TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS
DEPARTMENT OF CIVIL ENGINEERING

FINAL YEAR PROJECT REPORT on "REVIEW OF DETAILED ENGINEERING DESIGN DOCUMENT OF PUNYAMATI-CHANDESHWORI SECTION OF ARANIKO HIGHWAY"

By:

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Supervisor:

Asst. Prof. Anil Marsani

APRIL 2023

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IN PARTIAL FULFILLMENT OF THE REQUIREMENT FOR THE AWARD OF A BACHELOR'S DEGREE IN CIVIL ENGINEERING
(Course Code: CE755)

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CERTIFICATE

This is to certify that this project work entitled **"REVIEW OF DETAILED ENGINEERING DESIGN DOCUMENT OF PUNYAMATI-CHANDESHWORI SECTION OF ARANIKO HIGHWAY"** has been examined and declared successful for the fulfillment of academic requirements towards the completion of Bachelor Degree in Civil Engineering.

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ABSTRACT

The main aim of this project work is to review the Design document of the highway section abiding by the codes given by DOR and NRS 2070 and to incorporate drainage requirements, design traffic signals, management of displaced parking facilities, and addition of road safety elements for safe operation of traffic in highway section that enhance the overall urban environment. This report highlights the limitations of the original design document which fails to address the issues of flooding, accessibility, management of displaced vehicles and overall safety aspects of all road users ranging from vehicles to pedestrians. This project attempts to address all these issues through the identification of alternatives to be proposed carrying out detailed analysis, validation and study of implication part of the alternatives proposed.

Here in our study, we used ArcGIS for hydrological delineation to estimate flooding of various return periods, micro-analytical tool Sidra Intersection, version 8.0 for signal design and flow analysis, land use mapping through GNSS for management of displaced parking facilities and use of iRAP demonstrator for star rating in both proposed and reviewed elements of highway. From our study, the size of bridges at Pulbazar, Chardobato and Chandeshwori were calculated to be nominal with respect to design document. The size of side drains in Pulbazar-Chandeshwori section were calculated as per requirement. The intersection of Tindobato, a major miss in original design document, was again designed with traffic signals considering the total traffic volume of crossing. The design of Chardobato intersection was re-evaluated for different signal timing and lane grouping to develop a network that provide less delays and higher level of service in the network. A bunch of safety elements that were lacking in original document were added which in return improved the star rating significantly. The survey of parking facilities showed sufficient facilities available for displaced vehicles. With the hope that the design document, its recommendations, reassessment of highway elements, estimates, and drawings will be ample for justification for carrying out changes and modifications in actual implicative design of the project. And finally, this report is submitted to Department of Roads Suryabinayak- Dhulikhel sub-division and Municipal Office, Banepa for its implication.

NOTATION AND ABBREVIATIONS

AADT	Average Annual Daily Traffic
ADT	Average Daily Traffic
ArcGIS	Aeronautical Reconnaissance Coverage Geographic information system
AWSC	All-Way Stop Control
CCTV	Closed-Circuit Television
CSIR	Council of Scientific and Industrial Research
DEM	Digital Elevation Model
DHM	Department of Hydrology and Meteorology
DOR	Department of Roads
EIA	Environment Impact Assessment
FO	Collision with fixed object
GIS	Geographic information system
GNSS	Geographic Navigation Satellite System
HCM	Highway Capacity Manual
HFL	High Flood Level
HO	Head-on collision
IEE	Initial Environment Examination
iRAP	International Road Assessment Program
JICA	Japan International Cooperation Agency
LOS	Level of Service
MoPPW	Ministry of Physical Planning and Works
MoWT	Ministry of Works and Transport

MUTCD	Manual on Uniform Traffic Control Devices
NeBS	Network Based Solution
NBS	Nepal Bridge Standard
NMV	Non-motorized Vehicle
NURS	Nepal Urban Road Standard
OA	Other angle (opposing direction)
OC	Out of control
PCU	Passenger Car Unit
PED	Pedestrian
QGIS	Quantum Geographic Information System
RA	Right angle collision
RE	Rear-end collision
ROW	Right of Way
SF	Saturation Flow
SIDRA	Signalized Intersection Design and Research Aid
SW	Sideswipe (common direction)
TWSC	Two-Way Stop Control
TWYC	Two-Way Yield Control

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1 INTRODUCTION

1.1 Background

There has been a prolonged debate regarding the alignment of national projects and highways in accordance with the needs of local communities and minimizing the impact on the environment. Despite extensive discussions, local requirements and environmental concern have only gradually been integrated into project designs through the implementation of Environmental Impact Assessment (EIA) and Initial Environment examination (IEE). However, beyond the amendments made following the outcry of the affected communities, there has been little consideration given to the specific needs and impacts of the locality surrounding the projects. Inadequate assessment has been conducted on how these projects would impact the surrounding communities and their livelihoods, particularly in regards to the connectivity of urban towns. Although the objectives of the new project designs are to address the local challenges and improve overall efficiency through performance enhancement, it has not been given due consideration (Sudmeier-Rieux et al., 2019). A prime example of this can be observed in the town of Banepa, situated in the eastern flanges of the Araniko highway near Kathmandu.

Araniko Highway(112.83kms), one of the important highways of Nepal, connects Kathmandu with Kodari and serves as the lifeline of Banepa. It shapes all major economic activities of Banepa city including mobility, trade and business. activities of Banepa are totally dependent on Araniko highway. Originally Banepa was largely situated in north- eastern flanks of the town but slowly shifted to sides of highway as it opened several opportunities. There is no denying on the importance of Araniko highway for the region and it can be concluded that Araniko highway is lifeline of people in the region.

Nepal government had been planning to upgrade the Araniko Highway to four lanes from Kathmandu to Tatopani, which is located near the border with China. This project was part of a larger effort to improve Nepal's transportation infrastructure and boost trade with China. A 9km section of the highway from Kathmandu to Suryabinayak was widened to four lanes in 2011 however, the section from Suryabinayak to Dhulikhel remains a two-lane highway until now. As per the recent plan of the government to expand the Araniko highway, the increasing demand of traffic in the highway is now to be addressed through an expansion project of Suryabinayak to Dhulikhel which will convert the highway to six-lane expressway.

The recurrent flooding problems of Banepa section of the highway is widely raised in national media as it connects two important highway with Kathmandu. Araniko highway acts as eastern gateway to Kathmandu. In addition to that Banepa is already facing a lot of problems of sewers, land use management, crashes and solid waste management at times. And at time of expansion, the municipality is conscious about addressing all those issues and to make design as beneficial to Banepa as follows. The prime concern of municipality lies in maximizing the benefits of expansion in favor of locals.

From preliminary overview of the design document, major issues of flooding incorporating size and appropriateness of side drains and appropriate sewerage system, issues of highway safety, accessibility of local routes to highway promoting local business activities and management of displaced parking facilities that could solve problems in long run remains unsolved. Urban Mobility and accessibility to the highway mobility was found highly paralyzed, any of standard safety aspects were not given due respect in design and problem of flooding remaining unaddressed and all of those design limitations urged municipality of Banepa seek assistance from Department of Transportation, Institute of Engineering to address all these issues. After initial site visit of faculties from Pulchowk campus, this project group was assigned this task under supervision of Asst Pr. Anil Marsani with aim to submit a review document, on all design aspects of original design document, to the municipality and the Department of roads before implementation of the project so that necessary amendment could be made in time within budget.

1.2 Study Area

Banepa, a municipal town, is a valley situated at about 25 kilometers east from Kathmandu. Banepa itself lies between different Municipalities namely: Bhaktapur, Mandandeupur, Panchal, Dhulikhel and Panauti Municipality. Although Banepa is a small town of an area of 5.56 square kilometers (2.15 sq mi), it has a population density of 4,454 per square kilometer (11,540/sq mi). Banepa is important for its strategic location as it lies in major trade route to Tibet, Araniko Highway, the only highway that connects Nepal and China (Tibet) and also BP highway that connects to terai region of Nepal which serves as gateway to Kathmandu from eastern region of Nepal. Though it is a small town, it has great prospects of development in years to come. Banepa is destined to be a major economic center due of its closeness with Kathmandu and also due to the investment made by the government to develop the region. This is to be done by connecting it with Bhaktapur through the construction of a six lane Highway from Suryabinayak to Dhulikhel. Apart from that the land and resources at Banepa is yet to be utilized and has seen significant changes with time and is set to bloom with the development of Kathmandu valley.
(https://en.wikipedia.org/wiki/Banepa, n.d.).

Some General Information of Banepa Municipality is given below as:

Table 1.2.1 Information of Banepa Municipality

District	**Kavrepalanchowk**
Province	**Bagmati**
Beneficiary Population	**67629**
Latitude	**(Starting Point)- 27°37'55.31"N** **(Ending Point)- 27°37'42.91"N**
Longitude	**(Starting Point)- 85°30'53.08"E** **(Ending Point)- 85°31'48.20"E**

This project report mainly focuses on a short section of the expansion project of Araniko highway that lies in Banepa, Kavre (Figure 1.2.1). This section starts from Punyamati river bridge in the west to Chandeshwori river bridge in the east). The main importance of this highway for the region is to ensure easier transport of goods and people between in the region by accommodating all required parameters in design document maximizing accessibility, connectivity and mobility.

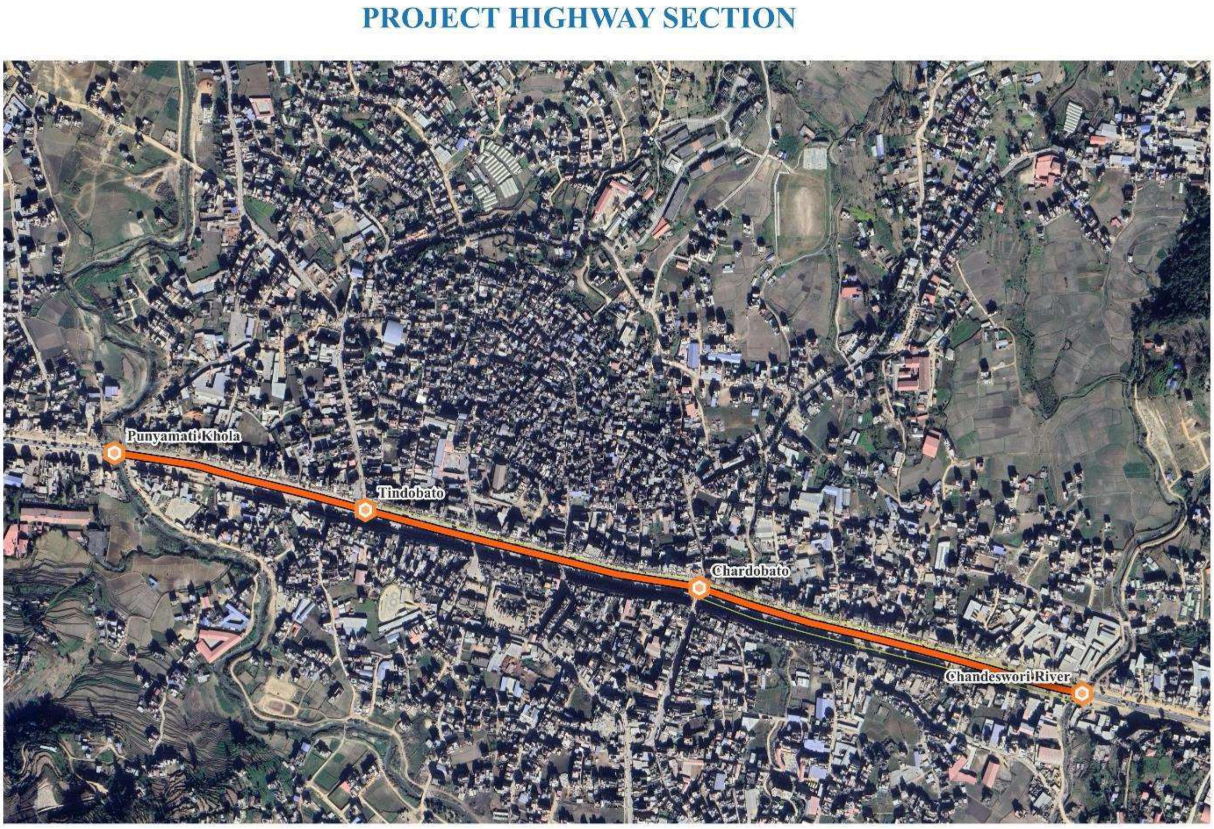

Figure 1.2.1 Site Location

1.3 PROBLEM STATEMENT

From initial discussion with the newly elected mayor of Banepa municipality, it was found that there is deep concern of the municipality on the ongoing expansion project of the Banepa section. This section of the highway from Punyamati to Chandeshwori has been facing multiple problems beforehand this expansion and municipality is concerned with addressing all these issues by considering all possible modifications and changes required to enhance the urban livable environment. Pulbazar area on bank of the Punyamati river has been prone to yearly recurrent flood in monsoon submerging the area which even obstructs the traffic flow for hours at times. This problem can be attributed to its elevation, part of the section lies at lower elevation compared to the High Flood Level (HFL) of the river which leads to backflow of water from drainage pipes. Also, large slopy paved road area with insufficient drainage facilities causes heavy runoff mixed with sewage causing submergence of the low-lying area within hours of runoff.

In the short site visit along the river corridor with the sub- engineer of the municipality, it was observed that the river width has been significantly reduced at a number of sections which include the supposed encroachment from the locals as well as natural silting in the meanders of the river. Also, it was observed that the free board of several bridges along the river were insufficient which further increased the siltation immediately upstream of the site and causing the river to overflow towards the land areas from these choke points. These play significant roles in the reduction of the river width and flow into the area.

In the meantime, the municipality has shown an urge to solve this problem beforehand so that highway facility becomes operatable in all season. They are also concerned about how the expansion of the highway will affect the nearby areas including the concern for the access of minor roads and overall facilitation of the intersections.

The initial design document of DOR shows that one of the important intersections such as Tindobato, has been treated as a minor intersection. This particular intersection has abundant commercial activity and traffic flow due to which it has a great demand of parking facility. The demand for it at present has been accommodated through On-street parking facility along the minor street as well as on the highway itself. The expansion project however, will displace all the parking supply and there is a need to accommodate the demand.

In addition to the parking concerns, the traffic demand in the section can lead to several safety issues which can be observed from the crash records of the section. This can be attributed to poor pedestrian facilities including traffic calming measures as well as provisions of proper lighting and signage. Furthermore, the intersections have been functioning through the provisions of two-way yield control at present, which warrants for a proper signal control for better operation of the intersection. The expansion project is likely to increase the speed of vehicles and also the cross width of the roads that the pedestrians require to cross together which results in legitimate concerns for their safety.

Figure 1.3.1 Flooding in the Pulbazar section of Araniko highway

Figure 1.3.2 Existing Condition of the project Highway Section

1.4 OBJECTIVES

The General Objective of the project is:

- To identify the problems faced by locals in project section of the highway including the concerns of urban flooding, traffic safety and parking to integrate their suggestions in the design document.

The specific objectives of the project are:

1. To analyze the existing design document for adequacy of the drainage facilities.
2. To evaluate the existing design document of the project section for safety concerns and suggest appropriate measures to address them including non-motorized travel.
3. To design signals of Chardobato and Tindobato Intersections of Banepa for smooth traffic flow
4. To study parking supply coverage in the project section.

1.5 SCOPE

Considering the problems encountered as highlighted in the previous section of this report, detailed study is conducted. to suggest possible measures for their mitigation which includes the analysis of impacts of the highway expansion project of the project section including the provisions of hydrology, safety, parking and overall traffic flow in the section.

Finally, this report will include a comprehensive analysis of existing vehicular and pedestrian traffic and will provide a signal design for the important intersections in the study section.

Based on the analysis of safety, parking, hydrology and traffic flow, this report will provide recommendations for improving short- and long-term recommendations to improve overall traffic operation in the study section.

The detailed deliverables of the project have been listed below:

Deliverables:

1. **Overview of Drainage facilities**: This report will provide detailed overview of the adequacy of the provided drainage facilities in the design document of the expansion project of the study section including cross section of the Punyamati river bridge and detailed flood estimation results.

2. **Crash Study Analysis**: This report will include the collection of Crash reports of the project section as well as its analysis for crash hotspots and provide possible measures of the same for its mitigation

3. **Overall safety analysis**: This project report will provide results of the safety analysis of the project section of the highway and will provide possible measures for its mitigation.

