Equation 3.17, we can compare the effects of two intersections on a given route through a road network. We can also compare the relative efficiency of two intersections i and j using the following equation:

$$\eta^{rel}_{st} = \eta^i_{st} - \eta^j_{st} = TT^i_{st} - TT^j_{st} \quad i \neq j \tag{3.18}$$

By convention, the added travel time measured by η^{rel}_{st} should be positive. Thus, we calculate always calculate η^{rel}_{st} where $TT^i_{st} \geq TT^j_{st}$.

3.4.2 Intersection Robustness

Alderson et al. (2018a) describes the structural resilience of a system as the ability of a system to recover or withstand a shock or a disturbance from an event like a natural disaster. Adopting this approach, we define a resilience measure based on ability of an intersection to provide the same service given a disruption or disaster event.

In particular, we measure how well an intersection can perform in a post-storm scenario in terms of travel time. In a poststorm scenario, some roads are going to be either blocked and traffic lights will operate as a stop sign. In all cases, we expect intersections to have larger delay after a disaster. We develop a normalized resilience score relating to total travel time through an intersection before and after a disaster:

$$\sigma_{resilience} = TT^i_{st}(prestorm) - TT^i_{st}(poststorm) \tag{3.19}$$
$$given : TT^i_{st}(prestorm) \geq TT^i_{st}(poststorm); \tag{3.20}$$

where:
$\sigma_{resilience}$ = the resilience score of the examined scenario road network [0,1];
$TT^i_{st}(prestorm)$ = the total travel time through intersection i before a disaster; and,
$TT^i_{st}(poststorm)$ = the total travel time through intersection i after a disaster.

It is also possible to compare intersections using this metric by setting $TT^i_{st}(poststorm)$ as $TT^j_{st}(poststorm)$ where $i \neq j$.

3.5 Intersection Analysis and Case Studies

We use the presented methods and measures to assess the efficiency-resilience tradeoffs for roundabouts and signalized intersections. Due to the many factors that influence intersection flows and delay times, we perform parametric analysis to compare η and σ. We also assess case study locations with the USVI to determine the potential benefits or travel time issues with roundabouts and traffic lights. Specifically, we identify several candidate intersections on STX and STT to assess efficiency-resilience tradeoffs. We use output from Good (2019) and Routley (2020) to determine vehicle flows through intersections during curfew and disaster scenarios. We use these flows to measure η and σ for each case study. The resulting analysis informs recommendations for transportation network design and territorial resilience.

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CHAPTER 4:
Analysis and Results

We use methods presented in Chapter 3 to assess intersection efficiency and resilience in road networks. We analyze the effects of design and flow variables on intersection delays to determine when engineers may prefer traffic lights over roundabouts, or vice versa. We present results for roundabouts and traffic signals during normal and disaster travel times for the following cases:

4.1. Four-way, single-lane intersection:
   (a) a generic intersection with equal traffic flows over all entries;
   (b) a generic intersection with unequal traffic flows over all entries;

4.2. Inspired by real intersections in the USVI:
   (c) an intersection with major and minor flow directions similar to the cross streets of the UVI campus and Centerline Rd. on STX;
   (d) an intersection with major and minor flow directions and nearby critical supplies similar to the cross streets of the Queen Mary Highway and Centerline Rd. on STX;
   (e) an intersection with high traffic and nearby critical supplies similar to the cross streets of Weymouth Rhymer Highway, Alton Adams Dr., and Rumer Dr. on STX; and,
   (f) an intersection with high traffic similar to the cross streets of the Weymouth Rhymer Highway an Merhindal Rd. on STT.

4.1 Efficiency vs. Resilience of a Four-Way Intersection

We conduct a parametric analysis of roundabout and traffic signal time delay functions for a generic intersection to determine general efficiency and resilience tradeoffs. We assess the delay time per intersection entry and the weighted average delay time over all entries. Specifically, we assess a four-way, single-lane intersection similar to the intersection presented in Figure 3.7.

4.1.1 Analysis Assumptions

We use the following parameters and assumptions for all intersection designs for parametric analysis:

- **Entry Flow** — The vehicle flows Q_{ei} $\forall i \in E$ range between 100 to 1,100 in steps of 100. This approach measures delay times up to the assumed intersection maximum capacity of 1,200 vehicles, which means intersection entries are undersaturated.
- **Turns Ratio** — We assume a uniformly distributed turns ratio between left turns, right turns, or straight. Hence, for all the four approaches, one third of the entry flow approaching into the intersection would turn left, right or straight.
- **Analysis Time Period** — When calculating the delay time, we assume the analysis period is 1 hr ($T = 1$). We assume that the entering flow is constant and steady-state conditions hold over the time period.

We also make specific assumptions for each intersection design.

Roundabouts

We assume vehicle flows are uniformly distributed across entries and turns and that no vehicle is making a U-turn. We further assume that driving culture is conservative, which affects minimum clearing time and critical gap when entering the intersection. This assumption provides the upper bound on expected delay time for all calculations and sets $T_c = 4.1$ and $T_f = 2.6$ (Transportation Research Board 2000).

Traffic Signals

As a convention, we use lane groups from the USVI, i.e., right turn lane (R) which is branched from the main lane and the driving straight or turning left lane (S/L). Then, we divide the green time between the different lane groups based on flow ratios: a third of the green light (33%) for right turns and two thirds of the green light (66%) for the driving straight/left turn. We set the cycle length to be 90 seconds as recommended for urban areas (National Association of City Transportation Officials 2013). We set start up time t_{start} and intersection clearance time t_{clear} to be 2 seconds each, or 4 seconds per entry. Finally, we assume that yellow light functions effectively as a green light for modeling purposes.

Table 4.1 summarizes all the assigned parameters for calculating the signalized intersection delay time. We also show the signal timing and order in the examined case in Table 4.2. Together, Tables 4.1 and 4.2 present the total and effective green light times for each lane group as well as which entries have green in each signal phase.

Cycle Length	Major-Minor - street ratio	Right-straightt/Left lanes group ratio	Start-Up Time	Clearance Time
90 [sec]	1:1	1:2	2	2

Table 4.1. Signals Parameters

Phase No.	Entries Involved	Lane Groups	Total Time	Effective Green time
Phase 1	1 & 3	Right	15	11
Phase 2	1 & 3	Straight / Left	30	26
Phase 3	2 & 4	Right	15	11
Phase 4	2 & 4	Straight / Left	30	26

Table 4.2. Signal Phases and Green Light Timing

Finally, we assume signals are pretimed and the intersection delay is not affected by nearby intersection traffic overflow. These assumptions correspond to a value of the incremental delay factor $k = 0.5$ and upstream filtering or metering adjustment factor $l = 1$, respectively.

Stop Signs

For stop signs, we assume similar parameters as traffic signals with slight modifications. Specifically, we assume that the ratio between green light and cycle length $\frac{g}{c} = 0.25$ which corresponds to each entry receiving equal green light timing. Moreover, it important to note that our approach to stop sign modeling assumes similar vehicle flows to a traffic signal, which only applies when the intersection is not empty (i.e., $Q_{ei} > 0;\ \forall i \in E$. For this reason, results for stop signs are most accurate for $Q_e \geq 450\ \frac{pcu}{h}$.

4.1.2 Case (a): Equal Flow over all Entries

We study intersection efficiency by comparing roundabouts to traffic signals and intersection resilience by comparing roundabouts to traffic signals after a disaster (i.e., operating as a stop sign).

Efficiency: Roundabout vs. Traffic Signals

Figure 4.1 shows the expected delay time for a four-way, single-lane roundabout. The results in Figure 4.1 applies to all intersection entries. The delay time for each entry is the same because each entry has the same entering flow and turns ratio. Figure 4.1 shows that the expected delay is less than a minute for entering flow of 600 $\frac{pcu}{h}$ or less. For example, entering flow of $Q_e = 600\ \frac{pcu}{h}$ is $\approx$ 10 s and $0 \leq Q_e \leq 400\ \frac{pcu}{h}$ is 5 s or less. However, for higher entering flow the delay time starts increasing non-linearly to over 20 minutes for $Q_e = 1{,}000\ \frac{pcu}{h}$.

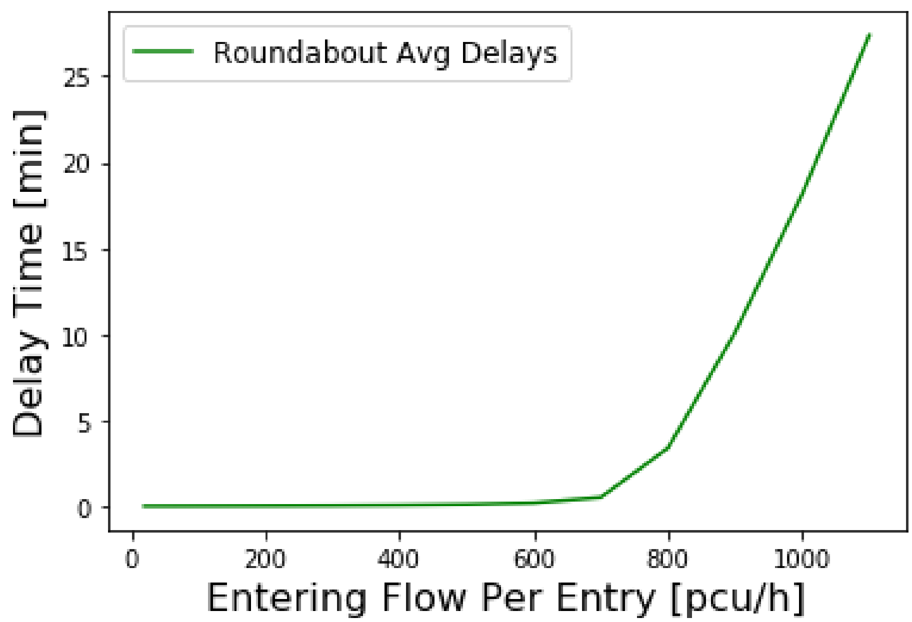

Figure 4.1. Equal Entering Flows—Roundabout Average Delay Times. Roundabout average delay as function of entering flow given equal enter-ing flow and exits ratio for all entries. Delay is short (few seconds) for light traffic ($Q_e \leq 700\ \frac{pcu}{h}$) and starts increasing dramatically for heavier traffic flow (up to 25 minutes).