4. **Parking inventory Survey**: This report will also present the results of detailed survey of parking supply in the study area

5. **Traffic and Pedestrian Flow Analysis**: This report will include a comprehensive analysis of existing vehicular and pedestrian traffic and provide signal design for important intersections in the study section.

6. **AutoCAD plan and Sketchup image** of the intersection will also be included in this project.

Limitations:

Although, the project team has tried to consider as many parameters as possible into the study, there have been some limitations that could not be taken into account. Some of these limitations are:

1. **Scope Constraint**: The study is limited to the section of Araniko highway under consideration. It does not include the intersection design (alignment, pavement, etc.) of the section. Also, the assessment of sewer system of the region is out of scope of this study.
2. **Absence of adequate data**: Due to the lack of blueprint of drainage system of Banepa municipality, its study could not be incorporated into the project. Also, the lack of proper CCTV footage caused the project team to make use of limited primary video graphic survey in the study. Lack of cooperation and correspondence from concerned authorities was also felt during the course of the study which ultimately affected the results.
3. **Availability of proprietary software:** Absence of latest version of relevant software, especially SIDRA intersection and its training restricted the project to certain extents.

2 LITERATURE REVIEW

To initiate any project first of all the ideas and concepts related to the requirements and parameters to be studied during the feasibility or prefeasibility study which actually depicts the real-life problem statement and its most feasible and economic solution are to be considered. For that purpose, the project team members collected and reviewed the maps and previous documents/ reports.

General Design Principle (NURS)

"Urban roads should be planned and designed to:

- provide safe, short and fast thoroughfare and access to all road users, being motor vehicles, cyclists and pedestrians;
- convey clearly the primary function to road users and encourage appropriate driver behavior;
- deliver traffic volumes at speeds compatible with function;
- provide convenient location for services;
- provide an opportunity for landscaping;
- allow for parking, where appropriate;
- have due regard to topography, geology, climate, environment and heritage of the site;

The appropriate design criteria for an urban road largely depend on a set of economic indicators, namely costs of construction and operation on one side, and the financial benefits to the community on another. These are strategic parameters that influence a decision to build a road. Economic analysis, in conjunction with the traffic analysis, determines the functional class of the road and the design speed." (NURS, 2076)

2.1 Storm Water Drainage

A storm drainage system is a mechanism that prevents flooding and erosion. Roads without a rainwater drainage system can experience significant longitudinal runoff and erosion of the road surface. Such degraded surfaces can cause accidents and cost more than direct maintenance costs. In flooded areas, pedestrians and cyclists are forced to navigate uncomfortable and potentially dangerous terrain hidden beneath the water. Pedestrians and cyclists usually have the lowest priority on a cross-section of the road and have to walk through water and mud during the rainy season. An efficient drainage system on the street must meet the following factors:

- Catch pits should be spaced at regular intervals depending on size, precipitation, catchment area and deepest point of the road section.
- The lowest point of the cross section must be on the road. Bike lanes, sidewalk bus stops, and street vending areas should be elevated. Drainage areas should be flush with the surrounding pavement unless provided in a landscaped area.
- Greener approaches should be prioritized, such as landscaping depressions, improving groundwater recharge, reducing stormwater runoff, and improving overall road habitability. Swamps vary in size from tree holes and landscape strips to large, low-lying neighborhood parks. Wetlands work best in wide driveways with lots of unused space, but not in tight spaces that take up space from pedestrians, cyclists, and street vendors.
- Minimize the number of storm sewers to reduce construction and maintenance costs.
- The grating should be designed so that it will not get caught on the bicycle wheel.

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Ribbon Development

Ribbon development refers to a type of urban or suburban development where buildings, often commercial or residential, are constructed along major transport routes, such as highways or railways, in a long and narrow strip, creating a ribbon-like pattern. This type of development is characterized by a linear layout of buildings with a continuous frontage along the road, which may lead to an elongated and irregularly shaped urban form. Ribbon development can cause several issues, including traffic congestion, limited space for parking and pedestrian walkways, and reduced access to green spaces. Also, the cost of essential infrastructure such as water-supply and sewerage lines extravagantly increase due to such development.

Following is the list of the effects of Ribbon development:

- Reduced aesthetics of the area
- Impediment of sight distance
- Rise in normal (90 degree) intersections
- Increase in congestion on Highways
- Subsequent rise in crashes
- Loss in overall LOS and capacity
- Rise in roadside adverts rise
- Linear development increases the cost of overall infrastructure to be built such as Water supply, drainage, electricity, etc.
- Travel time and cost rise than otherwise orderly town planning
- Residents suffer from dust, noise, fumes

Some of the measures that can be applied to prevent ribbon development are as follows:

- Acquisition of Adequate highway width (~50m)
- Zoning by Urban planning bodies (Land improvement)
- Set-back distance, building lines, Control lines, height limits

- Access points in NH and expressways not less than 300m
    - Parallel service roads to close existing access points
    - Prevent mushroom growth also (too far access)
    - Safety ensured for petrol pump accesses
- Bypasses: if existing ROW less than minimum 30m
- Roadside adverts and Ribbon Development

Universal accessibility of roads

Streets should be designed such that they are safe and comfortable for pedestrian, cyclists, public transport users and motorists. Universal accessibility of roads refers to designing roads and streets that are accessible and usable by all people regardless of their age, ability or mode of transportation. It gives special consideration to people using wheelchairs or mobility devices, the elderly and the people with children. Some of the features of such roads are:

➢ Comprehensive footpath design for walkability:

- Tactile paving on footpath for persons with visual impairments.
- Table top for safe and comfortable pedestrian crossing.
- Colored (yellow color) guard rails to warn persons with visual impairments.
- Alignment of street furniture in multi-utility zone and exclusive pedestrian and NMV lane.
- Shaded trees to safeguard against adverse weather.
- Wayfinding and guide maps for visitors and tourists.
- Kerb ramp for footpath as shown in Figure 2.1.1
- Anti-skid flooring for ease of movement.
- Alignment of street furniture and right of way for pedestrians.

- Shade trees provide a comfortable environment for pedestrians.
- Bollard (Figure 2.1.2) spacing is more than 1m to allow wheelchairs, tri-cycles and prams to cross.
- Tactile paving on footpath for persons with visual impairments.
- Anti-skid flooring texture demarcated with visual contrasts.
- Audio traffic signals to help persons with visual impairments to cross independently.
- Well, illuminated pedestrian crossing with white bollard spacing and signage at two levels.

Figure 2.1.1 Kerb Ramp

Figure 2.1.2 Bollards

- Best practices in providing facilities for non-motorized vehicles (NMV) are as follows:
  - NMV lanes connectivity for cyclists, tri-cycle users, etc.
  - Table top (raised footpath) to maintain continuity.
  - Cobble stones on slopes for reducing speed of motorized vehicles.
  - Designated NMV lane with concrete floor and broom finish (friendly for non-motorized vehicle users).
  - Demarcated cycle lanes in color contrast benefit persons with low vision and old people.
  - Segregated NMV lanes, which increases safe and comfortable mobility.
  - Cycle stands are given close to habitat and integrated with public transport stops.
  - Cycle lanes (Figure 2.1.3) should be in good color contrast and symbols of cycle should be used as road markings which are easy to identify by all non-motorized vehicle drivers, cyclists and persons with cognitive and intellectual disabilities using mobility aids.

Figure 2.1.3 Cycle Lanes

- Best practices in providing access to public transport are as follows:
  - Clearly marked bus stops, low Kerb height and weather-proof design of bus shelter.
  - Cost effective universally accessible Bus Queue Shelter (Figure 2.1.4), a solution for smaller cities where there are no shelters or those which are being renovated.
  - Easy identification of waiting areas by persons with visual impairments by the use of tactile pavers.
  - Modified bus stops, precision docking and drivers training helps in reduced efforts in boarding and alighting in high steps buses (e.g.: The Figure 2.1.4 is of a bus stop in Raipur, India)

Figure 2.1.4 Bus Shelter

A. Flood Control

You can try to respond to flood disasters in two ways:
A technical approach to flood control and a regulatory approach to reduce vulnerability to flooding.

I. Engineering approach

o Channel Modification - Creating a new channel in a stream increases the cross-sectional area and creates a situation where a higher level is needed before flooding. Channeling also increases velocity and reduces drain time.

o Check dam (Fig. 2.1.5) –

A dam can be used to hold water and regulate the downstream desired rate. In this way, water levels can be lowered before heavy rains, allowing more water to collect in the reservoir and be drained later in a controlled manner.

Source: (Water Resources Structures Check Dams, n.d.)

Figure 2.1.5 Check Dam

o Retention ponds - Similar to dams, retention ponds are used to hold back water. To avoid flooding downstream, water can be stored in a retention pond and then released at a regulated discharge.

o Flood Channel – A flood channel is an area that can be constructed to provide an outlet to a stream and allow flood water to enter an area designated as a flood channel. A flood channel is an area where development is not permitted and land is used for agricultural or recreational purposes when there is no danger of flooding, but which provides drainage of floodwaters during heavy runoff. The Bonnet Carespil Way, west of New Orleans, is one such flood channel he. At low tide on the Mississippi River, the land between the river and Lake Pontchartrain is used for recreational purposes such as hunting, fishing, and dirt biking. At the peak of the river, New Orleans may experience flooding, and spillways have been opened to allow water to flow into Lake Pontchartrain. This lowers the water level of the Mississippi River, reducing the chance of levee breaches and overflows.

II. Regulatory Approaches

With a better understanding of the behavior of streams, the probability of flooding, and areas likely to be flooded during high discharge, humans can undertake measures to reduce vulnerability to flooding. Among the regulatory measures are:

- Floodplain zoning - Laws can be passed that restrict construction and habitation of floodplains. Instead, floodplains can be zoned for agricultural use, recreation, or other uses wherein lives and property are not endangered.
- Floodplain building codes - Structures that are allowed within the floodplain could be restricted those that can withstand the high velocity of flood waters and are high enough off the ground to reduce risk of contact with water.
- Floodplain buyout programs - In areas that have been recently flooded, it may be more cost effective for the government, which usually pays for flood damage either through subsidized flood insurance or direct disaster relief, to buy the rights to the land rather than pay the cost of reconstruction and then have to pay again the next time the river floods.
- Mortgage limitations - Lending institutions could refuse to give loans to buy or construct dwellings or businesses in flood prone areas. (TULANE, 2023).

B. Retention ponds.

Following conditions are required to support a retention pond:

- The drainage area required to support a retention pond can be as tiny as 0.03-0.1 km2 (Agency, 2012), or even smaller if the retention pond is connected to another 3-7% of the upstream catchment area normally required for the pond (CIRIA, 2007).
- Source of water, such as a spring.
- There are no particular restrictions on the maximum drainage area for retention ponds, though the pond's flow path length to width ratio should be between 3:1 and 5:1. Inlets and outlets should be set to maximize the length of the flow path through the pond. (Baron)
- The permanent pool’s depth should be between 1.2 and 2 meters
- To protect public safety and maintenance access, slope angles should not exceed 1:3.
- To allow for biological treatment of dissolved pollutants, the permanent pond residence time should be at least 20 days.
- Ponds are frequently positioned in a low location in the watershed where gravity can assist drain the water. A large site may necessitate several ponds divided into topographic sub-catchments (Baron, 2012).

C. Aust roads Hazard Safety:

- This guide focuses on designing the road and features of the road to reduce the potential for vehicles to lose control and run off the road to deal with errant vehicles so as to minimize the potential for serious injury or death. Figure 2.1.6 and Figure 2.1.7 shows such measures like guide posts and chevron markers to denote sharp turns
- Visual cues, appropriate signing and line marking, sealed shoulders and well-maintained roads (i.e., road surface, road shoulders, line markings and signage)

should all be provided so as to minimize the role of the road as a factor in causing a vehicle to leave the road.

Figure 2.1.6 Guide Posts

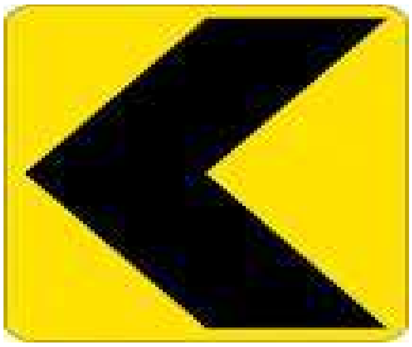


Figure 2.1.7 Chevron Marker

2.2 HYDROLOGICAL ANALYSIS

Watershed

A watershed is all the land and water area which contributes runoff to a common point. (Tideman, 1996) Thus, a watershed at a point on a defined drainage channel includes all the area which drains at that point. As shown in Figure 2.2.1, watersheds drain rainfall and snowmelt into streams and rivers. These smaller bodies of water flow into larger bodies of water such as lakes, bays, and oceans. Gravity helps direct the path of water through terrain.

Degraded watersheds lack protection measures for forests and agricultural land. Water flows at high speed, eroding soil and washing away crops. It pollutes streams and fills lakes with sediment. Flash floods occur frequently downstream. In contrast, well-managed watersheds seep most of the rainwater into the ground, increasing the groundwater supply and providing much-needed moisture and flood protection for plants, meadows, and trees.

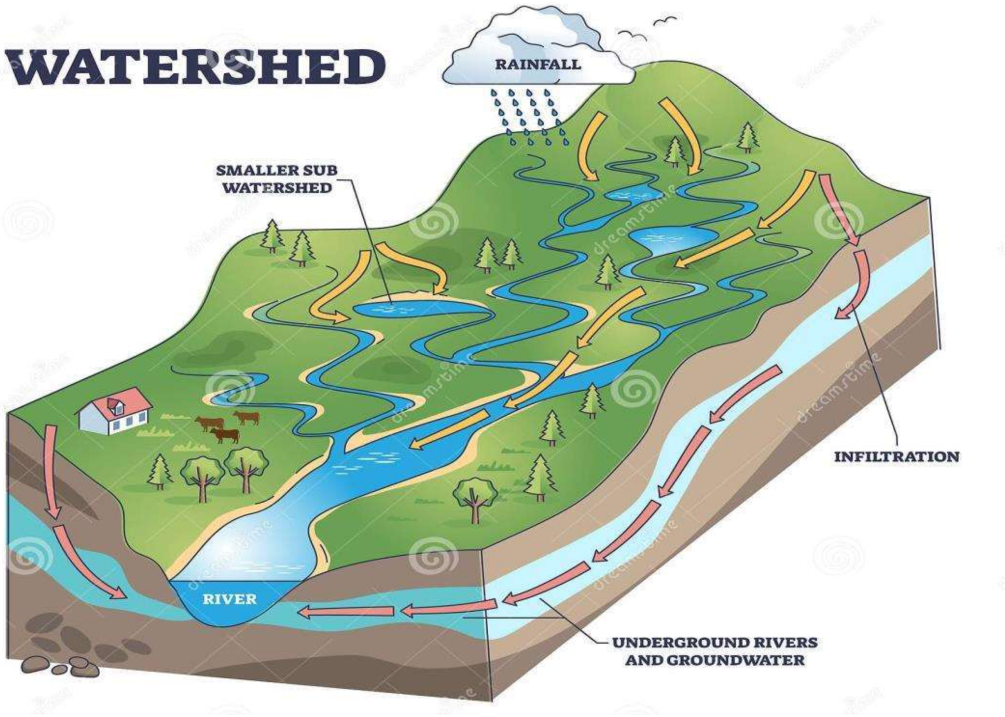


Figure 2.2.1 Watershed

Source: dreamstime.com

Watershed Management

“Watershed management, or protection, implies the wise use of soil and water resources within a geographical area to enable sustainable production and to minimize floods.” (Watermissionmp, n.d.) “Each watershed is unique in physiography, ecology, climate, water quality, land use, and human culture. Therefore, any generalized approach to watershed management must be customized to each setting when put into practice” (Coursehero, n.d.). Watershed management practices are those changes in land use, vegetative cover and other structural and non-structural actions that are taken in a watershed to achieve specific watershed management objectives.

. The major objectives of watershed management programs are:

· To increase infiltration into soil

· To control damaging excess runoff

· To manage and utilize runoff for useful purposes (hppcb, n.d.).