Figure 4.2 presents the delay time for a four-way, single-lane signalized intersection. Here, all entries have the same delay times (similar to roundabouts), but different lane groups have different delays. Specifically, right turns receive less green signal time, which corresponds to longer delay times. We provide a weighted average for the two lane groups to estimate the expected delay time per approaching car. Figure 4.2 shows similar results to roundabouts, where low values of Q_e correspond to delay times less than a minute. However, for greate values of Q_e the expected delay time increases at a non-linear rate that is less than roundabouts, with a maximum delay between 10-15 minutes for entering flow of $Q_e \geq 1000\frac{pcu}{h}$.

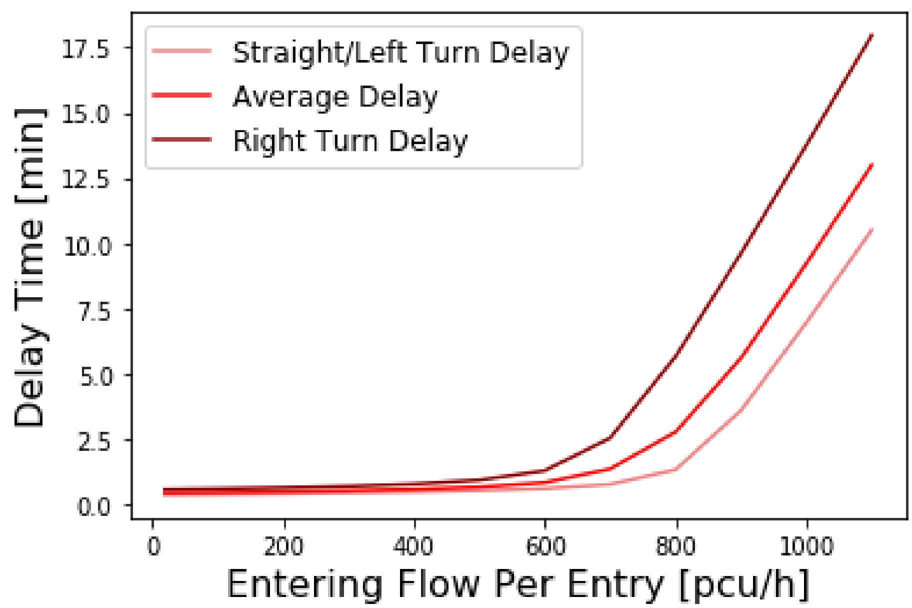

Figure 4.2. Equal Entering Flows—Signal Average Delay Times. Signal average delay as function of lane group (R: right; S/L: straight or left turn) and entering flow given equal entering flow and turns ratio for all entries. Delay is short (less than a minute) for light traffic ($Q_e \leq 600\,\frac{pcu}{h}$) and starts increasing dramatically for heavier traffic flow (up to 15 minutes).

Figure 4.3 presents the absolute difference (left) and relative difference (right) of delay times between roundabouts and traffic lights. Figure 4.3 is helpful for understanding intersection efficiency before a disaster. The relative difference shows that roundabouts are more efficient when traffic flows are small ($0 \leq Q_e \leq 800$), yet traffic lights are more efficient traffic flows are large ($Q_e \geq 800$). In general, this means that roundabouts will be faster during non-peak

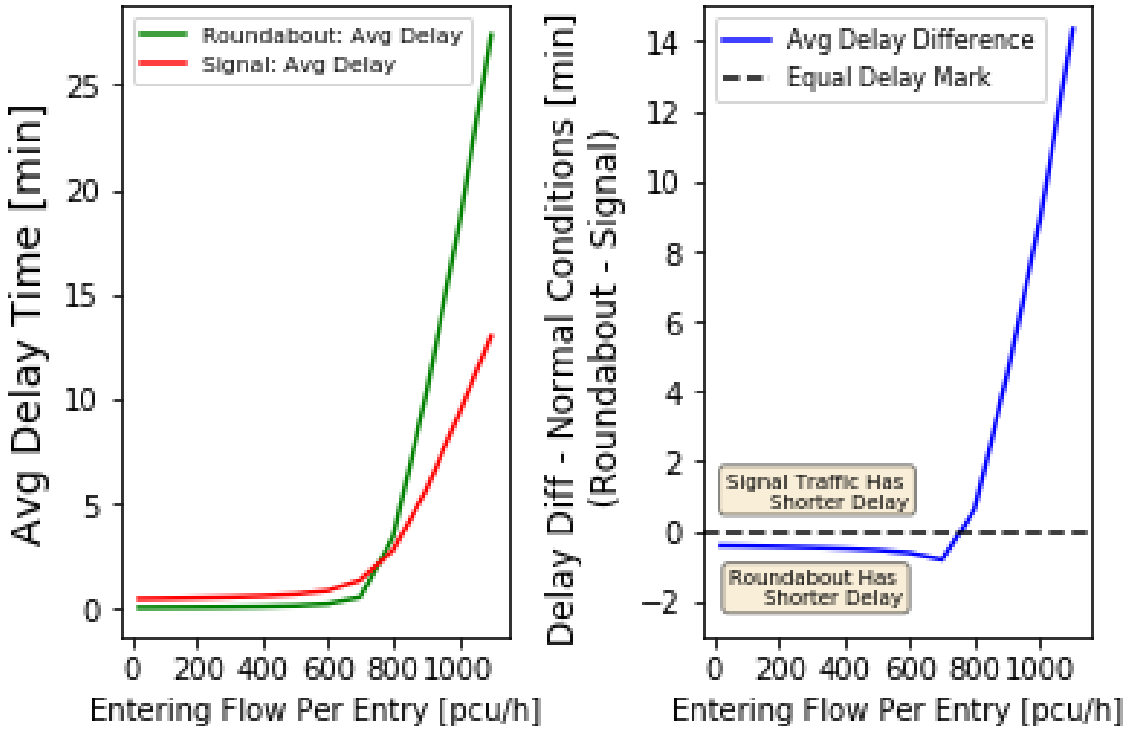

Figure 4.3. Equal Entering Flows—Roundabout & Signal Efficiency Comparison. Left: comparison of roundabout and signalized intersection average delay as function of entering flow given equal entering flow and turns or exits ratio. Right: the difference between these two curves. Roundabouts have shorter delay for light traffic ($Q_e \leq 700\frac{pcu}{h}$). For heavy traffic, signals have a shorter delay.

traffic periods, yet the absolute difference is small and is less than 1 minute. In contrast, traffic lights will be up to 14 min faster than a roundabout (on average) for the same intersection during high traffic conditions. For this reasons, during non-disaster conditions, the traffic signal is preferred.

Resilience: Roundabout vs. Stop Sign

In the absence of electricity (e.g., after a disaster), a traffic light will operate as a stop sign. Figure 4.4 presents the absolute difference (left) and relative difference (right) of delay times between roundabouts and stop signs. Figure 4.4 shows that there is no significance difference between the expected delay time of roundabouts and stop signs for low entering flow. However, for Entering flow $Q_e \geq 500\ \frac{pcu}{h}$ For example, for $700 \leq Q_e \leq 1000$, the relative time delay between intersections is between 15 - 18 minutes. Thus, a roundabout is preferred over a traffic light for a four way intersection with high flows after a disaster. This is opposite result between roundabouts and traffic lights.