Design of Bridge

For bridges that span over bodies of water, it is required that the minimum freeboard clearance from the design high flood level (HFL), with allowance for afflux, to the lowest point of the bridge superstructure, should be no less than 1.0 meter. The minimum freeboard clearance specifications are outlined in Table 2.2.1

Table 2.2.1 Minimum freeboard for discharge

Discharge (m^3/sec)	Minimum Free board (mm)
Less than 200	1000
201-500	1200
501-2000	1500
2001-5000	2000
5000 and above	more than 2000

Source: (NBS, 2067)

Design of Side drain

Side drains are an integral part of roads and are an essential means of preventing structural damage to the road. However, it is evident from observations that their design and construction are often neglected, leading to various issues. Typically, the selection of standard drawings is used to determine their size, without any proper calculations, which results in the creation of drains that are unnecessarily large, expensive, and unsafe. However, it is evident from observations that their design and construction are often neglected, leading to various issues. Typically, the selection of standard drawings is used to determine their size, without any proper calculations, which results in the creation of drains that are unnecessarily large, expensive, and unsafe.

Designing drainage ditches is a complex task as they need to be large enough to accommodate large amounts of discharge, while also being shallow and gently sloping to prevent harm to pedestrians and vehicles. However, drains also need to be deep enough to effectively drain subgrade water. Therefore, when choosing the most suitable drain, compromises must be made.

To create a safe and appropriate section of side drains for different terrains, careful planning is necessary. The drainage design must consider cross drainage, surface drainage, erosion control, and subsurface drainage. The drains should be designed in such a way that any vehicle that falls into them can remain upright, suffer minimal damage, and be easily recovered.

“In flat terrain, earthen drains with a gentle side slope of 1:4 are effective. Dish type drains are more forgiving than channel sections. The depth of channel drains should ideally be no more than 300 mm (with an absolute maximum of 450 mm), and if deeper, they should be covered. Shallow "V" drains, dish type drains, tick drains, or covered drains can also be used as footpaths and provide extra width where roads are narrow.” (NBS, 2067)

“To ensure easy maintenance and self-cleansing velocities, drains should have a flat bottom width of at least 400 mm. Calculating the discharge is necessary to determine the most effective drain section and avoid unnecessarily large sections that could be hazardous to traffic.” (NBS, 2067)

The final disposal of the water should always be at stable water courses.

2.3 CRASH STUDIES

The problem of Crash is very acute in highway transportation due to complex flow patterns of vehicular traffic, presence of mixed traffic along with pedestrians. Traffic Crash leads to loss of life and property. Thus, the traffic engineers have to undertake a big responsibility of providing safe traffic movements to the road users and ensure their safety. Road Crashes cannot be totally prevented but by suitable traffic engineering and management the Crash rate can be reduced to a certain extent. For this reason, systematic study of traffic Crashes are required to be carried out.

Proper investigation of the cause of Crash will help to propose preventive measures in terms of design and control. In our study, we carried out crash studies of Banepa from Pulbazar to Chandeshwori section of Araniko highway with available data of the last 3 years with the aim of promoting better safety to road users from available trends and analysis.

Causes of road Crashes

Road crashes have a variety of causes (DrT), including:

1. Motorists on the road - Excessive speeding and rash driving, breaking traffic laws, failing to recognize traffic conditions, signs, or signals in a timely manner, negligence, indolence, alcoholism, sleep deprivation, etc.

2. Vehicle - Errors such brake, steering, tire-burst, and lighting system failure.

3. Slippery Road surface, potholes, and ruts.

4. Poorly designed curves, inadequate shoulders, inadequate sight distances, ineffective traffic signals, and poor illumination are all examples of poor road design.

5. Environmental variables - poor weather conditions that reduce vision and make driving dangerous, such as mist, snow, smoke, and heavy rain.

6. There are other factors as well, such as incorrect placement of billboards and an open level crossing gate.

Crash Analysis

The Crash study's data collection phase is when it all begins. The police are largely responsible for gathering information on the Crashes. Reports of motorist crashes are secondary data that are submitted by the drivers themselves. All of the following parameters should be included in the data being collected:

1. **General** –Date, time, person involved in Crash, classification of Crash like fatal, serious, minor

2. Location - Description and detail of location of Crash

3. Details of vehicle involved - Registration number, description of vehicle, loading detail, vehicular defects

4. Nature of Crash - Details of collision, damages, injury and casualty

5. Road and traffic condition - Details of road geometry, surface characteristics, type of traffic, traffic density etc.

6. Primary causes of Crash - Details of various possible cases (already mentioned) which are the main causes of Crash.

7. Crash cost - Financial losses incurred due to property damage, personal injury and Casualty.

The following goals require proper storage and retrieval of the collected data:

1. Locating the locations of the unusually high concentrations of crashes.

2. To determine the causes of Crashes, a detailed functional examination of the relevant Crashes location is performed.

3. Creation of a technique that makes it possible to spot risks before many crashes happen.

4. The creation of multiple statistical measurements of various crash-related aspects to provide information on broad patterns, shared causes, driver profiles, etc.

Crash investigation

The Crash data collection involves extensive investigation which involves the following procedure (DrT):

1. Reporting:

It involves basic data collection in form of two methods:

(a) **Motorist Crash report -** It is filed by the involved motorist involved in all Crashes fatal or injurious.

(b) **Police Crash report -** For all collisions where a police officer is present, a report is made by the officer in attendance. This typically includes fatal crashes, crashes with severe injuries necessitating emergency room or hospital treatment, or crashes with significant property damage.

2. At Scene-Investigation:

It entails gathering data on the spot, such as measuring skid marks, inspecting vehicle damage, taking a picture of the vehicles in their final location, and assessing the state and functionality of traffic control equipment and other road equipment.

3. Technical Preparation:

It entails gathering data on the spot, such as measuring skid marks, inspecting vehicle damage, taking a picture of the vehicles in their final location, and assessing the state and functionality of traffic control equipment and other road equipment.

4. Professional Reconstruction:

In this step, an effort is made to ascertain how the Crash occurred using the data that is currently accessible. This concerns the crash reconstruction that was thoroughly covered in Section No. 7. Determine behavioral or mediate causes of Crash is the word used in the professional world.

5. Cause Analysis:

The analysis of crash reconstruction studies and the use of available data to try to establish why the crash happened.

Crash data analysis

The purpose is to find the possible causes of Crash related to the driver, vehicle, and roadway.

Crash analyses are conducted to produce data like:

1. Driver and Pedestrian **-** Age-related crash occurrence and correlations to physical and psychological test findings.

2. Vehicle **-** The characteristics of the vehicle, the severity, the location, and the amount of vehicle-related damage.

3. highway conditions - Relationships between crash frequency, severity, and highway characteristics, as well as the relative importance of changes affecting roadways

Calculating the crash rate—which measures crash involvement by kind of highway—is crucial. These rates offer a tool to compare the relative safety of various highway, street, and traffic management systems. Another is the participation of certain drivers and cars in collisions

2.4 HIGHWAY SAFETY AND USE OF iRAP FOR STAR RATINGS

iRAP:

iRAP is a UK-registered charity established to help tackle the devastating social and economic cost of road crashes. (iRAP, 2022) iRAP is a global road safety charity that was established in 2006 with the aim of reducing the number of road traffic fatalities and serious injuries around the world (itf20, 2020). The program is based on the premise that road crashes are preventable and that proactive interventions can make a significant difference in reducing the severity and number of crashes.

iRAP's approach to improving road safety is based on assessing the safety performance of roads using its star rating methodology, which assesses the safety of roads based on a range of factors such as road geometry, traffic volume, and speed management. By using this methodology, iRAP can provide a standardized approach to assessing and improving road safety that is applicable across a range of countries and contexts.

In addition to its star rating methodology, iRAP also offers a range of tools and resources to support road safety improvement initiatives, including training programs for road safety professionals, advocacy materials for government officials, and data collection tools to support road safety research.

iRAP coding

The core of an iRAP project is road attribute coding, which involves using geo-referenced images with location data gathered from road surveys or designs to document specific characteristics of each 100-meter stretch of road. This information is then integrated with other relevant data and uploaded into ViDA, where it is used to generate Star Ratings and Safer Roads Investment Plans. The purpose of road attribute coding is to record the road attributes that are visible in a geo-referenced image or road design. The ultimate goal of this process is to encourage the implementation of measures that can enhance road safety and save lives.

Further information on iRAP coding is provided in iRAP Coding Manual Drive on the left edition.

A sample of georeferenced image (Figure 2.4.1) and a coding file is as shown below.

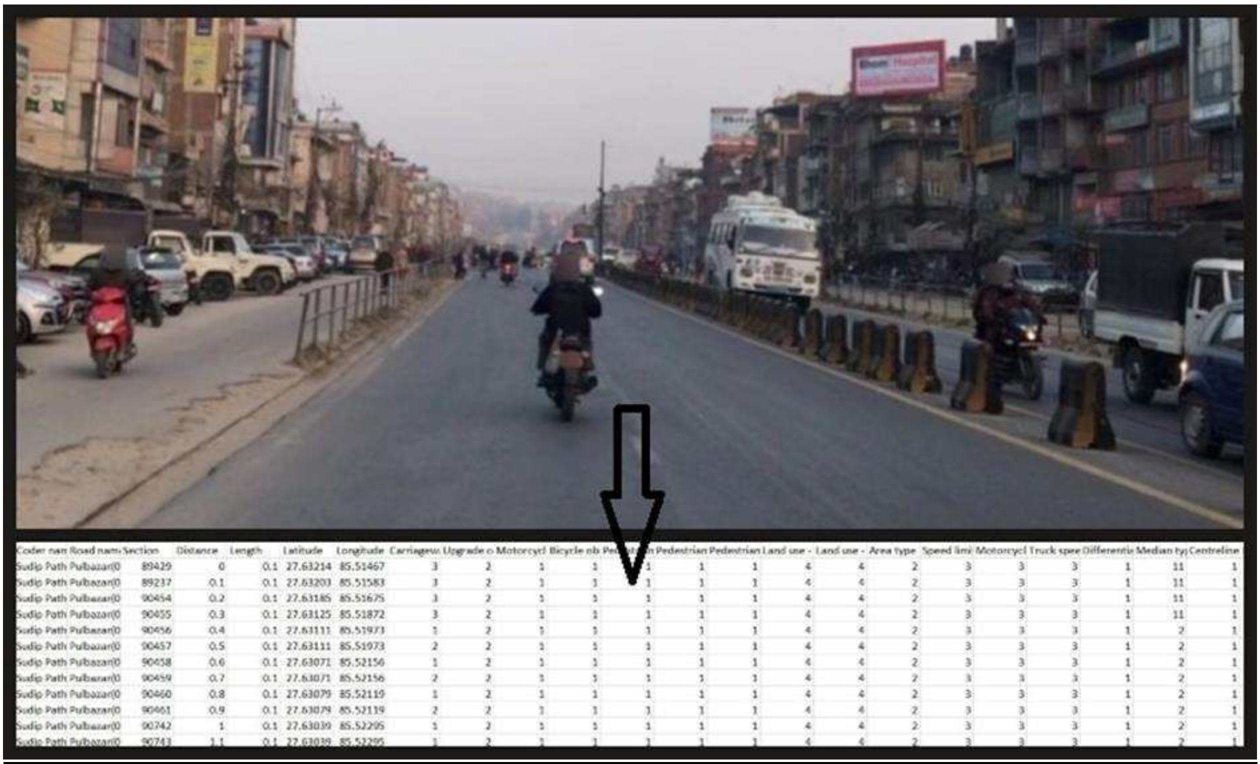


Figure 2.4.1 iRAP Coding

Safety measures

The final objective is to create specific improvement strategies to lessen the conditions that cause Crashes. Engineering, enforcement, and education are the three general categories into which the strategies to lower the crash rates are separated. The following is a list of engineering-related safety measures (DrT):

1. In order to achieve contrast in visibility, the road should have features that are different from their surroundings in terms of colors and patterns, such as edge markers, grass-covered shoulders, and shoulder strips.

2. Planting greenery along the sides of roadways is a good solution.

3. Another very helpful feature for visual guiding is the ability to see the tree crown from a distance.

4. The presence of guard rails in a variety of striking colors diverts attention from the road and prevents monotonous driving.

The driver is drawn to the planting of trees along the side of the road with the turning angle, alerting them to the impending bend. Another illustration is shown in the figure below. When a road has a risky at-grade intersection, trees are positioned so that it appears as though there is a dense forest ahead. As a result, drivers naturally prefer to slow down or halt their vehicles at that location to avoid accidents. Drivers frequently extrapolate the future course of the road. Therefore, it is the duty of the traffic engineer to ensure that the driver feels psychologically at ease when operating a motor vehicle in order to lower the likelihood of error and avoid mental fatigue.

Road reconstruction

As more cars are added to the road each year, traffic flow becomes more difficult to manage, roadways' functionality and transportation capabilities are drastically reduced, and the crash rate rises. As a result, new roads need to be built. It is important to correctly mark the crash sites so that the rebuilding may be organized accordingly. The process of reconstruction may also involve building a new road next to an existing one, replacing the pavement without altering the road's horizontal alignment or profile, or rebuilding a specific stretch of it (DrT).

Channelization

Different traffic streams are divided and given separate lanes to correspond to their practical itineraries by channelizing traffic at the intersection. Confusion is reduced by dividing the points of contention among crossings (DrT).

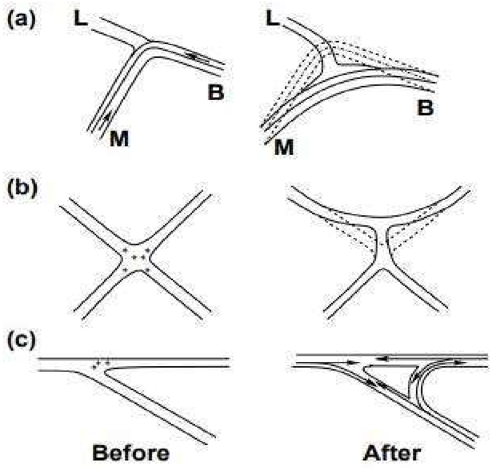


Figure 2.4.2 Road Reconstruction Technique

. The number of decisions required to be made by the driver at any time is reduced allowing the driver time to make the next decision.

The principles of proper channelized intersection are (DrT)**: -**

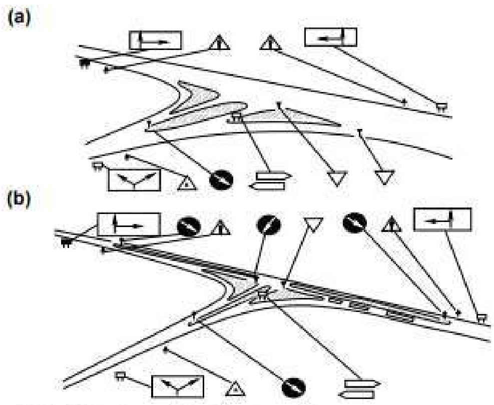
1. The driver should be able to clearly see, understand, and follow the intersection's structure.

2. Should guarantee superiority to vehicles using higher-class roads.

3. Due to the way the intersection is designed, a driver must make a decision between no more than two alternative routes of travel at any given time. Islands, marks, and visual assistance are used to do this.4. The offered island should divide through and turning traffic flows that are traveling at high speeds.

5. The width of the traffic lane should ensure that large vehicles can turn without being hindered. The straight part should be 3.5 meters wide without a kerb, and the width of the traffic lane next to the island should be 4.5 to 5 meters at entry and 6 meters at exit.

6. Pedestrian crossing should be provided.



a) Partially channelized intersection
b) Fully channelized intersection

Figure 2.4.3 Channelized Intersection Ensuring Safety

Road signs

Road signs are an essential component of safety since they ensure both the driver's personal safety (warning signs) and the safety of other drivers, pedestrians, and other road users (regulatory signs). In order to have adequate time to interpret and react, the driver should be able to read the sign from a distance. They must be installed and have the proper size, color, shape, and placement. Additionally, they must be maintained because their installation would be ineffective without maintenance to keep them in good shape. According to British investigation height of text in road sign should be

$H = (N + 6)\ v/\ 64 + 3/\ 4\ L$

Where, N = No. of words on the sign,

v = speed of vehicle (kmph)

L = distance from which inscription should be discernible (m)

Other methods

Various other methods of traffic Crash mitigation are described below (DrT):

1. Street lighting

Due to low visibility at night, street lighting of a sufficient standard aids in ensuring public safety. With the installation of good lighting, crashes overall are reduced by 21%, "all casualty" crashes are reduced by 29%, "non-pedestrian casualty" crashes are reduced by 21%, and "pedestrian casualty" crashes are reduced by 57%.

2. Improvement in skid resistance

Vehicles may skid on relatively smooth roads, and wet pavement increases the risk of wet weather crashes, which account for around 20–30% of all crashes. Therefore, it's crucial to increase the road's skid resistance. Construction of high-friction overlay or paving with grooves are two methods for enhancing the road's skid resistance.