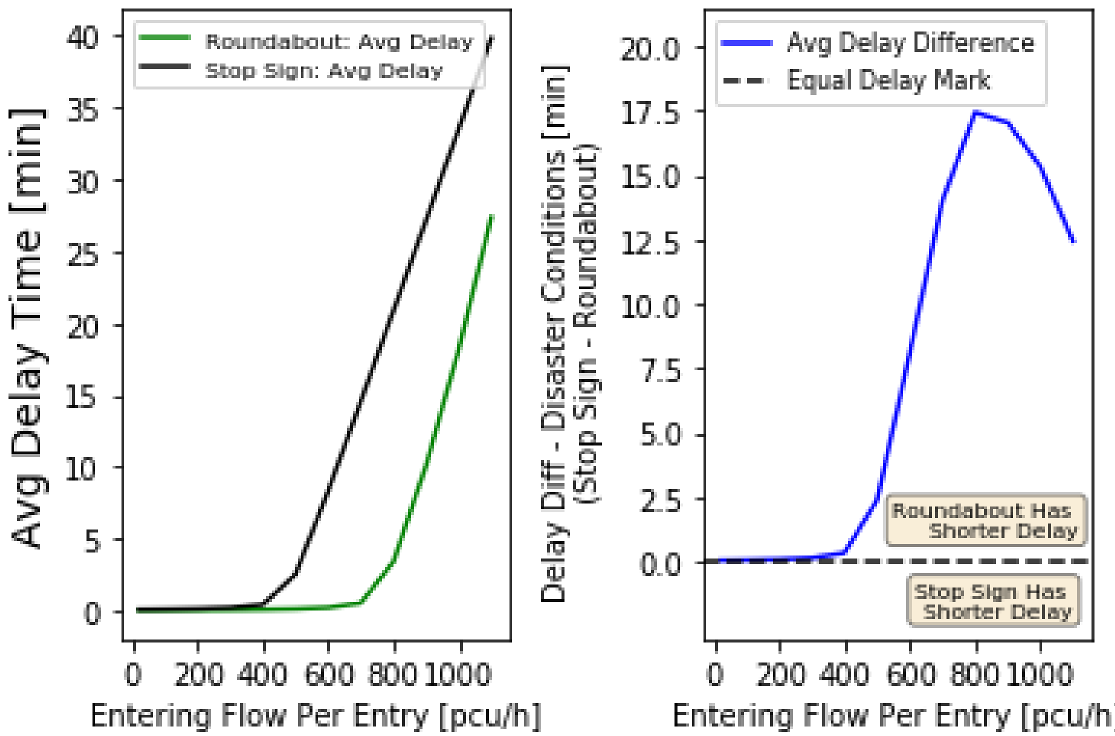

Figure 4.4. Equal Entering Flow—Roundabout & Stop Sign Resilience Comparison. Left: comparison of roundabout and stop sign average delay as function of entering flow given equal entering flow and exits ratio. Right: the difference between these two curves. Roundabouts and stop signs have similar delays for very light traffic ($Q_e \leq 400\frac{pcu}{h}$). For intermediate traffic and above ($Q_e \geq 500\frac{pcu}{h}$), roundabouts have shorter average delay time of up to 20 minutes. For very heavy traffic ($Q_e \geq 900\frac{pcu}{h}$), the difference is reduced.

Efficiency and Resilience Comparison

Figure 4.5 compares resilience and efficiency tradeoffs in delay time. In a post-disaster scenario, the average delay difference between roundabout and stop sign (left) shows that a roundabout has a shorter waiting time for traffic flow by roughly the same amount as the average delay different between traffic light and roundabout in a non-disaster scenario. These differences only matter when flows exceed certain thresholds, and are not significant for low values of Q_e. Table 4.3 summarizes the resilience and efficiency advantage for each intersection design. Here, we identify that roundabouts are preferred for all situations except high traffic conditions during normal day-to-day traffic.

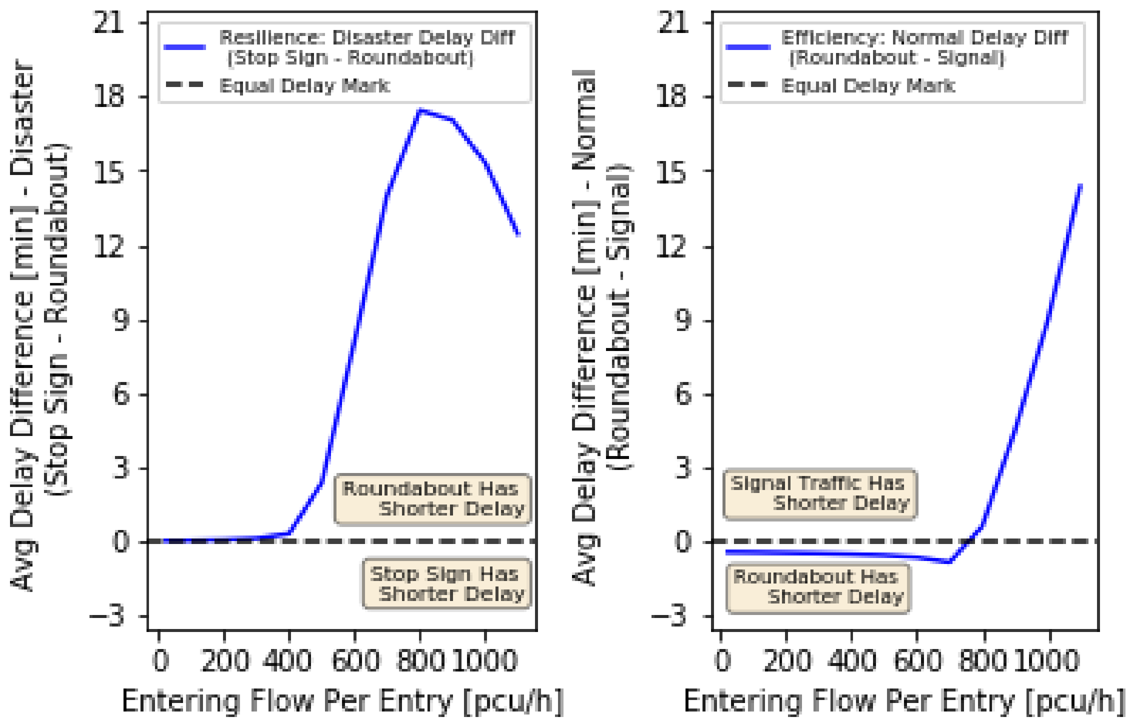

Figure 4.5. Equal Entering Flow—Roundabout & Signal Resilience and Efficiency Comparison. Left: comparison of roundabout and traffic signals average delay after a disaster (i.e., when signals act as stop signs). Right: comparison of roundabout and traffic signals during normal operations. The left graph demonstrates the resilience benefits of roundabouts where the right graph demonstrates the efficiency benefits for signals. Roundabouts are more resilient than signals regardless the entering flow, with shorter delay times for high traffic ($Q_e \geq 900\frac{pcu}{h}$). Signals are more efficient than roundabouts for heavy traffic during normal operations.

Entering Flow [pcu/h]	Efficiency Shorter Delay time in Normal Conditions	Resilience Shorter Delay time in Disaster Conditions
0-250	Roundabout	Roundabout
250-500	Roundabout	Roundabout
500-750	Roundabout	Roundabout
750-1000	Signal	Roundabout

Colored Time Difference Scale [min]
<0.5 minutes
0.5-5 minutes
>5 minutes

Table 4.3. Equal Flow Intersection—Resilience and Efficiency Advantage. We show the superior intersection design with respect to delay time based on traffic flows and time differences in minutes. Roundabouts are preferred for all situations except high traffic conditions during normal day-to-day traffic.

4.1.3 Case (b): Unequal Flow over Entries

Conclusions for Case (a) only hold assuming that traffic flows are equal over all entries. In many cases, Q_{ei} are different for different entries, such as in intersections that connect a major road with lots of traffic to a minor, local road with less traffic. We examine the effect of unequal traffic flows to assess how conclusions about efficiency and resilience hold for more realistic traffic conditions.

Efficiency: Roundabout vs. Traffic Signals

We estimate the additional delay time for roundabout and signalized intersection given a change in some of lanes where the rest of lanes remained with a base fixed entering flow. We chose the base flow for this estimation to be 500 $\frac{pcu}{h}$ since up to this flow there was no significance difference between roundabout to signalized intersection (less than half a minute).

Figures 4.6 and 4.7 show the average delay time for intersections with unequal flows. In all cases, having fewer entries with high flows reduces overall delay time. The total delay time for a roundabout increases as more entries have greater flow than a traffic signal. This rate of increased delay is due to the effects of circular flow within the intersection. We further plot curves for two entries to account for differences in delay time when entries with high flows are adjacent (e.g., entries (1) and (2)) or opposite each other (e.g., entries (1) and (3)) in the intersection. This difference in entering flow has a relatively small effect on delay time. For signalized intersections, Figure 4.7 shows similar effects of increased average delay time due to more entries with traffic.

We compare each case in Figure 4.8 by calculating the relative delay for each traffic situation. Here, we find that roundabouts are more efficient for high traffic flows when those flows are only entering one or two entries. This is in contrast to results presented with traffic flows equal over all entries. However, traffic lights are still more efficient when there are high traffic flows over 3 or more entries. Moreover, the relative efficiency for roundabouts remains small (less than 3 minutes), where the relative efficiency for traffic lights is greater when more efficient (almost 8 minutes for 3 entries).

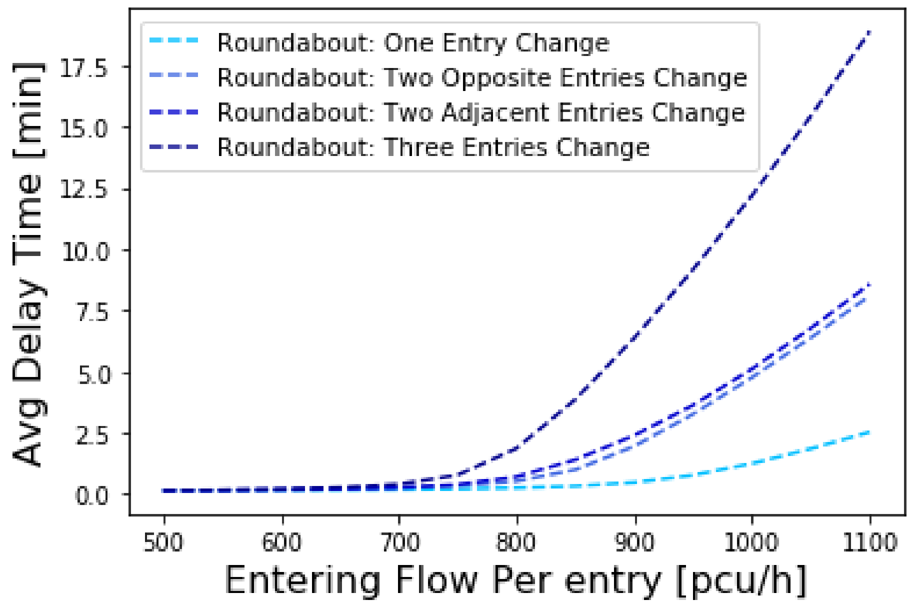

Figure 4.6. Unequal Entering Flows—Roundabout Average Delay. Roundabout average delay as function of entering flow given equal unequal entering flows with equal exits ratio. Some entries have fixed entering flow ($Q_e = 500$) while the rest vary ($500 \leq Q_e \leq 1100$). Increasing flow across different combination of entries leads to different increases in average delay time.

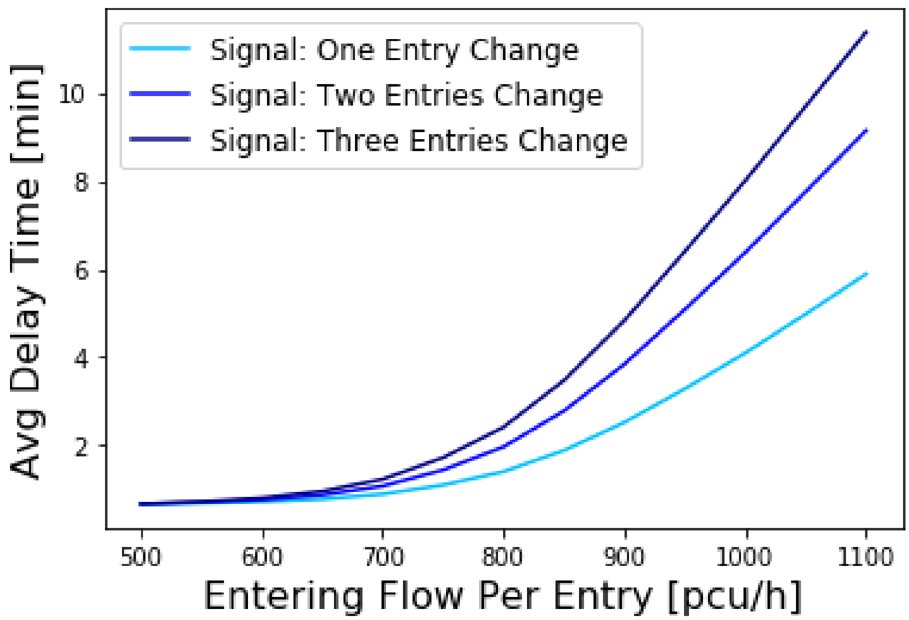

Figure 4.7. Unequal Entering Flows—Signal Average Delay. Signal average delay as function of entering flow given unequal entering flows and equal exits ratio. Some entries have fixed entering flow ($Q_e = 500$) while the rest vary ($500 \leq Q_e \leq 1100$). Unlike roundabout, entries do not interact with each other, and each additional congested entry affects the average delay the same.

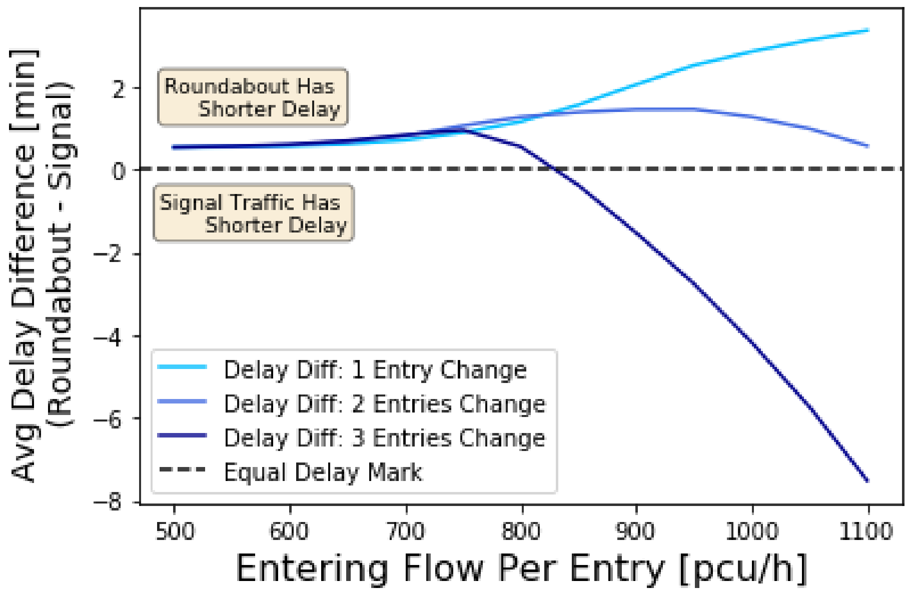

Figure 4.8. Unequal Flows—Roundabout & Signal Average Delay Difference. The difference in average delay time between roundabouts and signals in minutes as a function of entering flow given unequal flow over entries and equal exit ratio. Roundabouts are more ecient for high trac flows when vehicles enter the intersection from one or two entries. In contrast, when vehicles enter 3 or more, trac signals are more ecient.

Resilience: Roundabout vs. Stop Sign

We compare roundabouts to stop signs with unequal traffic flows to characterize relative resilience of each intersection. Figure 4.9 shows the excepted delay time for stop signs with respect to the number of entries with traffic flows. Here we see a more linear relationship between entering flow and delay time. Moreover, increased flow over more entries leads to greater delay times, similar to both roundabouts and traffic signals during normal operations. We compare stop signs to roundabouts in Figure 4.10 to determine resilience trade offs. Here, we find roundabouts to be more efficient for all traffic situations. In particular, situations where fewer entries have traffic flow, and roundabouts experience less circular flow, lead to roundabouts having even greater efficiency benefits than in situations where all entries have traffic.

Efficiency and Resilience Comparison

Results presented in Figures 4.8 and 4.10 establish that our conclusions summarized in Table 4.3 hold for unequal traffic flows. Overall, we find roundabouts to be more efficient than traffic lights, except when vehicle flows are high during peak periods. Moreover, we find that situations where only one or two intersection entries have traffic, roundabouts are more efficient than traffic lights for all entring flow values. However, this relative difference is small. When traffic lights are more efficient than roundabouts, their absolute difference is large. In contrast, roundabouts are always more efficient than traffic lights after a disaster. This makes roundabouts more efficient resilient than traffic lights and are preferred in most four-way, single-lane intersections with unequal flows. Traffic lights may be more preferred for intersections that do not experience disasters and have high traffic over many entries.

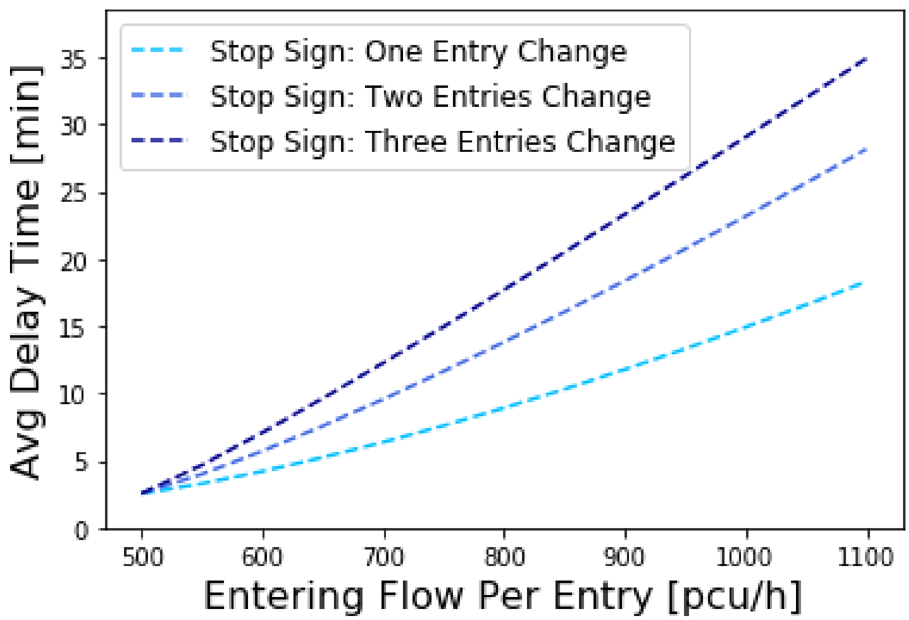

Figure 4.9. Unequal Entering Flows—Stop Sign Average Delay. Stop sign average delay as function of entering flow unequal entering flows. Some entries have fixed entering flow ($Q_e = 500$) while the rest vary ($500 \leq Q_e \leq 1100$). Like signals, stop sign entries do not interact within the intersection. Hence, each additional entry affects the average delay the same.