3. Road markings

On a highway, road markers guarantee proper traffic control and direction. They perform an additional role for traffic signs. They act as a psychological barrier and define the traffic path, ensuring that it is kept clear of obstacles and other vehicles for the safe flow of traffic. Their goal is to provide a steady and secure flow of traffic.

4. Guide posts with or without reflector

They are situated along the edge of the road to stop the cars from deviating off the path. Their availability is crucial on mountainous roads to stop the car from rolling off the top. During the night, reflective guideposts direct the movement of moving vehicles.

5. Guard rail

Guide posts and guard rails both serve similar purposes. High embankments, hilly roads, roads parallel to riverbanks or lakeshores, near rock protrusions, trees, bridges, and abutments are all significant hazards for car collisions. It is necessary to keep the car on the road if the driver's error or improper behavior caused it to veer off the path. This solution can stop the serious problem that uncontrollable drivers pose.

6. Driver reviver stop

In countries like the U.S.A., driver refresher stops are frequently used so that drivers can stop and refuel with food, entertainment, and rest. They greatly contribute to road safety by relieving the driver's stress from frequent driving. Every two hours of travel must include one of these stops.

7. Constructing flyovers and bypass

To reduce the crash rate in places with heavy local traffic, bypasses must be built to separate through traffic from local traffic. Flyovers are necessary for enhanced safety and a decrease in crash rates in order to reduce conflicts at important intersections.

8. Regular Crash studies

Preventative measures are implemented based on the past records of crashes, and then the data linked to crashes is again collected to assess the effectiveness of the measures and for the deployment of additional preventative measures in the future.

The following is a description of safety measures connected to enforcement:

The following is a list of the several types of enforcement that may be helpful to stop crashes at locations where crashes are likely to happen. These regulations are periodically updated to make them more thorough.

Speed control

All vehicles should have their spot speeds checked at various times and locations, and anyone caught exceeding the speed limit may face legal consequences.

Training and supervision

When granting permits to taxi and public transportation drivers, the transportation authorities should exercise thorough oversight. After a certain amount of time, a driver's license may be renewed, but only after undergoing specific tests to determine whether the driver is fit.

Medical check

At regular intervals, the drivers should have their vision and reaction times evaluated.

Safety measures related to education

The various measures of education that may be useful to prevent Crashes are enumerated below.

Education of road users

By establishing the essential training in the schools for the children and with the aid of posters displaying the serious consequences as a result of reckless road users, passengers and pedestrians should be taught the rules of the road, the proper manner of crossing, etc.

Safety drive

Enforcing traffic safety week when the public is being educated and the traffic police are guiding traffic appropriately. Various regions of the nation should host driver training seminars and workshops.

Safety audit

It is the process of evaluating the road safety measures in use. It has the benefits of minimizing future crashes through effective planning and decision-making, lowering long-term planning costs, and making it easy for all types of users to understand how to use it securely. According to Wrisberg and Nilsson (1996), a safety audit is conducted in five steps.

Four fundamental (non-negotiable) principles are as follows:

• People make mistakes that lead to road crashes

• The human body has a limited physical ability to tolerate forces before harm occurs

• There is a shared responsibility amongst those who design/build/manage/use roads and vehicles and provide post-crash care

• All parts of the system should be strengthened in combination; if one part fails, road users are still protected

2.5 EXISTING TRAFFIC FLOW OVERVIEW

Introduction

Intersections are places where two or more roads meet and allow drivers to change directions. They range in complexity from simple four-way crossings to more intricate junctions where several roads converge. At intersections, drivers must choose which route to take, unlike on straight sections of the road. This decision-making process increases the likelihood of crashes, making intersections high-risk areas. Intersections are crucial to the flow of traffic on any road, as their efficiency affects overall traffic flow. Intersections fall into three broad categories: grade separated without ramps, grade-separated with ramps (known as interchanges), and at-grade intersections.

At-grade intersections basically means that the intersecting roads meet at the same level and therefore there exist conflicts between intersecting streams of traffic. The elementary types of at-grade intersections are T i.e., three-leg intersections that consists of three approaches; cross intersections, which consist of four approaches; and multi leg intersections, which consist of five or more approaches.

Chardobato Intersection is a typical four-legged intersection while Tindobato is a typical T intersection.

Delay normally occurs due to sharing of space and time between conflicting streams / movements of vehicles, making intersections the critical points of any road network. The type of control at the intersections includes uncontrolled, stop controlled, roundabout and signalized.

"The level of service at any intersection on a highway has a significant effect on the overall operating performance of that highway. Thus, improvement of the level of service at each intersection usually results in an improvement of the overall operating performance of the highway. An analysis procedure that provides for the determination of capacity or level of service at intersections is therefore an important tool for designers, operation personnel, and policy makers. Factors that affect the level of service at intersections include the flow and distribution of traffic, the geometric characteristics, and the signal system." (Hoel, 2018)

Several methods of controlling conflicting streams of vehicles at intersections are in use. The choice of one of these methods depends on the type of intersection and the volume of traffic in each of the conflicting streams. Guidelines for determining whether a particular control type is suitable for a given intersection have been developed and are given in the (MUTCD, 2009). These guidelines are presented in the form of warrants, which have to be compared with the traffic and geometric characteristics at the intersection being considered.

In other words, At-grade intersection improvement can be done in several methods. Spatially, the conflicting flows can be directed using **Channelization** or can be temporally given access using **Signs and Signalization**.

"AASHTO defines channelization as the separation of conflicting traffic movements into definite paths of travel by traffic islands or pavement markings to facilitate the safe and orderly movements of both vehicles and pedestrians. A traffic island is a defined area between traffic lanes that is used to regulate the movement of vehicles or to serve as a pedestrian refuge" (Aksan, 1998). Vehicular traffic is excluded from the island area. A properly channelized intersection will result in increased capacity, enhanced safety, and increased driver confidence. On the other hand, an intersection that is not properly channelized may have the opposite effect. It is important to avoid excessive channelization at intersections because it can cause confusion for drivers and reduce the intersection's efficiency. Islands used for channelization should be carefully designed and positioned so that they do not pose a danger to vehicles, but also effectively prevent drivers from driving over them. Sign Controlled intersections use Yield or Stop Signs in a Yield Control or Stop Control intersection respectively. If the signs are kept at minor approaches only, these are referred to as Two-Way Stop Controlled Intersections (TWSC) or Two-Way Yield Controlled Intersections (TWYC). Else, these are known as All-Way Stop Controlled Intersections (AWSC). Chardobato and Tindobato intersections both have TWYC sign control although not formally but in effect.

All drivers on approaches with **yield signs** are required to slow down and yield the right-of-way to all conflicting vehicles at the intersection. Stopping at yield signs is not mandatory, but drivers are required to stop when necessary to avoid interfering with a traffic stream that has the right-of-way. Yield signs are therefore usually placed on minor-road approaches where it is necessary to yield the right-of-way to the major-road traffic. Yield signs shall also be placed at roundabout approaches to control traffic on the approaches, but shall not be used on the circulatory roadway.

Terminologies

Capacity: "At an unsignalized intersection, the capacity of each non-priority movement or flow is determined by calculating the maximum number of vehicles that can perform that movement based on the prevailing traffic conditions on all other approaches. This capacity is typically expressed in Passenger Car Units per hour (PCU/h). Expressed as Cx, the capacity of movement 'x'." (Indo-HCM, 2017)

Critical Gap: "The term "critical gap" refers to the smallest gap that a driver performing a non-priority maneuver considers safe to proceed, and this varies from one driver to another. The critical gap always falls between a driver's maximum rejected gap (which is too short to cross) and their accepted gap (which is long enough to cross safely). Expressed as, tc,x: Critical Gap (s) for Vehicle Type 'x'". (Indo-HCM, 2017)

Follow-up Gap: "The follow-up time is the time headway between successive minor street vehicles while accepting the same gap size in the priority stream, when there is a continuous queuing on the minor approach. Expressed as (tf,x): Follow-up time for movement 'x' (s)" (Indo-HCM, 2017)

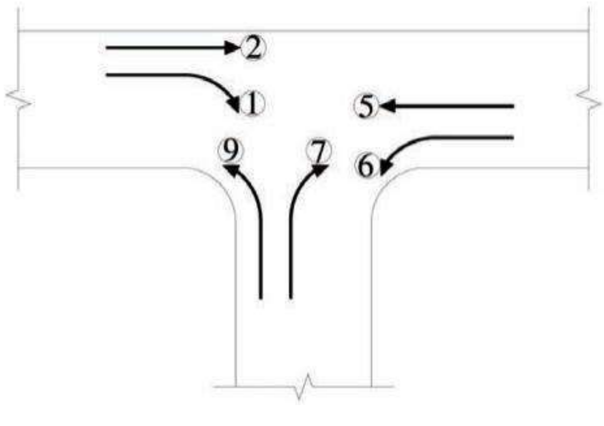
Major and minor Streets: "A road that has greater priority and volume is referred to as a major street, while a road with lower priority and volume is referred to as a minor street. It is up to the analyst to determine which road has priority, but it is recommended that the hierarchy of the road network and volume be taken into account when making this determination." (Indo-HCM, 2017)

Conflicting Flow Rate corresponding to movement x (PCU/h) is denoted by Vc,x. (Indo-HCM, 2017)

Unsignalized Intersection:

"An unsignalized intersection is where two roads intersect without any traffic signals or manual controls, with or without a central island. One of the roads is taken as the major road, and the other as the minor road, depending on their relative importance. When STOP signs are used to control traffic on the minor road, it is called a Two-Way Stop Controlled (TWSC) intersection. If STOP signs are placed on all approaches, it is referred to as an All Way Stop Controlled (AWSC) intersection. If YIELD signs are placed on minor roads, it is referred to as a Two-Way Yield Control (TWYC) intersection. In Nepal, due to weak enforcement of traffic regulations and a lack of understanding of priority rules among road users, the difference between TWYC, TWSC and AWSC intersections is not evident. Unsignalized intersections can have three legs, four legs, or multiple legs, and the wider or more heavily trafficked road is considered the major road." (Indo-HCM, 2017)

Different Vehicular movements and priority ranks are described below:



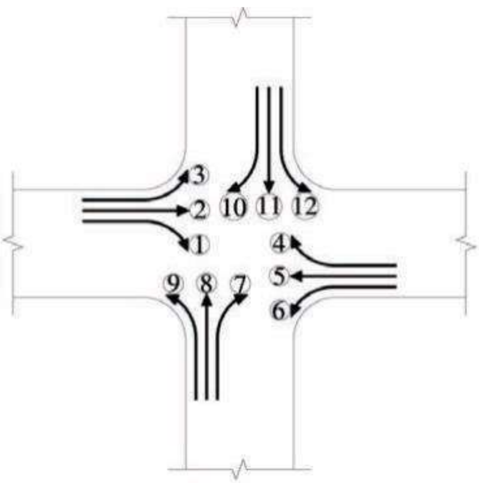


Figure 2.5.1 Vehicular Movements at a Typical Four-Legged intersection
Figure 2.5.2Vehicular Movements at a Typical Three-Legged intersection

Source: Indo-HCM (2017

Priority Rank	Movement
1	Movement 2
	Movement 3
	Movement 5
	Movement 6
2	Movement 1
	Movement 4
3	Movement 7
	Movement 10
4	Movement 8
	Movement 11

Figure 2.5.3 Priority Ranks for Different Movements (Indo-HCM, 2017)

Note: Minor Left turns do not pose significant importance in the context of Nepali roads.

Conflicting Movement/Flow: “Any movement of higher priority with which the subject movement shares the right of way is to be included as conflicting flow for that movement.” (Indo-HCM, 2017)

For example, rank 2 movements are relatively easier to execute, as they have to cross only one stream of traffic. The conflicting traffic for right turn movements is presented in the figure below:

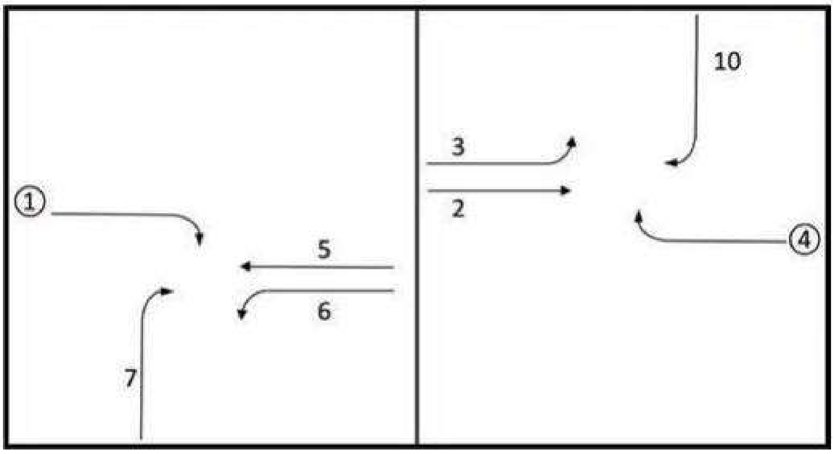


Figure 2.5.4 Conflicting Movement for Different Movements

The representation is suitable for the three-legged Tindobato intersection and four-legged Chardobato intersection respectively.

The Google Earth Image of both the intersection is as below:

Figure 2.5.5 Chardobato

Figure 2.5.6 Tindobato

As shown in the figure above, the intersection consists of no signals and proper signage to be classified into a type of intersection. However, inspection of the field shows that the flow pattern behaves as a TWYC intersection where the major traffic flows from E-W legs and minor Traffic from N-S legs.

Peak Hour Volume Calculation

Traffic volume is defined as the number of vehicles passing a point on a highway, or a given lane or direction of a highway, during a specified time interval. Daily Volume should not be solely used for design or analysis as volume varies considerably over a 24-hour period. Peak hour flow is the maximum flow hour of the day. Variation of volume within the peak hour is important as the traffic flow quality is dependent on the system's ability to meet short-term fluctuations. 15 minutes is the most commonly used sub-hourly study period since it is believed to be the smallest time window over which traffic conditions are statistically stable. For a 15-minute analysis period, peak hour factor is defined as follows:

$$PHF = \frac{V}{4 * V_{m15}} \qquad (5\text{-}2)$$

where: V = hourly volume, vehs

V_{m15} = maximum 15-minute volume within the hour, vehs

PHF = peak-hour factor

Saturation Flow Rate

When the traffic signal turns GREEN, there is a queue of stored vehicles that were stopped during the preceding RED phase, waiting to be discharged. If the headways of successive vehicles are measured and plotted, we obtain the following result:

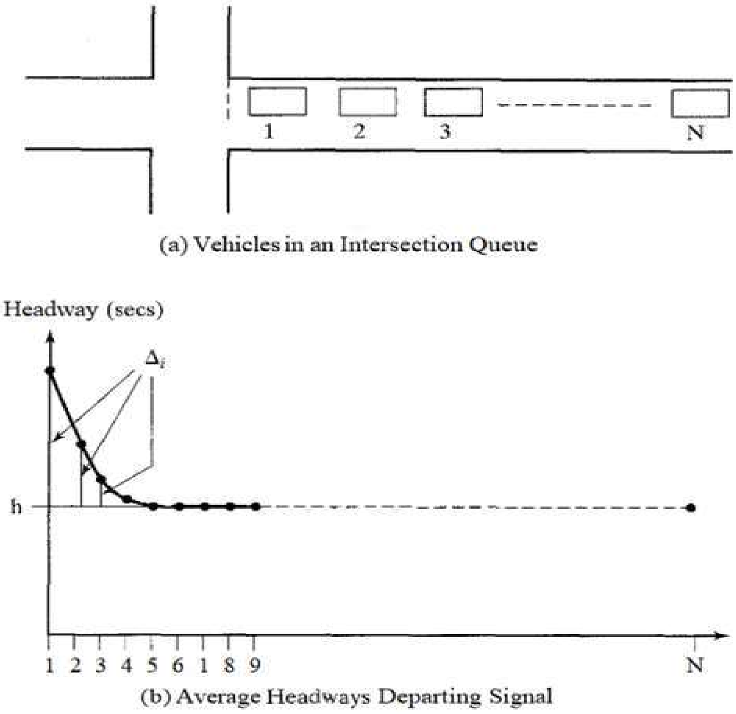


Figure: Average Headways at Signal

“As seen in above figure, average headways tend toward a constant value. The constant headway achieved is referred to as the saturation headway, as it is the average headway that can be achieved by a saturated, stable moving queue of vehicles passing through the signal. It is convenient to model behavior at a signalized intersection by assuming that every vehicle (in a given lane) consumes an average of “h” seconds of green time to enter the intersection. If every vehicle consumes “h” seconds of green time and if the signal were always green, then “s” vehicles per hour could enter the intersection. This is referred to as the saturation flow rate: “(Indo-HCM, 2017)

$$s=3600/h$$

Capacity, v/c ratio and Critical Lane groups

Capacity at signalized intersections is based on the concept of saturation flow and saturation flow rate. The flow ratio for a given lane group is defined as the ratio of the actual or projected demand flow rate for the lane group and the saturation flow rate. The capacity of a given lane group may be stated as:

$$c_i = s_i \frac{g_i}{C}$$

where

c_i = capacity of lane group i (veh/h),

s_i = saturation flow rate for lane group i (veh/h), and

g_i/C = effective green ratio for lane group i.

“The ratio of flow rate to capacity (v/c), often called the volume to capacity ratio, is given the symbol X in intersection analysis. It is typically referred to as the degree of saturation. It is a measure of the sufficiency of available capacity to handle existing or projected demands. Obviously, cases in which X > 1.00 indicate a shortage of capacity to handle the demand. For a given lane group i, Xi is computed using:” (HCM, 2000)

$$X_i = \left(\frac{v}{c}\right)_i = \frac{v_i}{s_i\left(\frac{g_i}{C}\right)} = \frac{v_i C}{s_i g_i}$$

where

X_i = $(v/c)_i$ = ratio for lane group i,
v_i = actual or projected demand flow rate for lane group i (veh/h),
s_i = saturation flow rate for lane group i (veh/h),
g_i = effective green time for lane group i (s), and
C = cycle length (s).

"For a particular phase, the lane group with the maximum demand (v/s ratio) is the critical lane group. For example, with a two-phase signal, opposing lane groups move during the same green time. Generally, one of these two lane groups will require more green time than the other (i.e., it will have a higher flow ratio). This would be the critical lane group for that signal phase. Each signal phase will have a critical lane group that determines the green-time requirements for the phase. The critical v/c ratio of an intersection is determined as follows:" (HCM, 2000)

$$X_c = \Sigma\left(\frac{v}{s}\right)_{ci}\left(\frac{C}{C-L}\right) \qquad (16\text{-}8)$$

where

X_c = critical v/c ratio for intersection;
$\Sigma\left(\frac{v}{s}\right)_{ci}$ = summation of flow ratios for all critical lane groups i;
C = cycle length (s); and
L = total lost time per cycle, computed as lost time, t_L, for critical path of movements (s).

Four laning of Highways

HCM defines "Level of service in terms of total control delay per vehicle in a lane group. "Total control delay" is essentially time in queue delay plus acceleration-deceleration delay. The average control delay per vehicle for a given lane group is given by:" (HCM, 2000)

$$d = d_1(PF) + d_2 + d_3 \quad (16\text{-}9)$$

where

d = control delay per vehicle (s/veh);

d_1 = uniform control delay assuming uniform arrivals (s/veh);

PF = uniform delay progression adjustment factor, which accounts for effects of signal progression;

d_2 = incremental delay to account for effect of random arrivals and oversaturation queues, adjusted for duration of analysis period and type of signal control; this delay component assumes that there is no initial queue for lane group at start of analysis period (s/veh); and

d_3 = initial queue delay, which accounts for delay to all vehicles in analysis period due to initial queue at start of analysis period (s/veh) (detailed in Appendix F of this chapter).

Uniform delay assumes uniform arrivals, stable flow, and no initial queue and is calculated by:

$$d_1 = \frac{0.5C\left(1-\frac{g}{C}\right)^2}{1-\left[\min(1,X)\frac{g}{C}\right]} \quad (16\text{-}11)$$

where

d_1 = uniform control delay assuming uniform arrivals (s/veh);

C = cycle length (s); cycle length used in pretimed signal control, or average cycle length for actuated control (see Appendix B for signal timing estimation of actuated control parameters);

g = effective green time for lane group (s); green time used in pretimed signal control, or average lane group effective green time for actuated control (see Appendix B for signal timing estimation of actuated control parameters); and

X = v/c ratio or degree of saturation for lane group.

“Incremental delay estimates the incremental delay due to nonuniform arrivals and temporary cycle failures (random delay) as well as delay caused by sustained period of oversaturation (oversaturation delay).” (HCM, 2000)

$$d_2 = 900T\left[(X-1)+\sqrt{(X-1)^2+\frac{8kIX}{cT}}\right] \tag{16-12}$$

where

d_2 = incremental delay to account for effect of random and oversaturation queues, adjusted for duration of analysis period and type of signal control (s/veh); this delay component assumes that there is no initial queue for lane group at start of analysis period;
T = duration of analysis period (h);
k = incremental delay factor that is dependent on controller settings;
I = upstream filtering/metering adjustment factor;
c = lane group capacity (veh/h); and
X = lane group v/c ratio or degree of saturation.

“When a residual queue from a previous time period causes an initial queue to occur at the start of the analysis period (T), additional delay is experienced by vehicles arriving in the period since the initial queue must first clear the intersection. This is known as the initial queue delay.” (HCM, 2000)

“The procedure for delay estimation yields the control delay per vehicle for each lane group. It is often desirable to aggregate these values to provide delay for an intersection approach and for the intersection as a whole. This aggregation is done by computing weighted averages, where the lane group delays are weighted by the adjusted flows in the lane groups. Aggregate delay for an approach is calculated by:” (HCM, 2000)

$$d_A = \frac{\Sigma d_i v_i}{\Sigma v_i}$$

where

d_A = delay for Approach A (s/veh),

d_i = delay for lane group i (on Approach A) (s/veh), and

v_i = adjusted flow for lane group i (veh/h).

Individual approach delays can be further aggregated to arrive at the delay for the whole intersection.

$$d_I = \frac{\Sigma d_A v_A}{\Sigma v_A}$$

where

d_I = delay per vehicle for intersection (s/veh),

d_A = delay for Approach A (s/veh), and

v_A = adjusted flow for Approach A (veh/h).

Level of service of the intersection is assumed to be directly related to the control delay of the intersection as shown below: (HCM 2000)

EXHIBIT 16-2. LOS CRITERIA FOR SIGNALIZED INTERSECTIONS

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

2.6 Proposed DESIGN ANALYSIS AND OVERVIEW

Signalized Intersection Terminologies (HCM, 2000) **:**

- **Analysis Period (T):** The time-period (in hours) during which the capacity analysis is performed (0.25hrs in this study)
- **Approach:** Roadway leading to the stop line of the intersection that accommodates one or combination of right-turn, through and left-turn movement of vehicles
- **Approach Volume (V):** The number of vehicles arriving on an intersection approach upstream of the queue influence per unit time; expressed in PCU/h.
- **Approach Flow Rate (Vp**): Approach volume during peak hour divided by the peak hour factor (PHF); expressed in PCU/h.
- **Change Interval**: The amber (yellow) plus all-red interval that occurs between phases of a traffic signal to provide for clearance of the intersection before conflicting movements are released.
- **Control delay (d):** This is the average delay experienced by a vehicle due to the presence of signal control. This includes stopped delay and the time lost due to queue move-up, deceleration to stop and acceleration back to the desired speed. Control delay is expressed in sec/vehicle or sec/PCU.
- **Cycle (also signal cycle):** One complete sequence of signal indications covering all phases.
- **Cycle time (C):** Duration of a cycle in seconds.
- **Degree of Saturation (X):** A measure of how much demand (volume) an approach is experiencing compared to its capacity. It is the ratio of approach volume to approach capacity.
- **Delay**: Additional travel time (in sec) experienced by a vehicle to traverse through an intersection.
- **Discharge flow rate**: The rate at which the vehicles in a movement group cross the stop line during the green interval. This is measured in PCU/h.
- **Downstream**: direction of flow of the traffic
- **Green Time (G):** Duration of time (in seconds) for which the signal indication is green for a traffic movement.

- **Effective Green Time (g):** Green time minus the time lost at the beginning of the green phase when vehicles are still accelerating, plus the time gained by vehicles making use of the amber period.
- **Exclusive lane**: An approach lane dedicated to a particular departure movement (typically left-turn movement, through movement or right-turn movement).
- **Fixed-time signal operation**: A control mode of a signalized intersection in which the sequence and duration of all signal indications (timing program) remains unchanged (from one cycle to another).
- **Flow Ratio (v/SF):** Ratio of actual flow rate to saturation flow rate of a movement group.
- **Interval**: Duration of time during which all traffic signal indications remain constant
- **Level of Service (LOS**): A qualitative measure used to describe the operational condition of a traffic facility.
- **Lost Time (L):** The time, in seconds, during which an intersection is not used effectively by any movement; it is the sum of clearance lost time and start-up lost time.
- **Movement Group**: Any one or combination of through or right turning or left turning movements at an intersection approach that are allowed in a shared operation in the same phase. Movement group is treated as a separate entity by assigning appropriate effective width of the approach for capacity and LOS analysis. The movement group is of importance when input data for the intersection are being specified, while the lane group is of importance when the level of service at the intersection is being computed.
- **Lane Group**: Lane groups for each approach are established following these guidelines:

  • Separate lane groups should be established for exclusive left-turn lane(s).

  • Separate lane groups should be established for exclusive right-turn lane(s).

  • A separate lane group should be established for each shared lane.

  • A single lane group should be established for all lanes that are either exclusive turn lanes or shared lanes.

  • When exclusive left-turn lane(s) and/or exclusive right-turn lane(s) are provided on an approach, all other lanes are generally established as a single lane group.

  • When an approach with more than one lane also has a shared left-turn lane, the operation of the shared left-turn lane should be evaluated to determine whether it is effectively operating as an exclusive left-turn lane because of the high volume of left-turn vehicles on it.

- **Passenger Car Unit (PCU):** The representation of a vehicle in equivalent units of standard passenger car under specific roadway, traffic and control conditions.
- **Phase**: The part of the signal cycle allocated to any combination of traffic movements receiving the right-of-way (green time) simultaneously
- **Phase composition**: The combination of vehicular, pedestrian and other movements, if any, legally permitted during a phase.
- **Phase sequence**: The order in which the phases follow each other in a cycle.
- **Queue:** A line of vehicles waiting at the stop line for the green phase to be served by a signalized intersection. Traffic moving slowly and joining the rear of the queue is usually considered as part of the queue. The internal queue dynamics may involve a series of stops and starts.
- **Queue Length**: The number of vehicles in a queue, or the longitudinal distance which is covered by the queue at the stop line of the approach of a signalized intersection.
- **Saturation Flow Rate (SF):** It is the steady state discharge rate of queued vehicles from an approach at a signalized intersection with continuous green and an infinite queue. In practice, it is measured as the maximum departure rate of queued vehicles from an approach during the green interval measured at the stop line under prevailing conditions. It is expressed in PCU/hour of green.
- **Start-up lost time:** The additional time, in seconds, consumed by the first few vehicles in a queue at a signalized intersection because of the need to react to the initiation of the green phase and to accelerate.

2.6.1 Signal Warrants (Hoel, 2018):

"One of the most effective ways of controlling traffic at an intersection is the use of traffic signals. By allowing different traffic streams to use the shared space at different times , conflicts can be avoided. However, this results in a delay for all vehicles, so it's important to only use traffic signals when required. The primary factor in determining whether traffic signals are necessary at a particular intersection is the volume of traffic approaching the intersection, although other factors such as pedestrian traffic and accident history may also be important. The Manual on Traffic Signal Design gives the vital concepts and standard practices used in the design of traffic signals."

The (MUTCD, 2009) describes nine warrants in detail, at least one of which should be satisfied for an intersection to be signalized.

The factors considered in the warrants are:

• Warrant 1. eight-hour vehicular volume

• Warrant 2. Four-hour vehicular volume

• **Warrant 3. Peak hour**

• **Warrant 4. Pedestrian volume**

• Warrant 5. School crossing

• Warrant 6. coordinated signal system

• **Warrant 7. crash experience**

• Warrant 8. roadway network

• Warrant 9. Intersection near a grade crossing

"For example: Warrant 3 is used to justify the installation of traffic signals at intersections where traffic conditions during one hour or longer of an average day result in undue delay to traffic on the minor street entering or crossing the intersection. The conditions are:" (Hoel, 2018)

A. The three following criteria should be satisfied for the same hour (any four consecutive 15-minute periods) of an average day:

1. The total stopped delay during any four consecutive 15-minute periods on one of the minor-street approaches (one direction only) controlled by a stop sign is equal to or greater than 4 veh-h for a one-lane approach and 5 veh-h for a two-lane approach.

2. The same minor-street approach (one direction only) volume should be equal to or exceed 100 and 150 vehicles per hour for one moving lane of traffic and two moving lanes of traffic, respectively.

3. The total intersection entering volume is equal to or greater than 650 vehicles per hour for three-leg intersections and 800 vehicles per hour for four-leg and multi leg intersections.

Fully controlled signalized intersections provide a clear indication to drivers of when it is safe to enter the intersection, thereby eliminating the need for them to assess and choose safe gaps in traffic flow. This, in turn, helps to lower the incidence of collisions involving turning vehicles and oncoming through traffic. Hence, traffic signals are a way to stop conflicting flows of traffic entering the intersection at the same time and can reduce crash risk.

The distribution of green times among these conflicting flows significantly affects both capacity and operation of the intersection. Other factors such as lane widths, traffic composition, grade, and speed also affect the level of service at intersections in a similar manner as for highway segments.

2.6.2 SIDRA INTERSECTION SOFTWARE

"SIDRA INTERSECTION is an advanced micro-analytical traffic evaluation software tool designed for the design and evaluation of individual intersections and networks of intersections. It can be used to analyze a variety of traffic flow conditions including signalized intersections, pedestrian crossings, roundabouts, interchanges, and uninterrupted traffic flow conditions. The software allows modeling of different vehicle types, movement classes, and signal phases to provide estimates of capacity and performance statistics such as delay, queue length, and stop rate. It employs a lane-by-lane and vehicle path model coupled with an iterative approximation method to provide more accurate estimates. The software also takes into account midblock lane changes and provides facilities to calibrate its traffic models for local conditions. Unlike traditional network models, SIDRA INTERSECTION uses a lane-based model to create second-by-second platoon arrival and departure patterns for signalized sites to calculate signal coordination effects as a function of signal offsets for internal approaches in network analysis. In particular, the US HCM (Customary and Metric) software setups of SIDRA INTERSECTION is calibrated using model parameters based on the US Highway." (Akcelik, 2021)

"Capacity Manual. Among many model parameters, the saturation flow parameter for signalized intersections and the critical gap and follow-up headway parameters for unsignalized roundabouts and sign-controlled intersections are identified as key parameters for calibration to match real-life traffic conditions" (Akçelik R., 2020)
"SIDRA INTERSECTION is a unique lane-based model that can identify backward spread of congestion, midblock lane changes and unequal approach lane use at closely-spaced intersections. This unique lane-based micro-analytical network model allows analysis of all intersection types– signals, roundabouts, sign control - in one network." (Sidrasolutions.com, n.d.)

"SIDRA Intersection serves as a tool to assist in the design and assessment of various types of intersections, including signalized intersections (both fixed-time and actuated), signalized pedestrian crossings, roundabouts, single point interchanges, roundabout metering, two-way stop sign control, all-way stop sign control, and give-way/yield sign control." (Besley, 2003)

2.6.3 FOUR LANING OF HIGHWAYS

Four-laning of highways refers to the process of expanding a two-lane roadway to four lanes. This is typically done to improve the capacity and safety of the roadway, as well as to accommodate increased traffic volumes.

The process of four-laning a highway typically involves widening the existing roadway to add two additional lanes, as well as making other improvements to the roadway, such as adding turn lanes, improving drainage, and installing safety features like guardrails and rumble strips.

Four-laning of highways can provide several benefits, including reducing congestion and improving travel times, as well as improving safety by reducing the potential for head-on collisions and other types of Crash. Additionally, four-laning of highways can improve access to rural areas and support economic development by improving transportation infrastructure.

However, the process of four-laning highways can also have negative impacts, particularly on local communities and the environment. For example, expanding a roadway may require the acquisition of private property or the displacement of residents, businesses, or cultural resources. Additionally, the construction and operation of a four-lane highway can have negative impacts on air and water quality, as well as wildlife habitat and other natural resources.

To address these concerns, transportation agencies and communities may engage in community outreach and consultation to identify potential impacts of four-laning projects and to develop strategies to mitigate these impacts. This may include designing roadways that minimize impacts on nearby communities and natural resources, as well as implementing measures to reduce air and water pollution and support alternative transportation options.