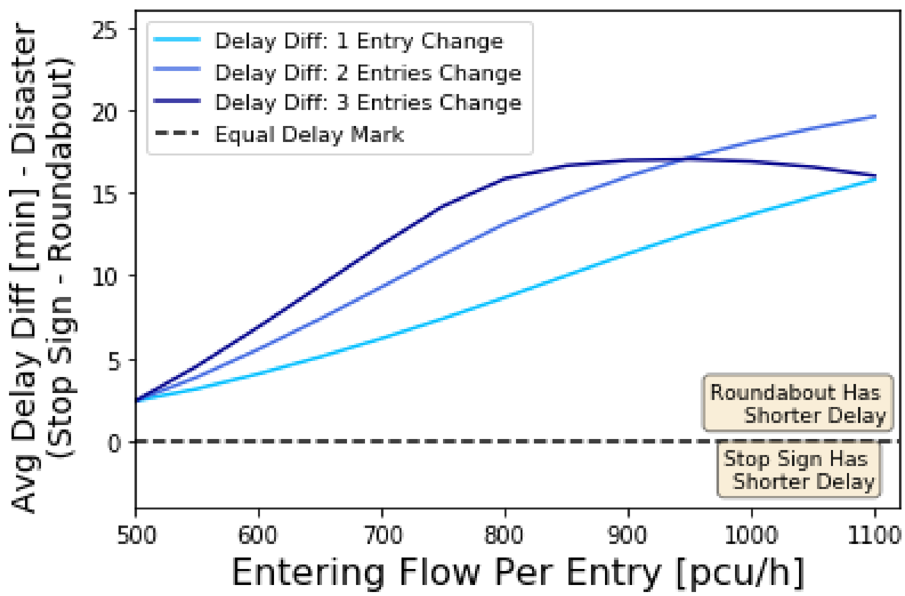

Figure 4.10. Unequal Entering Flows—Roundabout & Stop Sign Average Delay Difference. The difference in average delay time between a round-about and stop sign given unequal entering flows and equal exits ratio. Roundabouts are more efficient than stop signs for all examined entering flow situations.

4.2 Efficiency vs. Resilience of Intersections in the USVI

We assess the effects of intersection design for four realistic intersections and flow scenarios in the USVI. We choose these intersections since they are (1) points of interest in the territory and represent central intersections, (2) they have high range of flow rates from very low to exceeding normal intersection capacity, and (3) since some of them might get highly congested in general and in post-disaster scenario in particular they are candidate to consider converting from signals (as they currently are) into roundabout.

4.2.1 Modeling and Assumptions

We build on the assumptions made for generic four-way intersections to analyze realistic intersections in the USVI. First, we generate the expected flow rate for each intersection based on the model presented in Section 3.1. Flow values were acquired from studies developed and implemented by Good (2019) for STX and Routley (2020) for STT. This approach estimates the expected flow rate for each road on each island given high traffic flows during post-disaster curfews and with origin-destination pairs defined by the post-disaster supplies communities need to access on a day-to-day basis. Resulting flows for roads estimate the round trip congestion traveling through intersections. While these flows are more appropriate for disaster scenarios rather than daily traffic, the flows capture worst-case effects of all drivers on the roads at the same time. We assume traffic during these scenarios is comparable or worse than daily peak traffic conditions. Thus, the assumed flows are helpful for both pre- and post-disaster analysis.

Additional assumptions are needed to calculate delay time for signalized intersections. We assume that VIDPW sets intersection signal timing based best practices in the literature, and obtain the signal timing and phasing each lane group based on the intersection flow with the following procedure:

1. We define a major and minor flow direction for each intersection. The major direction is the pair of entries with the largest entering flow rate.
2. We set the green cycle time as 60% and 40% of the cycle for major and minor flow directions, respectively.
3. For each entry, we then divide the green cycle time for right turn and straight/left lane groups using the same strategy for entries, but with signal timing weighted by entering flow rates. We do this for parallel (counter flow) entries and choose the entry with the greatest flow as the cycle time for both.

In addition to these assumptions, each case study intersection has idiosyncrasies that make them interesting to model. These idiosyncrasies require additional assumptions, outlined for each study with their analysis.

4.2.2 Case (c): University of the Virgin Islands and Centerline Rd., St. Croix

We choose to study an signalized four-way intersection near UVI campus on STX because it has low traffic flow rates, but is on a major road for the island. Figure 4.11 presents an image of the intersection modeled and the node splitting representation with entry labels and flows from Good (2019). For this case study, we set the green light ratio for a full 90 seconds cycle and lost time $t_{start} + t_{clear} = 4$ seconds per phase.

Table 4.4 presents the expected delay time in minutes for Case (c) as a roundabout, traffic signal, and stop sign. Results show that there is limited delay time for all intersection times given the low number of vehicles entering the intersection. Traffic signals have the greatest delay times, so we present results for lane groups to show the effects of green timing. For this case, roundabout and stop sign intersections experience similar delay time regardless the turn they need to make. Signals experience the greatest delay time for right turns at entry (1). Still, this delay time is less than a minute, and will not have significant effects on traffic or total travel time for a vehicle.

Overall, we cannot make a clear recommendation to change intersection design. In fact, based on these results, we may expect faster travel times after a disaster as the stop sign model produces lower delay times than the signalized intersection.

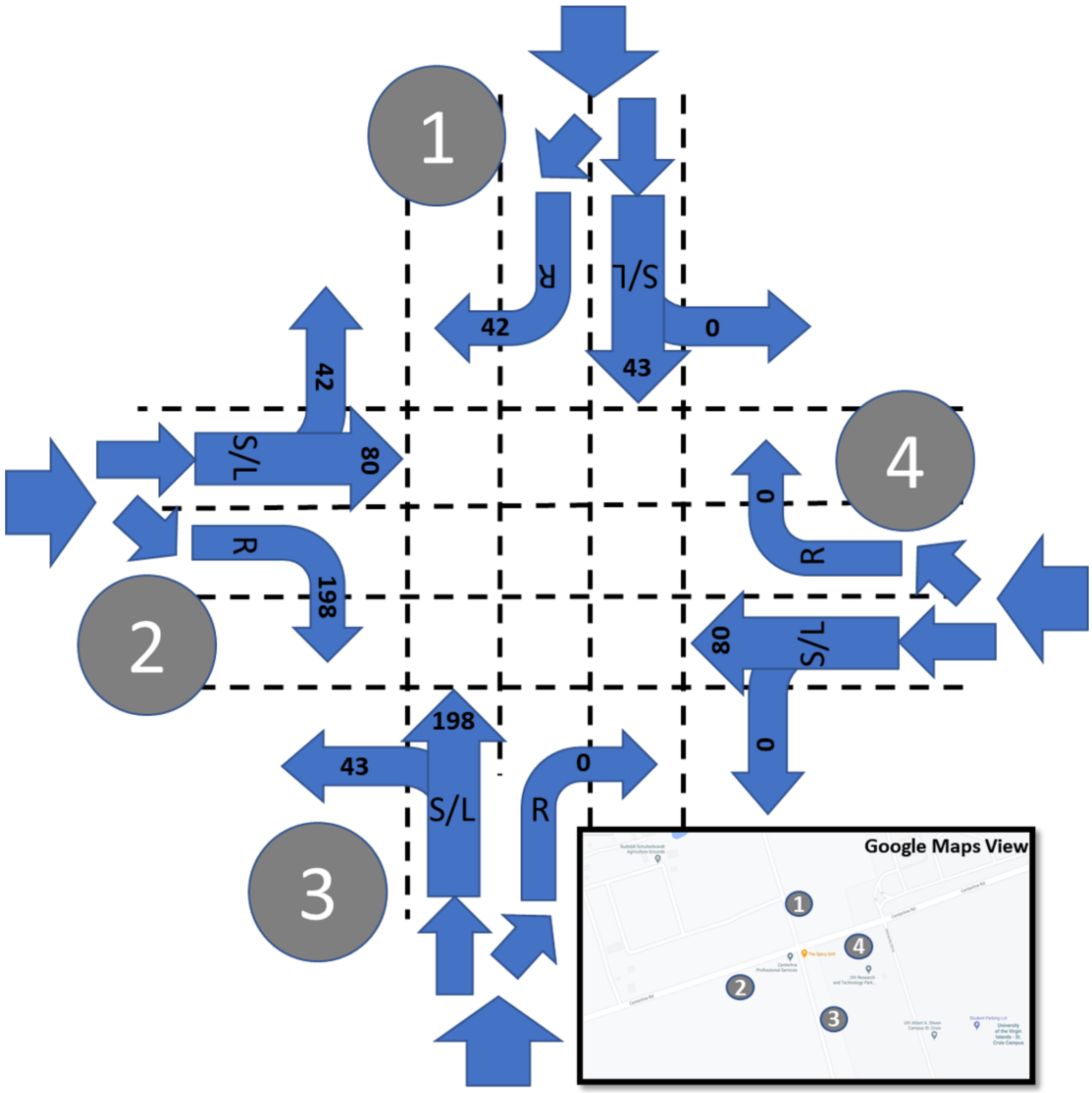

Figure 4.11. UVI Campus & Centerline Rd. Intersection Illustration and Flow Rates. Source for inner map: Google Maps (2020).

UVI Campus Intersection Expected Delay Time [minutes]									
Intersection Structure	Entry 1		Entry 2		Entry 3		Entry 4		Weighted
	S/L	R	S/L	R	S/L	R	S/L	R	Avg
Roundabout	0.06	0.06	0.07	0.07	0.06	0.06	0.06	0.06	0.06
Signal	0.45	0.79	0.71	0.33	0.56	0.65	0.64	0.29	0.54
Stop Sign	0.11	0.11	0.24	0.24	0.17	0.17	0.11	0.11	0.19

Table 4.4. UVI Campus and Centerline Rd. intersection comparison. When traffic flow is low, delay times are relatively short. We find an average delay of half a minute or below for all intersection designs.

4.2.3 Case (d): Queen Mary Highway and Centerline Rd., St. Croix

The Queen Mary Highway and Centerline Rd. intersection on STX is a signalized intersection with relatively high flow rate. Moreover, there is a gas station located near the intersection, which is an important destination for vehicles during normal traffic and after a disaster. Figure 4.12 describes the intersection and the estimated entering flow rates driving through it based on Good (2019).