2.6.4 HIGHWAY CAPACITY MANUAL

"The Highway Capacity Manual (HCM) is a widely popular reference manual provides a set of techniques to evaluate the quality of service on highway and street facilities and intersections, in a consistent manner. The HCM does not dictate policies regarding the quality of service but aims to present the latest research results and methods for practitioners to assess transportation facilities. The manual provides a standardized approach to assess the capacity and level of service for surface transportation systems and individual facilities, including pedestrian and bicycle facilities. The manual provides analytical procedures for different performance measures and establishes a link between operational and planning models to evaluate broader systems of facilities. It is designed for use by traffic engineers, traffic operations personnel, design engineers, planners, management personnel, teachers, and university students with some technical background or training." (HCM, 2000)

Chapter 16 of the HCM: Signalized Intersection is of the importance for this project.

2.6.5 INDO-HCM:

Indo-HCM is a customized version of the Highway Capacity Manual (HCM) developed specifically for use in India. The Indo-HCM is based on the HCM 2010 and includes modifications to reflect the unique characteristics of Indian traffic and roadway conditions. The Indo-HCM provides guidelines for assessing the performance and capacity of Indian roadways, intersections, and pedestrian facilities, and is intended for use by transportation professionals and researchers in

India. The Indo-HCM is a result of a research project sponsored by the Council of Scientific and Industrial Research (CSIR) in India.

The HCM procedure can be used to evaluate the performance of a signalized intersection with respect to the quality of service provided to the road users at the intersection. It also provides performance measures for different travel modes that can be used to identify problem sources and help the analyst to develop appropriate strategies for development. Although the procedure is primarily for an isolated intersection, the impact of a preceding signalized intersection is considered by introducing input variables that reflect the arrival time distribution of vehicles from an upstream signalized intersection.

The capacity at a signalized intersection is given for each lane group and is defined as the maximum rate of flow for the subject lane group that can go through the intersection under prevailing traffic, roadway, and signalized conditions. Capacity is given in vehicles per hour (veh/h) but is based on the flow during a peak 15-min period. The capacity of the intersection as a whole is not considered; rather, emphasis is placed on providing suitable facilities for the major movements of the intersections. Capacity therefore is applied meaningfully only to major movements or approaches of the intersection. Note also that in comparison with other locations such as freeway segments, the capacity of an intersection approach is not as strongly correlated with the level of service. (Indo-HCM, 2017)

Saturation Flow Rate: The concept of a saturation flow or saturation flow rate (s) is used to determine the capacity of a lane group. The saturation flow rate is the maximum flow rate on the approach or lane group that can go through the intersection under prevailing traffic and roadway conditions when 100 percent of the green time is available. The saturation flow rate is given in units of veh/h of effective green time. (Indo-HCM, 2017)

Base Saturation flow rate: Saturation flow rate under stated base conditions of intersection relating to traffic, geometric and control conditions and is expressed in PCU/h. of green. (Indo-HCM 2017)

Section 6.6 of HCM 2017 for the calculation of Saturation flow rate is best suited among other methods for the case of Nepalese intersection.

The types of signals at isolated intersections can be pretimed (fixed), semi actuated, or fully actuated. The right-of-way at pre-timed signals is assigned to different traffic streams in accordance with a fixed timing program. A signal has a pre-determined cycle length for a specific period of the day or for the entire day and does not change based on changes in traffic volume. Several design strategies exist for determining the optimum cycle length. Three strategies frequently used for pre timed signals are:

- Minimizing the total delay at the intersection. In some cases, other performance measures such as number of stops and fuel consumption are incorporated in the optimization factor. An example methodology of this strategy is the Webster method.
- A second strategy is to ensure that the volume-to-capacity ratios for the critical lane groups are the same by assigning the green times among the different phases in the same proportion as the flow ratio of their respective critical lane groups.
- A third strategy is to ensure that the levels of service for all critical lane groups are the same. Although this strategy is an improvement on the two described above, as it leads to obtaining a level of service at each approach that is similar to the overall level of service of the intersection, it tends to give the movements on the minor approaches a higher delay per vehicle. (Indo-HCM, 2017)

The Highway Capacity Method: (Hoel, 2018)

This method is used to determine the cycle length, and it is based on the capacity (the maximum flow based on the available effective green time) of a lane group. Since the saturation flow rate is the maximum flow rate on an approach or lane group when 100 percent effective green time is available, the capacity of an approach or lane group depends on the percentage of the cycle length that is given to that approach or lane group. The capacity of an approach or lane group is given as:

Capacity(ci):

$$ci = si\ \frac{gi}{C}$$

Where,

c_i= capacity of lane group i (veh/h),

s_i = saturation flow rate for lane group i (veh/h), and

g_i /C = effective green ratio for lane group i.

The ratio of flow to capacity (v/c) is usually referred to as the degree of saturation and can be expressed as:

Degree of saturation (Xi):

$$X_i = \left(\frac{v}{c}\right)_i = \frac{v_i}{s_i\left(\frac{g_i}{C}\right)} = \frac{v_i C}{s_i g_i}$$

where

X_i = $(v/c)_i$ = ratio for lane group i,

v_i = actual or projected demand flow rate for lane group i (veh/h),

s_i = saturation flow rate for lane group i (veh/h),

g_i = effective green time for lane group i (s), and

C = cycle length (s).

(HCM, 2000)

When the overall intersection is to be evaluated with respect to its geometry and the total cycle time, the concept of critical volume-to-capacity ratio (Xc) is used. The critical (v/c) ratio is usually obtained for the overall intersection but considers only the critical lane groups or approaches, which are those lane groups or approaches that have the maximum flow ratio (v/s) for each signal phase. For example, in a two-phase signalized intersection, if the north approach has a higher (v/s) ratio than the south approach, more time will be required for vehicles on the north approach to go through the intersection during the north-south green phase, and the phase length will be based on the green time requirements for the north approach. The north approach will therefore be the critical approach for the north-south phase. The critical v/c ratio for the whole intersection is given as: (Hoel, 2018)

Critical v/c ratio (Xc):

$$X_c = \Sigma\left(\frac{v}{s}\right)_{ci}\left(\frac{C}{C-L}\right)$$

where

X_c = critical v/c ratio for intersection;

$\Sigma\left(\frac{v}{s}\right)_{ci}$ = summation of flow ratios for all critical lane groups i;

C = cycle length (s); and

L = total lost time per cycle, computed as lost time, t_L, for critical path of movements (s).

(HCM 2000)

The cycle length can be derived from the above formula for a required Xc.

2.7 PARKING SUPPLY SURVEY

The Banepa-Araniko highway section is an important transportation route that connects the Kathmandu valley with the eastern region of Nepal. The government has recently announced plans to expand the highway section to accommodate the increasing traffic volume, which will require the displacement of parking spaces along the highway. In response to this issue, a parking supply survey was conducted to gather data on the current parking situation and explore alternative parking solutions to address the potential displacement of parking spaces.

This report provides an overview of the parking supply survey of the Banepa-Araniko highway section and focuses on the displacement of parking spaces due to the highway expansion. The report aims to provide insights into the current parking situation, including the availability and accessibility of parking spaces, parking demand, and the impact of the highway expansion on parking behavior. Additionally, the report will explore potential alternative parking solutions and recommendations for mitigating the impact of the parking displacement on commuters, residents, and businesses in the area.

On-street Parking and Off-street Parking

On-street parking refers to the practice of parking a vehicle on a public street or roadway. Drivers are typically allowed to park their vehicles in designated spaces along the side of the road, as long as they comply with local parking regulations and restrictions.

Off-street parking, on the other hand, refers to parking facilities that are located off the public street or roadway. This can include parking garages, parking lots, or private driveways. Off-street parking is typically designed to provide more secure and convenient parking options for drivers, particularly in areas where on-street parking is limited or restricted.

Both on-street and off-street parking have their advantages and disadvantages. On-street parking is often more convenient for drivers, as they can park closer to their destination. However, it can be more difficult to find a space during peak times, and drivers may be subject to parking restrictions and ticketing. Off-street parking is typically more secure and offers more space, but it can be more expensive and may require a longer walk to reach the final destination.

Displacement of On-street Parking

Displacement of on-street parking due to road expansion can occur when a municipality or transportation authority decides to widen or improve a roadway, which may result in the removal of on-street parking spaces. This can be a significant concern for businesses and residents who rely on on-street parking for convenient access to their homes or workplaces.

In some cases, displacement of on-street parking due to road expansion can be mitigated by providing alternative parking options nearby. For example, a municipality may decide to build a new parking lot or parking garage to replace the on-street parking spaces that are lost due to road expansion. However, this can be costly and may not always be feasible, particularly in urban areas where space is limited.

To minimize the impact of on-street parking displacement, municipalities and transportation authorities may engage in community outreach and consultation to gather feedback from stakeholders who may be affected by the loss of parking. This can help to identify alternative parking options or transportation alternatives, such as public transit or bike lanes, that can help mitigate the effects of on-street parking displacement. Additionally, policies such as permit parking or time limits on parking may be implemented to ensure that available parking spaces are used efficiently and fairly.

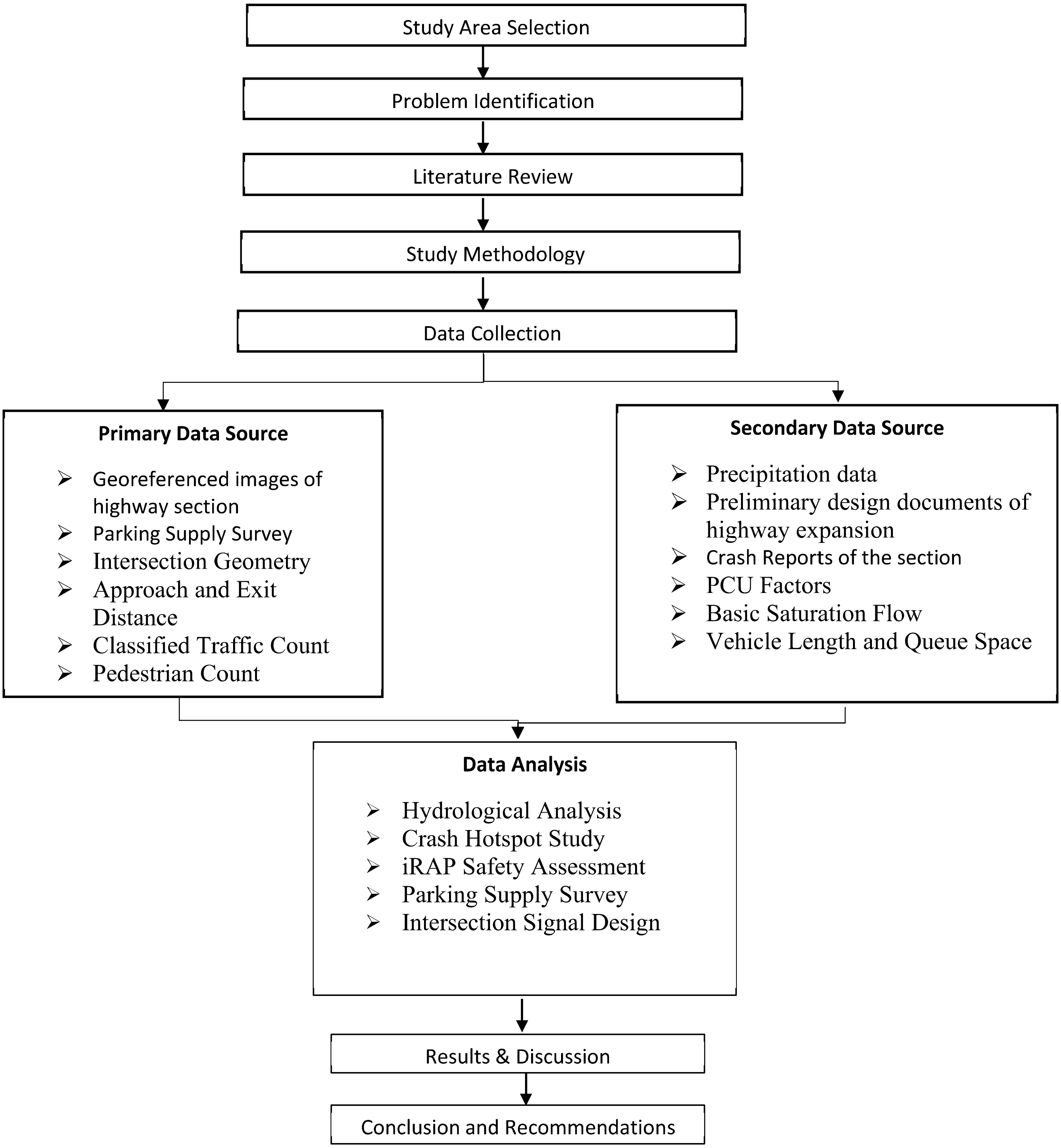
Supply side of Displaced Parking

The supply side of parking refers to the factors that influence the availability of parking spaces. This includes the number and types of parking facilities available, as well as their location and pricing.

On the supply side, there are several factors that can affect the availability of parking. For example, the number and size of parking facilities can impact the total number of available spaces. The location of parking facilities can also play a role, as areas with high demand for parking may require more parking facilities in close proximity. Additionally, pricing can affect the supply of parking, as higher prices may deter drivers from using certain parking facilities, while lower prices may attract more drivers.

Municipalities and businesses often have to consider the supply side of parking when planning for new development or managing existing parking facilities. This can involve conducting parking studies to determine the demand for parking in a given area and identifying opportunities to increase or improve the supply of parking. Strategies for increasing the supply of parking can include building new parking facilities, converting existing spaces into parking areas, or implementing policies that encourage more efficient use of parking resources.

3 METHODOLOGY



At various stages of the project study, different methodologies were used to prepare the report and conduct the analysis:

1. **Planning:** Reconnaissance and map study (preliminary, inventory and secondary)

2**. Site Visit:** site visit to Punyamati river and its flooding problem with assistance of municipal engineer; meeting with mayor and interaction with locals

3.**Literature review:** extensive research on available materials and gathering every resource available for possible use.

4. **Brainstorming:** search for extensive solution of flooding problem trying all possible remedial measures

5. **Traffic volume counting:** video capturing using multiple mobile camera; Manual counting from video and their validation

6. **Crash studies**: collection of crash record and carrying out safety analysis for severity diagram and high frequency location

7. **Star ratings using iRAP**: use of iRAP tool to code the roadside and safety facilities to obtain star ratings; purpose estimate of safety measures to be installed

8. **Intersection Design**: intersection design for existing unsignalized intersection using traffic analysis software SIDRA intersection; design of Proposed signalized intersection

9. **Parking supply survey:** survey for displaced parking facilities from supply side

11. **Design overview and overlay:** review of design facilities and provide improvements wherever needed

12.**Documentation:** presentation of all obtained data and results from analysis of highway section from different perspective

Primary data and information were generated through field observation and surveys (i.e. questionnaire to the locals). After that we came to know their opinions and problems which they are facing for a long time, mainly during the monsoon. Secondary information was collected through published and unpublished reports and maps. Detailed Drawings were collected from Department of Roads (DOR), Rainfall data from Department of Hydrology and Meteorology (DHM) and CCTV footage of traffic volume from Regional Police Office Banepa. After this we gathered and reviewed topographical maps of the site, GIS data, available reports, guidelines, and other site-related information.

3.1 HYDROLOGICAL ANALYSIS

Watershed Delineation:

Delineating a watershed means identifying and drawing the watershed boundary on a topographic map. (teachengineering, n.d.) It refers to the creating a boundary that represents the contributing area for a particular control point or outlet. Arc GIS was used to delineate watersheds of the rivers in our study area. A representative image of delineated watershed using ArcGIS is shown in Figure 3.1.1 . Following steps were used to delineate watershed in ArcGIS (Karn, 2022):

1. **Setting up of working environment**

· Open ArcMap and create a new, blank map document.

· Use the Add Data button to add the shapefile for your area of interest to the map.

2. Run the Fill tool.

· In ArcCatalog, navigate to Toolboxes > System Toolboxes > Spatial Analyst Tools > Hydrology > Fill.

· Use a digital elevation model (DEM) as the 'Input surface raster'.

· Verify the path name for the 'Output surface raster'.

· Click OK·.

3. **Run the Flow Direction tool.**

· In ArcCatalog, navigate to Toolboxes > System Toolboxes > Spatial Analyst Tools > Hydrology > Flow Direction.

· Use the DEM output from Step 2 as the 'Input surface raster'.

· Verify the path name for the 'Output flow direction raster'.

· Click OK.

4. **Run the Flow Accumulation tool.**

· In ArcCatalog, navigate to Toolboxes > System Toolboxes > Spatial Analyst Tools > Hydrology > Flow Accumulation.

Use the output raster from Step 3 as the 'Input flow direction raster'.

Verify the path name for the 'Output accumulation raster'.

· Click OK.

5. **Run the Snap Pour Point tool to locate the pour points to cells of high accumulated flow.**

· In ArcCatalog, navigate to Toolboxes > System Toolboxes > Spatial Analyst Tools > Hydrology > Snap Pour Points.

· Either input a point feature class or a raster as the 'Input raster or feature pour point data'.

· Use the output raster from Step 4 as the 'Input accumulation raster'.

· Verify the path name for the 'Output raster'.

· Click OK.

6. Run the Watershed tool.

· In ArcCatalog, navigate to Toolboxes > System Toolboxes > Spatial Analyst Tools > Hydrology > Watershed.

· Use the output raster from Step 3 as the 'Input flow direction raster'

· Use the output from Step 5 as the 'Input raster or feature pour point data'.

· Verify the path name for the 'Output raster'.

· Click OK.

7.: Run the 'Raster to Polygon' tool to create polygon features from the watershed raster.

· In ArcCatalog, navigate to Toolboxes > System Toolboxes > Conversion Tools > From Raster > Raster to Polygon.

· Use the output from Step 5 as the 'Input raster'.

· Verify the path name for the 'Output polygon features'.

· Click OK.

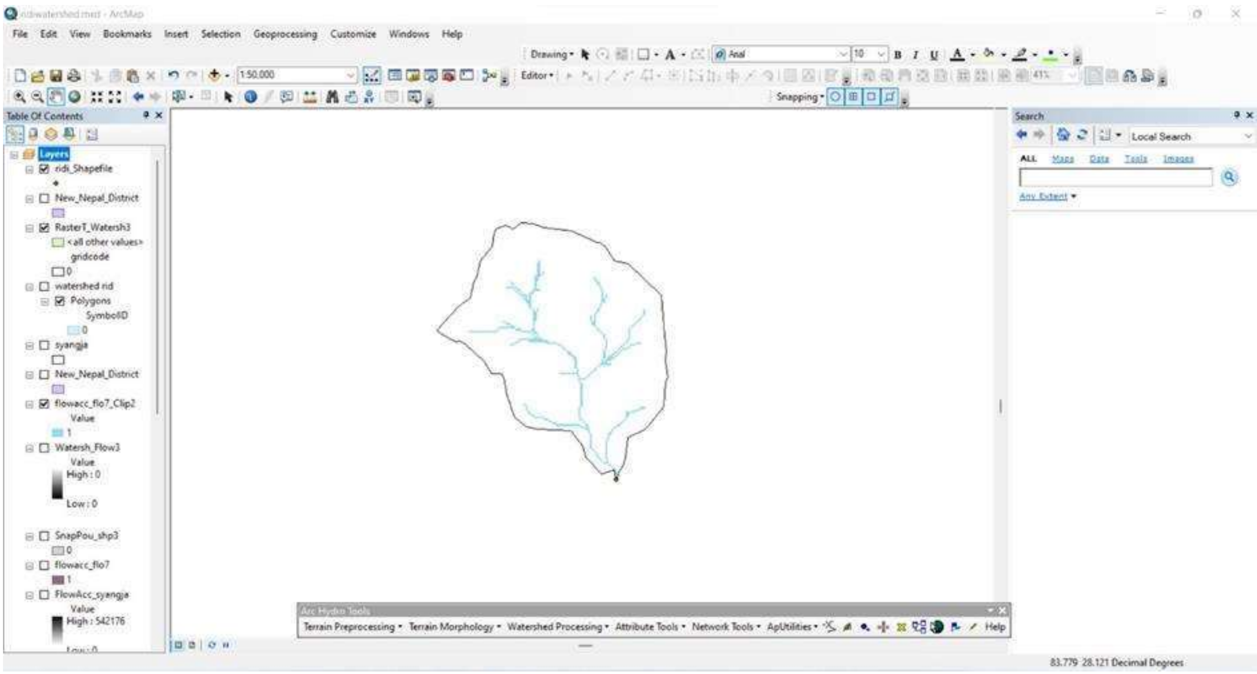


Figure 3.1.1 Watershed Delineation using ArcGIS

Estimation of Rainfall Intensity:

A. Frequency analysis of extreme rainfall data of rain gauge stations

Gumbel Distribution was used for frequency analysis of rainfall data. It involved following procedures:

· List and arrange annual floods (x) in descending order of magnitude.

· Assign rank 'm', m = 1 for highest value and so on.

· Calculate return period (T) and/or probability of exceedance (P) by equations n + 1/m and m/n +1 respectively. These values together with respective flood magnitude give plotting positions.

· Using tabular form calculate x^2 and ∑x and Ex^2.

· Now calculate mean x; squared mean x^2; mean of squares x^2 and standard deviation S.

· From the

Table 3.1.1 of frequency factors for Gumbel method, the values for desired return periods and the available sample size can be taken.

- Using relation x = X + KS calculate flood values for various return periods. Using the extreme value probability paper, plot the x values against respective return periods or P values and join the points to obtain the required frequency curve. (Dulal, 2012)

Table 3.1.1 Gumbel Table

Reduced mean $\bar{y}_n$ in Gumbel's Extreme Value Distribution

N = sample size

N	0	1	2	3	4	5	6	7	8	9
10	0.4952	0.4996	0.5035	0.5070	0.5100	0.5128	0.5157	0.5181	0.5202	0.5220
20	0.5236	0.5252	0.5268	0.5283	0.5296	0.5309	0.5320	0.5332	0.5343	0.5353
30	0.5362	0.5371	0.5380	0.5388	0.5396	0.5402	0.5410	0.5418	0.5424	0.5430
40	0.5436	0.5442	0.5448	0.5453	0.5458	0.5463	0.5468	0.5473	0.5477	0.5481
50	0.5485	0.5489	0.5493	0.5497	0.5501	0.5504	0.5508	0.5511	0.5515	0.5518
60	0.5521	0.5524	0.5527	0.5530	0.5533	0.5535	0.5538	0.5540	0.5543	0.5545
70	0.5548	0.5550	0.5552	0.5555	0.5557	0.5559	0.5561	0.5563	0.5565	0.5567
80	0.5569	0.5570	0.5572	0.5574	0.5576	0.5578	0.5580	0.5581	0.5583	0.5585
90	0.5586	0.5587	0.5589	0.5591	0.5592	0.5593	0.5595	0.5596	0.5598	0.5599
100	0.5600									

Reduced Standard Deviation S_n in Gumbel's Extreme Value Distribution

N = sample size

N	0	1	2	3	4	5	6	7	8	9
10	0.9496	0.9676	0.9833	0.9971	1.0095	1.0206	1.0316	1.0411	1.0493	1.0565
20	1.0628	1.0696	1.0754	1.0811	1.0864	1.0915	1.0961	1.1004	1.1047	1.1086
30	1.1124	1.1159	1.1193	1.1226	1.1255	1.1285	1.1313	1.1339	1.1363	1.1388
40	1.1413	1.1436	1.1458	1.1480	1.1499	1.1519	1.1538	1.1557	1.1574	1.1590
50	1.1607	1.1623	1.1638	1.1658	1.1667	1.1681	1.1696	1.1708	1.1721	1.1734
60	1.1747	1.1759	1.1770	1.1782	1.1793	1.1803	1.1814	1.1824	1.1834	1.1844
70	1.1854	1.1863	1.1873	1.1881	1.1890	1.1898	1.1906	1.1915	1.1923	1.1930
80	1.1938	1.1945	1.1953	1.1959	1.1967	1.1973	1.1980	1.1987	1.1994	1.2001
90	1.2007	1.2013	1.2020	1.2026	1.2032	1.2038	1.2044	1.2049	1.2055	1.2060
100	1.2065									

Source: (Dulal, 2012)

Use of Mononobe's equation to estimate rainfall intensity

Mononobe's equation was used to calculate the rainfall intensity for various return periods. The Mononobe's equation is given as:

$$Itc = \left(\frac{R}{24}\right) * \left(\frac{24}{tc}\right)^{\frac{2}{3}}$$

where,

tc= time of concentration in hour

R=Daily maximum rainfall of T return period obtained from frequency analysis in mm

I_{tc} = Intensity of rainfall in mm/hr

Estimation of Design Flood:

A design flood is a hypothetical flood (peak discharge or hydrograph) adopted as the basis in engineering design of project components. (Dulal, 2012) Estimation of the design flood is usually carried out by fitting observed plotting position data with a suitable probability distribution.

Rational method was used to calculate the flood discharge of the rivers in the study area.

A rational formula, for flood discharge, considers the intensity, distribution, and duration of rainfall as well as the area, slope, and permeability of the basin. A typical rational formula is:

$$Q_T = \frac{CIA}{360}$$

Where,

Q_T is the maximum flood discharge in for the required return period T

C is the runoff coefficient and can be selected based on the type of basin. The values of C for different types of basins are given in

Type of Basin	C
Rocky & Permeable	0.8 – 1.0
Slightly impermeable, bare	0.6 – 0.8
Cultivated or Covered with vegetation	0.4 – 0.6
Cultivated absorbent soil	0.3 – 0.4
Sandy soil	0.2 – 0.3
Heavy forest	0.1 – 0.3

Table 3.1.2.

A is the catchment area in hectare

I is the mean rainfall intensity in mm/hour for time of concentration tc (Dhakal, 2019).

Type of Basin	C
Rocky & Permeable	0.8 – 1.0
Slightly impermeable, bare	0.6 – 0.8
Cultivated or Covered with vegetation	0.4 – 0.6
Cultivated absorbent soil	0.3 – 0.4
Sandy soil	0.2 – 0.3
Heavy forest	0.1 – 0.3

Table 3.1.2 Runoff Coefficient Range for different materials

Source: www.dhakalvivek.com.np

Determination of dimension of the cross section and side drain

Manning's equation was used to calculate the depth of flow for the given width of the river. The manning's roughness coefficient and the slope of the river were taken from the design document provided by the Department of Roads.

Manning's equation relates the flow of water in an open channel to the geometry of the channel, the roughness of the channel surface, and the slope of the channel. It is expressed as:

$$Q = \frac{A}{n} R^{2/3} S^{1/2}$$

Where,

Q = discharge (volume of water flowing per unit time)

n = Manning's roughness coefficient

A = cross-sectional area of the channel = b*y (assume rectangular channel)

R = hydraulic radius = (cross-sectional area divided by wetted perimeter)

P= b+2y (for rectangular section)

S = slope of the channel

b = width of the river

y= depth of flow

Steps involved:

For bridges,

1. 100-year design flood(Q_{100}) is estimated.
2. Manning's coefficient(n) and bed slope (S) is taken from the judgment of an expert.
3. Depth 'y' is computed using manning's equation.
4. Assume minimum freeboard of 1m then minimum depth of bridge = y+1 m.

For side drain,

1. 50-year design flood(Q_{50}) is estimated.
2. Manning's coefficient(n) is taken from the judgment of an expert.
3. Bed slope (S) is taken from the road profile.

4. Manning's equation is used to estimate the cross section of the longitudinal drain.

Using hit and trial, a section is selected such that its discharge carrying capacity is more than the required capacity and the velocity of flow is within the safe limit.

3.2 Excel and GIS for Crash studies

- After collecting data from traffic records, they are converted into analyze table form and presented in excel format.
- With the help of excel data are studied and classified into different elements of importance. Such analyzed data are presented in tables and are better described using various bar charts and graphs.
- Severity location and point of maximum crash occurrence in terms of fatal, major, minor, pedestrian, and many more are classified into the individual format.
- Such classified data are plotted and presented by using the QGIS software tool.

3.3 iRAP for SAFETY AUDIT

Data Collection

Karta view app was used to collect the images of the road section from Pulbazar to Chandeshwori. The observer sat on a motorcycle by keeping the camera on through the Karta view app and the vehicle was moved throughout the road section. The Karta view app generated the georeferenced image which was used for further analysis in Vida.

Data Analysis

For each 100 m segment, the georeferenced image was observed carefully and the parameters were entered in www.demonstrator.vida.iRAP.org. The road side parameters were observed and based on iRAP Coding Manual; the parameters were selected in the Vida. The AADT value of the Pulbazar- Chandeshwori road section was obtained from a report of JICA,2018. Once the data entry was completed, the website generated the star rating for car, motorcycle, pedestrian and bicycle. A minimum of 3 stars is recommended by iRAP for each vehicle type. Some changes in the parameters were done to those road sections for which vehicle types had less than 3-star rating so that a minimum of 3 stars could be achieved.

Coding Procedures

The basic algorithm for coding is explained as followings:

1. At first the type of highway was chosen and it was found to be a close resemblance to a low standard urban highway.
2. The road side severity was analyzed for different sections and were given as input under the Roadside icon in the demonstrator. Most of the sections constituted parked vehicles which were treated as semi rigid structures and respective distances were given as input which was observed in the field.
3. The various physical characteristics including the geometric features, lighting condition, functional criteria as well as other parameters as desired in the coding sequence were entered as per the field conditions.
4. The various characteristics of intersection were given as input if present at the section to be coded.
5. The flow characteristics including the Average Annual Daily Traffic (20000), vehicular flow characteristics and peak flows of various vehicle types were given into the demonstrator from the observations made in the field.
6. Under the VRU facilities and land use, various characteristics of pedestrian crossing facilities as well as facilities for motorbikes and bicycles were given as input as observed in the field.
7. The differential speed limits as well as the observed 85 percentile speed was noted in the field and such value was entered into the demonstrator.
8. At last star rating was obtained which was then altered by making changes in the road facilities and various parameters so as to obtain a minimum rating of 3 stars.

3.4 SIDRA for INTERSECTION DESIGN AND ANALYSIS

The study used the following methodology for the design of Signal for Intersections in Banepa

1.Data Collection

a. Primary Data Sources

i. Intersection Geometry

ii. Approach and Exit Distance
iii. Classified Traffic Count
iv. Pedestrian Count
v. Signal Timing
vi. Cruise Speed
vii. Back of Queue

b. Secondary Data Sources
i. PCU Factors
ii. Basic Saturation Flow
iii. Growth Rate
iv. Vehicle Length and Queue Space

2. Data Analysis using SIDRA

SIDRA Intersection was used to model the existing case as well as proposed case.

3.5 GNSS for PARKING SUPPLY SURVEY

- We members of our project group had carried out survey on supply side of parking facilities of Banepa Araniko highway section by using GNSS survey using SW Maps Mobile software
- Both on street and off-street parking facilities within 100-200 m from the road section are located and recorded by creating layers and attributes.
- Attributes like distance, route, capacity and type of parking -on-street or off-street parking are created and data are recorded accordingly by creating a plot in google maps
- Thus, obtained data is post-processed by using QGIS tool to obtain information in the presentable form with location capacity pictures and route.

3.6 AUTOCAD AND SKETCHUP for DESIGN OVERLAY AND PRESENTATION

- AutoCAD file of the design document was gathered and analyzed thoroughly.
- Comparison is made between original design document with our proposed design of intersection using SIDRA tool

- considering safety and level of service as utmost priority improvements are made in design document
- improvement is shown by overlying layers of improvement in design document
- Finally, a SketchUp presentation was made with an aim to visualize the interface.

4 DATA EXPLORATION /ANALYSIS

4.1 HYDROLOGICAL ANALYSIS

Because of restriction on size of the report and to enhance comprehensiveness a sample of observation is only presented here.

Refer Appendix A for Detailed Meteorological data.