The presence of the gas station changes the effects of entering flows. As shown in Figure 4.12, the gas station is located on the corner between entry (1) and (2). Based on the model from Good (2019), some of the flow entering this intersection is destined for the gas station, which complicates intersection delay time calculations. Figure 4.12 shows that only entries (2) and (4) have flows towards the gas station. We make the following assumptions for these vehicles:

1. We assume cars entering from (4) to travel o the gas station a car must exit through either entry 1 or entry 2 prior to exiting the road and entering the station. We assume half of these cars entering (4) exit through (1) and the other half through (2).
2. We assume cars entering (2) and destined in the gas station as exiting the intersection from entry (1). We make this assumption to include cars entering the S/L lane group for turning into the gas station as affecting signalized intersections. In contrast, this flow will not affect roundabouts as this does not generate circular flow that blocks entries.
3. We assume cars leaving the gas station exit towards entry (2) and do not impact intersection delay time.

In addition, we model the Queen Mary Hwy & Centerline Rd. intersection with a longer signal cycle time (100 seconds) and smaller lost time of 3 second) to match the high speeds associated with the highway roads.

Table 4.5 presents the expected delay time in minutes at the Queen Mary Hwy and Centerline Rd. intersection as a roundabout, traffic signal, and stop sign. The expected delay time for a roundabout are 0.2 and 1.4 minutes in entries (3) and (4) respectively, and 7.8 and 17.8 minutes for entries (1) and (2) respectively. This large difference in delay time across entries leads to a weighted average delay time of 8.7 minutes across all four entries. In contrast, the average expected delay time for a traffic signal are about two times higher at $\approx$ 15 minutes. In the case of stop sign the delay time are even higher with average of 34 minutes per entry per car, and long delay time are expected across all entries besides entry 3 which has low entering rate.

The increased delay time for the traffic signal is expected as the majority of vehicle traffic is traveling through a single entry. However, the green phases and turns also play a role in this calculation In our model we implemented mirrored green time phases, which means a green light for entries 1 & 3 and 2 & 4 simultaneously for similar lane group R or S/L. Therefore, it might be the case that when one direction is busy, the other one might be empty and the green timing is not optimized to reduce delay. Moreover, a roundabout is efficient with a high rate of left turns (and driving straight) since the circulating flow does not block other entries. Together, the large entering flow $Q_{e2} = 830$ still leads to the roundabout being more efficient.

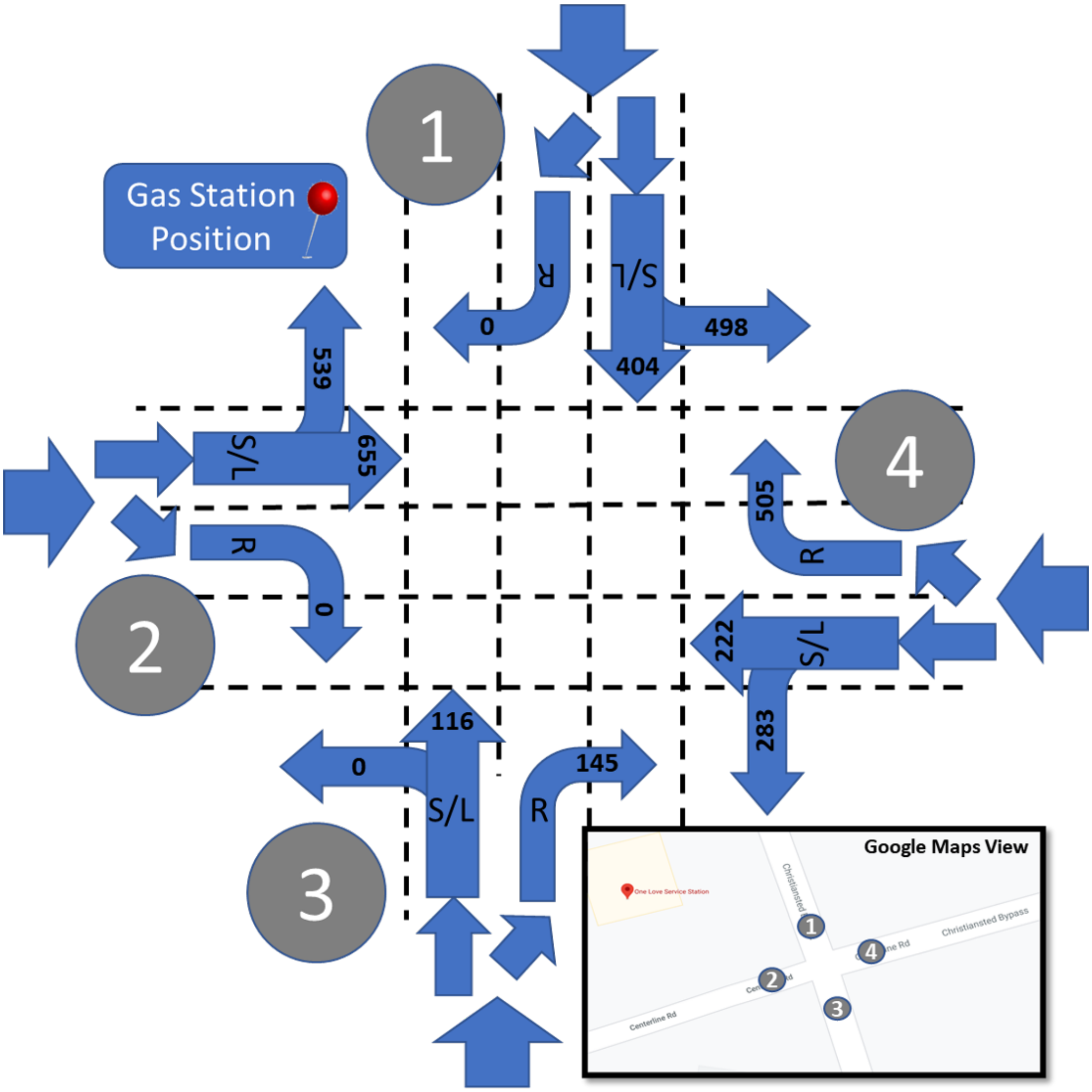

Figure 4.12. Queen Mary Hwy & Centerline Rd. Intersection Illustration and Flow Rates. Source for inner map: Google Maps (2020).

Queen Mary Intersection Expected Delay Time [minutes]									
Intersection Structure	Entry 1 S/L	Entry 1 R	Entry 2 S/L	Entry 2 R	Entry 3 S/L	Entry 3 R	Entry 4 S/L	Entry 4 R	Weighted Avg
Roundabout	7.8	7.8	17.8	17.8	0.2	0.2	1.4	1.4	8.7
Signal	18.3	0.7	18.0	0.6	0.5	10.1	0.5	20.8	15.0
Stop Sign	27.2	27.2	45.7	45.7	0.2	0.2	34	34	33.9

Table 4.5. Queen Mary Hwy and Centerline Rd. Intersection Comparison. Roundabout has a shorter average delay when compared to traffic signals and stop signs. However, delay time depends on lane group and entry. For example, right turns for entry (1) are much faster with traffic signals. Similarly, entries (2) and (4) each have one lane group where signals are more efficient due to limited traffic flow.

4.2.4 Case (e): Weymouth Rhymer Hwy & Alton Adams Dr. / Rumer Dr., St. Thomas

This intersection is four-way signalized intersection on STT where three major streets cross (Weymouth Rhymer Hwy crosses Alton Adams Dr. and Rumer Dr.) and one entrance into the Lockhart Gardens Shopping Center. The intersection is also located near the hospital on STT. Together, the intersection is relatively congested and has a central role in getting through the area of Charlotte Amalie East. Figure 4.13 describes the intersection and the estimated entering flow rates driving through it acquired from Routley (2020).

For this intersection, we assume a signal cycle of 100 second with 2 seconds of lost time for each phase. These assumptions match those of Section 4.1.

Table 4.6 presents the expected delay time for roundabout, traffic signals, and stop signs. We find the expected delay time for roundabouts and signals to both be less than 1 minute for all directions, with roundabouts have the shortest delay times. In contrast, we find the stop sign has some entries with delays shorter than a traffic light or with a significant delay of several minutes. The expected delay time for entry (2) is more than 10 minutes, which makes the weighted average edlay time as ≈ 5 minutes.

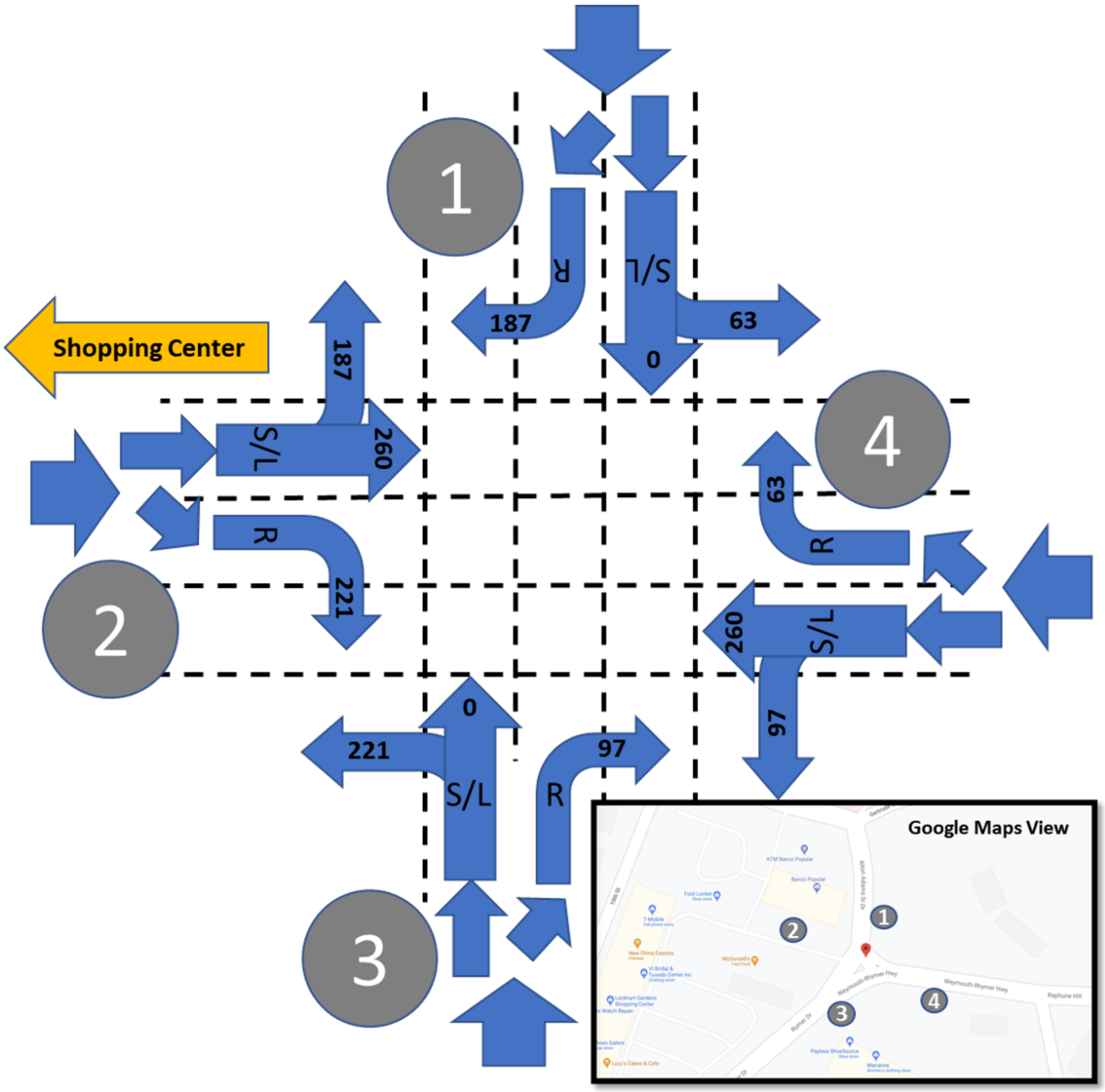

Figure 4.13. Weymouth Rhymer Hwy Intersection Illustration and Flow Rates. Source for inner map: Google Maps (2020).

Weymouth Rhymer Intersection, St. Thomas - Expected Delay Time [minutes]									
Intersection	Entry 1		Entry 2		Entry 3		Entry 4		Weighted
Structure	S/L	R	S/L	R	S/L	R	S/L	R	Avg
Roundabout	0.1	0.1	0.11	0.11	0.1	0.1	0.1	0.1	0.1
Signal	0.48	0.67	0.38	0.65	0.59	0.57	0.34	0.51	0.49
Stop Sign	0.18	0.18	12.55	12.55	0.24	0.24	0.55	0.55	5.27

Table 4.6. St. Thomas, Weymouth Rhymer Hwy & Alton Adams Dr. / Rumer Dr. Intersection Comparison. Average delay time for roundabouts and traffic signals are short, much less than a minute. In contrast, heavy traffic in entry 2 makes the average delay for a stop sign to be greater than 5 minutes.

4.2.5 Case (f): Tutu Choke Point—Weymouth Rhymer Hwy and Merhindal Rd., St. Thomas

This intersection is four-way signalized intersection with three-way streets (Weymouth Rhymer Hwy crosses Merhindal Rd.) and one entrance into a shopping center STT. It is similar to Case (e), however it has significantly more vehicle traffic during a disaster. It a central location in getting from population areas to goods and supplies for the entire eastern side of STT. Figure 4.14 shows the intersection and the estimated entering flow rates driving through it based on Routley (2020).

Table 4.7 presents the expected delay time in minutes for roundabouts, traffic signals, and stop signs. We find significant variation in delays based on intersection entry and lane group. For example, entry (1) S/L has over 10 minutes of additional delay for a signal over a roundabout. In contrast, entry (1) R is nearly 10 minutes faster for a signalized intersection. Similarly, signalized intersections are over 5 minutes faster than roundabouts for entry (4). Moreover, entries (1), (3), and (4) have large delays greater than 5 minutes for most intersections, but entry (2) is very fast for all intersection designs. The high variability in delay times leads to roundabouts having a lower weighted average, yet higher delay times for most entries compared to a traffic signal. Stop signs appear to cause significant delays, with entry (1) requiring upwards of 1 hr to drive through.

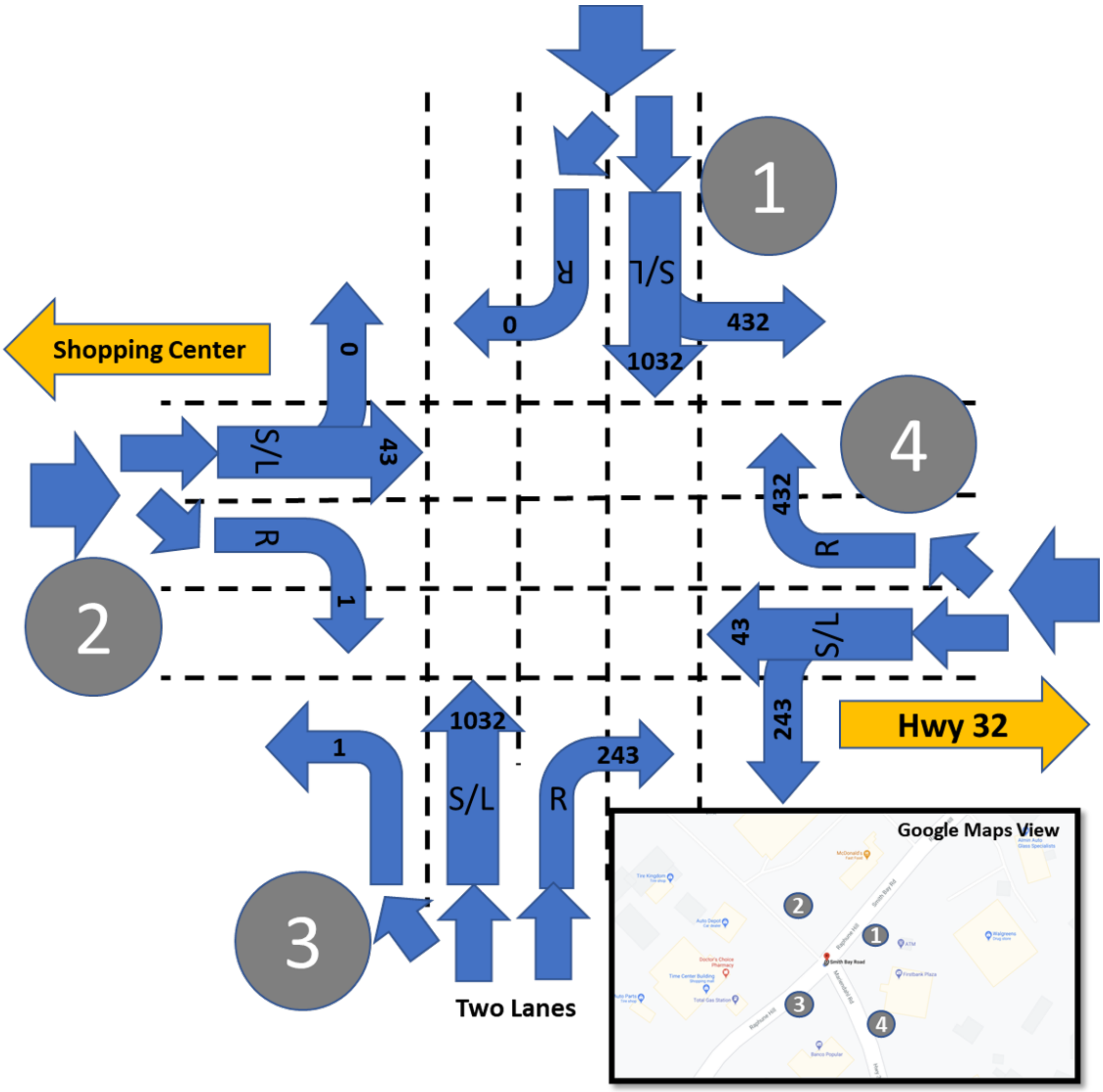

Figure 4.14. Tutu Choke Point Intersection Illustration and Flow Rates. Source for inner map: Google Maps (2020).

Tutu Choke Point, St. Thomas - Expected Delay Time [Minutes]									
Intersection	Entry 1		Entry 2		Entry 3		Entry 4		Weighted
Structure	S/L	R	S/L	R	S/L	R	S/L	R	Avg
Roundabout	10.0	10.0	0.2	0.2	10.5	10.5	6.2	6.2	9.3
Signal	21.4	0.6	0.6	0.5	6.2	9.8	0.7	0.6	11.1
Stop Sign	63.0	63.0	0.1	0.1	51.0	51.0	15.7	15.7	48.1

Table 4.7. St. Thomas, Tutu Choke Point—Weymouth Rhymer Hwy and Merhindal Rd. Intersection Comparison. Roundabouts and signal have similar average delay. However, this number is due to significant congestion at entry (3). Average delay for a stop sign is almost five t imes g reater t han both roundabouts and signals.

4.2.6 USVI Intersection Comparison

Table 4.8 summarizes the mean expected time delay in each intersection in the four case studies for different intersections in STX and STT. Table 4.8 shows that the traffic flow and intersection geometry has an important effect on total travel time. Whereas all four intersections are located on major roads, Case (c) and (e) experience short delay times and Case (d) and (f) experience much longer delay times. The benefits of roundabouts, traffic signals, and stop signs are also different for each case. For example, stop signs are faster than traffic signals for Case (c) based on our model, but much slower for all other cases. Roundabouts have a shorter delay time than traffic signals for all intersections, but this does not hold for all entries within intersections. Together, realistic case studies on intersections in the USVI demonstrate how difficult it is to determine which design is preferred and for what reasons.

Table 4.9 presents the efficiency-resilience trade offs for case study intersections to determine the total benefits of each intersection over another. In general, the benefits of changing intersection designs to manage disasters depends on expected flows. Case (c) and (e) have low vehicle flows, whereas Case (d) and (f) have much higher flows. With low flows signals are less efficient than roundabouts, and even sometimes less efficient than stop signs. As the flows are greater in (d) and (f) the benefits of roundabouts are more pronounced. This is due to several reasons.

First, roundabout are more robust to changes in entering flows when they occur in specific

Case Study	Mean Expected Delay Time [minutes]		
	Roundabout	Signals	Stop Sign
UVI & Centerline Rd. (c)	0.07	0.53	0.18
Queen Mary Hwy & Centerline Rd (d)	8.7	15.0	33.9
Weymouth Hwy (Lockhart Gardens) (e)	0.10	0.48	5.27
Weymouth Hwy (Tutu Choke Point) (f)	9.3	11.1	48.1

Table 4.8. Mean Expected Delay Time in Examined Case Studies for the USVI. Roundabouts have shorter delay time across all examined case studies. Roundabouts are particularly efficient when traffic is high across few entries, such as Case (d). Roundabout benefits are reduced in light traffic such as Case (c) and (e), or when many entries have high traffic like Case (f). Stop signs are inefficient in all cases besides Case (c).

directions (less than two entries) and with specific turns (left turns). Second, our assumptions for mirrored signal phases is not the optimal signal phasing opposing entries have significantly different values of Q_e. Third, there are relatively limited right turns in the intersection, which means circular flows do not block other entries.

Overall in all examined cases roundabouts have the advantage of both efficiency and resilience when analyzing flow rate based in post-disaster conditions.

Case Study	Mean Expected Delay Time Diff [minutes]		
	η_1^{rel} Roundabout vs. Signals	σ_1 Roundabout vs. Stop Sign	σ_2 Signal vs. Stop Sign
UVI & Centerline Rd. (c)	0.47 (Roundabout)	0.12 (Roundabout)	0.35 (Stop Sign)
Queen Mary Hwy & Centerline Rd. (d)	6.3 (Roundabout)	25.2 (Roundabout)	18.9 (Stop Sign)
Weymouth Hwy (Lockhart Gardens) (e)	0.38 (Roundabout)	5.17 (Roundabout)	4.79 (Signal)
Weymouth Hwy (Tutu Choke Point) (f)	1.8 (Roundabout)	38.8 (Roundabout)	37 (Signal)

Table 4.9. USVI Case study summary. η_1^{rel} is the average delay time difference between a roundabout and traffic signals, σ_1 is the average delay time difference between roundabout and traffic signals as stop signs, and σ_2 is the comparison of traffic signals before and after the disaster. All the results presented in minutes. All the scores represent the additional delay time for the slower intersection type, where the more efficient intersection is written in brackets. For example, η_1^{rel} for Case (c), UVI and Centerline Rd. is 0.47 min additional delay time for traffic signals (roundabouts are more efficient).

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CHAPTER 5:
Conclusions

We build on our results in Chapter 4 to provide general conclusions on intersection design, efficiency, and resilience. We also present the main limitations of our model and describe ideas for further research.

5.1 Conclusions and Recommendations

For the general case, we conclude that roundabouts, traffic signals, and stop signs each have their own ideal operating range of traffic entering the intersection. Within each operating range, the intersection will outperform the alternatives. Assuming traffic flows and intersection parameters are known, a superior alternative for an intersection can be measured based on delay time. For example, in the general case of a four-way intersection with equal flow entering from each direction, a roundabout has a shorter delay time than signal when traffic is light ($Q_e \leq 800$ [pcu/hr]). However, above this threshold, a roundabout would have longer expected delay time than a traffic signal. In the same general case, roundabouts are more efficient than a traffic light after a disaster (i.e., a stop sign). The larger the entering flows become, the greater the difference between the expected average delay time of roundabout and stop sign. This trend lasts until the threshold entering flow of about 850 [pcu/h] where the circulating flow gets higher and correspondingly the roundabout capacity is reduced.

We also conclude from the general case that traffic signals are far less robust to disaster than roundabouts. We find significant differences in delay time for traffic signals and stop sign. Assuming traffic signals act as stop signs after a disaster, this additional delay time reflects the resilience of the intersection to failure. However, stop signs outperform traffic lights in some situations, suggesting they operate with shorter delay times after a disaster. This is true for very low traffic conditions ($Q_e \leq 400$ [pcu/hr]).

Overall, the results for USVI case studies match the results for the generic intersection case, but emphasize that roundabouts are generally better for the territory. We find roundabouts are more efficient and robust for all realistic intersections. However, the benefits of roundabouts only truly matter for intersections with significant traffic flows. In general, no intersection studied has large traffic flows approaching the intersection for all entries. Since only one or two entries have high traffic, roundabouts are most efficient intersection on average. Where traffic flows are large, roundabouts can reduce total travel time by many minutes in pre-storm and post-storm scenarios. Thus, we find roundabouts to be generally better in all cases, especially when disasters lead to signals failing.

Based on our analysis, we recommend that at least one intersection on STX — Queen Mary Highway and Centerline Rd. – and one on STT — Tutu Choke Point – should consider changing their design to a roundabout. In both cases, the direction of traffic flows, the high number of vehicles, and the geometry of the intersections lend themselves to roundabouts for efficiency. Moreover, they are located adjacent to critical supplies and will be important after a major disaster. These intersections could achieve reductions in delay time of greater than 25 minutes during these situations with a roundabout.

5.2 Limitations

Several assumptions embedded in our delay time models influence our final conclusions are worthy of scrutiny and further analysis. First, the recommendations are highly dependent on traffic flows, which are from network flow models rather than real-world data. We analyze intersections based on studies by Good (2019) and Routley (2020), rather than traffic information from VIDPW. This is partly due to a lack of data for this form of analysis. It is also to try and assess pre- and post-disaster conditions, for which limited data exists. However, modeled traffic make strong assumptions about origin-destination pairs and resulting intersection flows and turns. Significant deviations in assumed flows and real flows may alter results for all intersections.

Our models require analysis to be far from intersection saturation, but our most important conclusions on delay time occur near or above saturation. High traffic conditions lead to greater roundabout benefits. However, the closer we get to the saturation threshold for an intersection model, the model is less trustworthy. For this reason, we may be underestimating total delay times for all intersections. Further work may focus on developing more accurate models as the number of vehicles entering roundabouts and traffic signals reaches the assumed capacity.

Our analysis assumes traffic flow rates are constant for large travel periods ($T = 1h$). The USVI may experience very sharp increases and decreases in traffic conditions during peak hours. More accurate assessment of traffic flows and characteristic analysis times is necessary to match models to context.

We ignore some delay time measures that may be salient for comparing intersections. For a roundabout we neglect the delay time caused by approaching speed, low circulating speed, or longer circular path to cross to exit the intersection. These factors lead to additional delays caused by circular flow that we are not considering. Their inclusion in analysis may shift the roundabout delay curves to make signals look more favorable for lower flow rates.

When modeling stop signs, we only consider startup time for drivers to enter the intersection when it is their turn, but we do not consider time to clear the intersection between cars. Thus, our results for expected delay time for stop signs might be more optimistic (shorter) than what it might be in reality.

There are several assumptions that may improve or worsen the delay time measured for traffic signals that makes them more beneficial than roundabouts. We assume pre-timed signals, rather than adaptive signal timing based on uneven traffic flows. This is inefficient and can be overcome with common traffic signal infrastructure, potentially dramatically reducing delay time. Moreover, in our current model, we assume that a single lane splits into a protected lane groups without interactions. With delay times of minutes, we may expect real-world traffic queues to exceed the length of these protected lanes and interacting. This spillover effect would increase total delay time for all lane groups for a single entry.

5.3 Further Research

In addition to studies that overcome possible issues with model assumptions, there are several avenues of work that this thesis supports. First, while we developed methods to extend our analysis and implement it into a multi-commodity network flow model, we did not complete this implementation or assess the effects of intersections on traffic routing. Future work can build on this thesis to develop a multi-commodity flow model with intersection traffic delays. This future model will support the assessment of intersection design on traffic routing before and after failures due hurricane disasters and related effects (e.g., blackouts).

We also recommend that future studies continue to assess efficiency-resilience tradeoffs in infrastructure systems. There are few studies assessing efficiency-resilience tradeoffs for transportation systems and critical infrastructure in general. Ganin et al. (2017) is one of the only studies showing trade offs for transportation, but focuses entirely on road networks. Some of the conclusions from this work on network redundancy and resilience requires further consideration of intersection effects. This work is a step towards understanding how intersections influence these tradeoffs. Moreover, this work provides a framework for considering similar questions for other systems that also have efficiency-resilience tradeoffs, such as telecommunications networks and water distribution systems.

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