Design flood estimation of Rivers

Estimation of 100-year design flood for Punyamati river, Chandeshwori river and Chardobato outlet involves following procedures:

1. Punyamati River

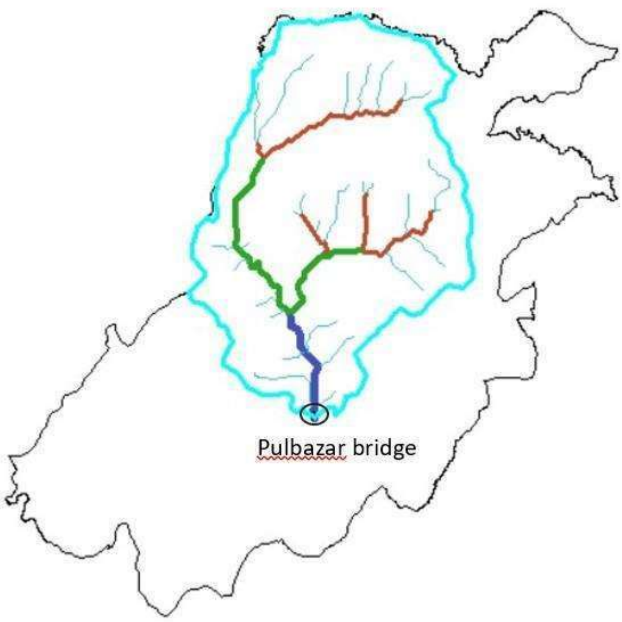


Figure 4.1.1 Catchment area of Punyamati river with outlet at Pulbazar bridge

The delineation of catchment area using ArcGIS at Pulbazar is shown in Figure 4.1.1 . The data of various parameters obtained are listed below.

Catchment area = 21.76 km^2

Longest stream length = 8959.36 m

Head difference = H2-H1 = 2166 – 1452 = 714 m

Slope(S) $=\frac{H_2-H_1}{Lc} = \frac{714}{8959.36}$ =0.0797

Using,

Time of concentration (tc)=$0.01947L^{0.77}\,S^{-0.385}$

tc = 56.96 min

Coefficient of runoff:

Table 4.1.1 Estimation of Weighted Runoff Coefficient

	Area(km ^{2})	% Area(A)	Runoff coefficient (C)	AxC/100
Forest	7.011	32.22	0.2	0.0644
Agricultural	13.67	62.82	0.35	0.2198
Residential	1.079	4.95	0.9	0.0445
			Total	**0.3287**

The Table 4.1.1 shows that the weighted runoff coefficient of the catchment area is **0.3287.**

Thiessen Polygon of rainfall station

The figure 4.1.2 shows that the total runoff in the Pulbazar catchment is due to the contribution of station 1082, 1024, and 1043.

The Thiessen Polygon area contributed by these stations are listed below:

Station 1082= 4.349 km ^{2}

Station 1024= 4.269 km ^{2}

Station 1043= 13.15 km ^{2}

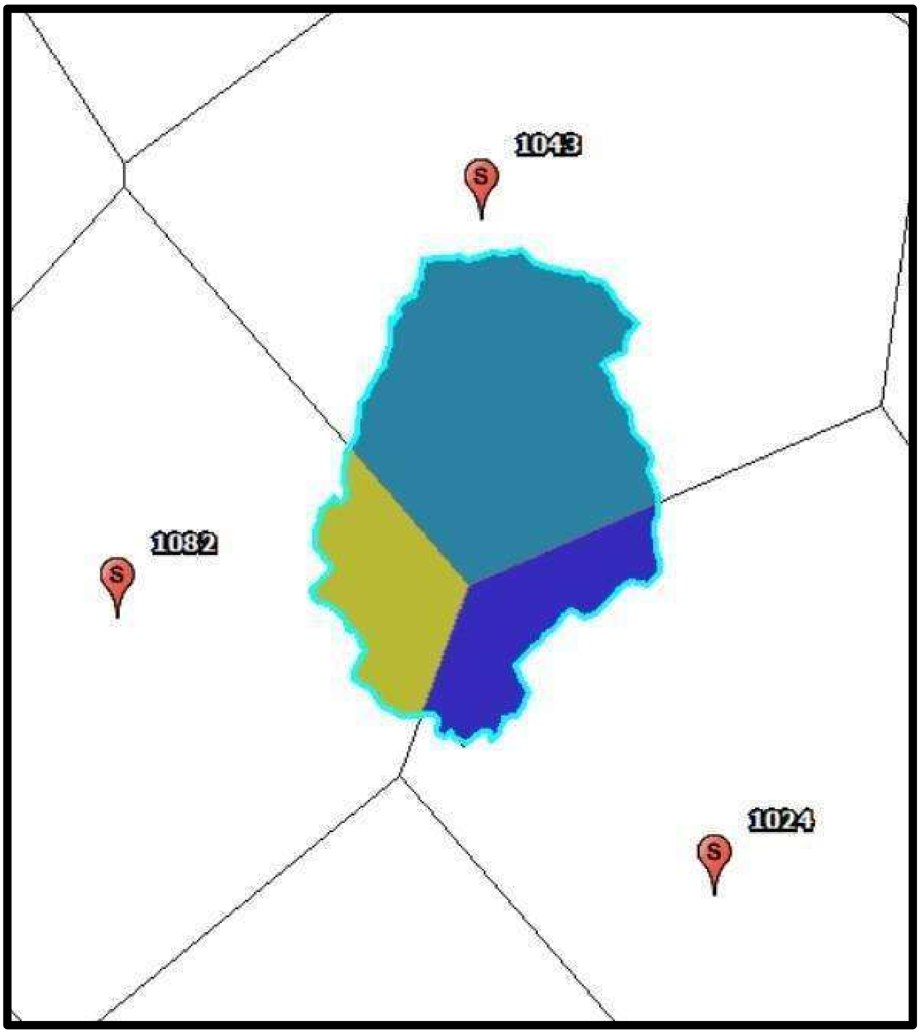


Figure 4.1.2Thiessen polygon of watershed at Pulbazar

Flood Estimation

A. Rational Method

Area = 21.76 km^2 = 2176 ha

Length of Concentration (Lc) = 8959.36 m

Head(H) = $H_2 - H_1$ = 2166 – 1452 = 714 m

Slope(S) = $\frac{H_2-H_1}{Lc} = \frac{714}{8959.36}$ =0.0797

Coefficient of runoff (C)= 0.3287

t=tc= 56.96 min = 0.949 hr

Frequency Analysis

Frequency analysis of station 1024, 1043 and 1082 were carried out and the Gumbel distribution gave the best result for all the stations.

Sample calculation of frequency analysis of station (1043) is shown in Table 4.1.2 which shows that a 100-year rainfall is obtained as 179.7 mm.

Table 4.1.2 Frequency Analysis

Return period (T)	YT=-ln(ln(T/T-1))	KT=((YT-Yn)/Sn)	XT=X+KT*std (mm)
10	2.26	1.55	129.39
20	2.98	2.2	144.89
33	3.49	2.66	155.86
50	3.91	3.03	164.68
100	4.61	3.66	179.7
200	5.3	4.28	194.49
300	5.71	4.65	203.31
500	6.22	5.11	214.28

Then using Mononobe's equation:

Intensity is obtained as:

$$I_{t_c} = \left(\frac{R}{27}\right) \times \left(\frac{24}{t_c}\right)^{2/3}$$

where, R=179.7 mm

Table 4.1.3 shows computation of rainfall intensity for various return period of varying time of concentration. The rainfall intensity for 100-year return period of time of concentration 0.9493 is found to be 64.5 mm/hr.

Table 4.1.3 Intensity at each time period

Tc(hr)	I(10)	I(20)	I(33)	I(50)	I(100)	I(200)	I(300)	I(500)
0.25	113.03	126.57	136.16	143.86	156.98	169.90	177.61	187.19

1	44.86	50.23	54.03	57.09	62.30	67.43	70.48	74.29
2	28.26	31.64	34.04	35.97	39.25	42.48	44.40	46.80
3	21.57	24.15	25.98	27.45	29.95	32.42	33.89	35.71
4	17.80	19.93	21.44	22.66	24.72	26.76	27.97	29.48
5	15.34	17.18	18.48	19.52	21.31	23.06	24.11	25.41
6	13.59	15.21	16.36	17.29	18.87	20.42	21.35	22.50
7	12.26	13.73	14.77	15.60	17.02	18.43	19.26	20.30
8	11.21	12.56	13.51	14.27	15.57	16.86	17.62	18.57
9	10.37	11.61	12.49	13.19	14.40	15.58	16.29	17.17
10	9.66	10.82	11.64	12.30	13.42	14.53	15.19	16.00
11	9.07	10.16	10.92	11.54	12.60	13.63	14.25	15.02
12	8.56	9.58	10.31	10.89	11.89	12.86	13.45	14.17
13	8.11	9.09	9.77	10.33	11.27	12.20	12.75	13.44
14	7.72	8.65	9.30	9.83	10.72	11.61	12.13	12.79
15	7.38	8.26	8.88	9.39	10.24	11.09	11.59	12.21
16	7.06	7.91	8.51	8.99	9.81	10.62	11.10	11.70
17	6.78	7.60	8.17	8.64	9.42	10.20	10.66	11.24
18	6.53	7.31	7.87	8.31	9.07	9.82	10.26	10.82
19	6.30	7.05	7.59	8.02	8.75	9.47	9.90	10.43
20	6.09	6.82	7.33	7.75	8.46	9.15	9.57	10.08
21	5.89	6.60	7.10	7.50	8.18	8.86	9.26	9.76
22	5.71	6.40	6.88	7.27	7.93	8.59	8.98	9.46
23	5.55	6.21	6.68	7.06	7.70	8.34	8.72	9.19
24	5.39	6.04	6.49	6.86	7.49	8.10	8.47	8.93
0.9493	46.44	52.00	55.94	59.11	64.50	69.80	72.97	76.91

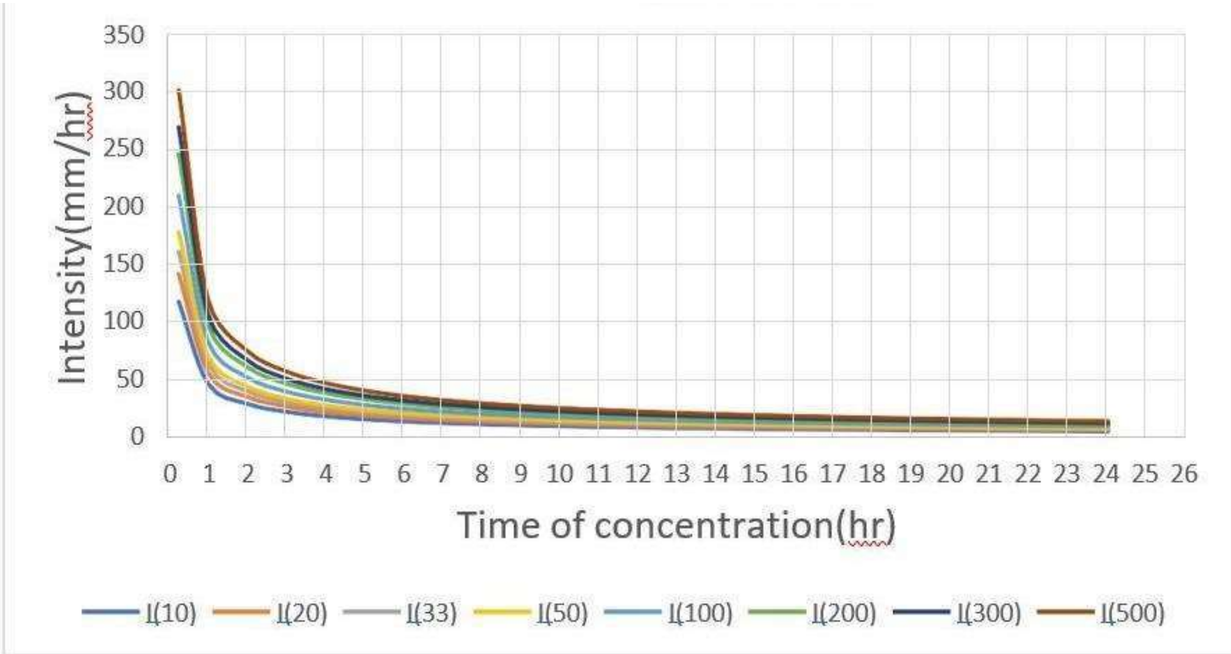


Figure 4.1.3 Intensity Duration Frequency Curve

The rainfall intensity for various time period can also be taken from the Intensity Duration Frequency Curve as shown in Figure 4.1.3.

Then, discharge is computed as:

Sample calculation:

Discharge $(Q) = \frac{CiA}{360} = \frac{0.3287*48.596*2176}{360} = 96.551$ m^3/s

The tables below show intensity and discharge at various return periods.

Table Flood estimation using mononobe's intensity

Table 4.1.4 Flood at each time period

Return Period	I_1 (Dhulikhel) (mm/hr)	I_2 (Nagarkot) (mm/hr)	I_3 (Nangkhel) (mm/hr)	Weighted intensity (I)	Q(m^3/s)
10	53.488	46.43917318	50.315	48.596	96.551
20	62.411	52.00225521	58.298	55.301	109.873
33	68.727	55.93948166	63.929	60.044	119.295
50	73.806	59.10505479	68.623	63.890	126.937
100	82.452	64.49586073	76.254	70.366	139.805
200	90.965	69.80411772	83.884	76.767	152.522
300	96.044	72.96969085	88.460	80.590	160.116
500	102.361	76.9069173	94.095	85.333	169.540

The Table 4.1.4 shows the calculation of flood at various return periods. Hence 100-year flood was found to be **139.805m^3/s.**

2. Chandeshwori River

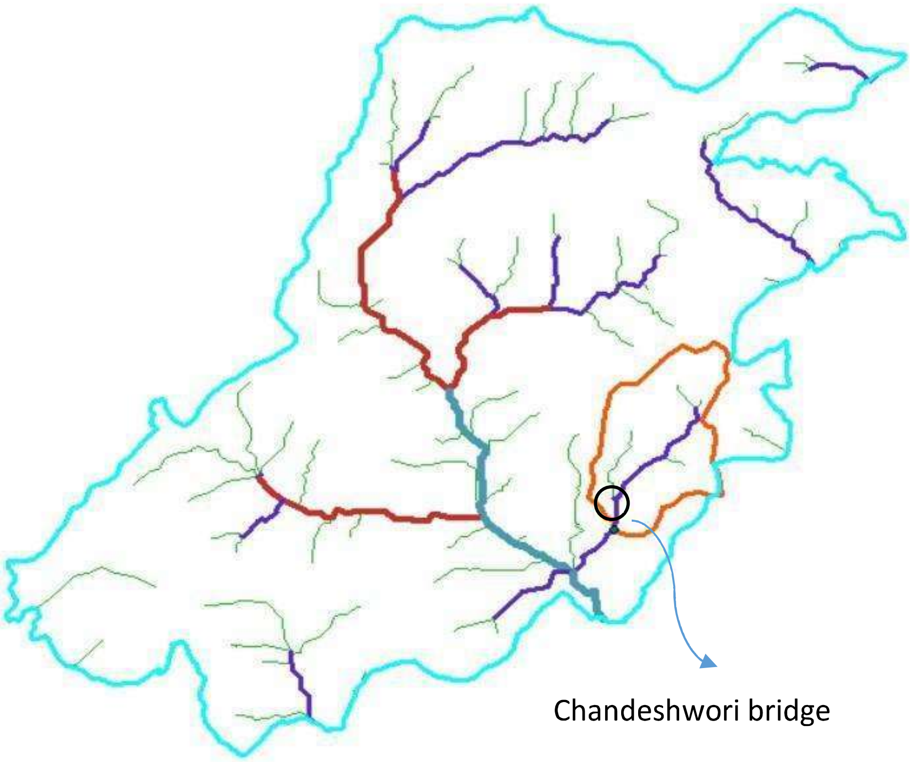


Figure 4.1.4 Catchment area of Chandeshwori river with outlet at Chandeshwori bridge

The delineation of catchment area using ArcGIS at Chandeshwori bridge is shown in Figure 4.1.4. The data of various parameters obtained are listed below.

Catchment area = 2.7515 km^2

Longest stream length = 2556.7 m

Head difference = H2-H1 = 1841 – 1455 = 386 m

Slope(S) = $\frac{H_2-H_1}{Lc} = \frac{386}{2556.7} = 0.1509$

tc=$0.01947L^{0.77}S^{-0.385}$

Time of concentration = 16.96 min

The calculation of weighted coefficient of runoff for various land use is given in table below:

Table 4.1.5 Coefficient of Runoff

	Area (km^2)	% Area(A)	Runoff coefficient (C)	AxC/100
Forest	0.927	33.69	0.2	0.0674
Agricultural	1.532	55.67	0.35	0.1948
Residential	0.292	10.61	0.9	0.0955
			Total	**0.3577**

The weighted runoff coefficient of the Chandeshwori catchment area is found to be 0.3577.

Thiessen Polygon of rainfall station

The Figure 4.1.5 shows that the total runoff in the Pulbazar catchment is due to the contribution of station 1024 only.

The Thiessen Polygon area contributed by these stations is:

Station 1024= 2.7515 km^2

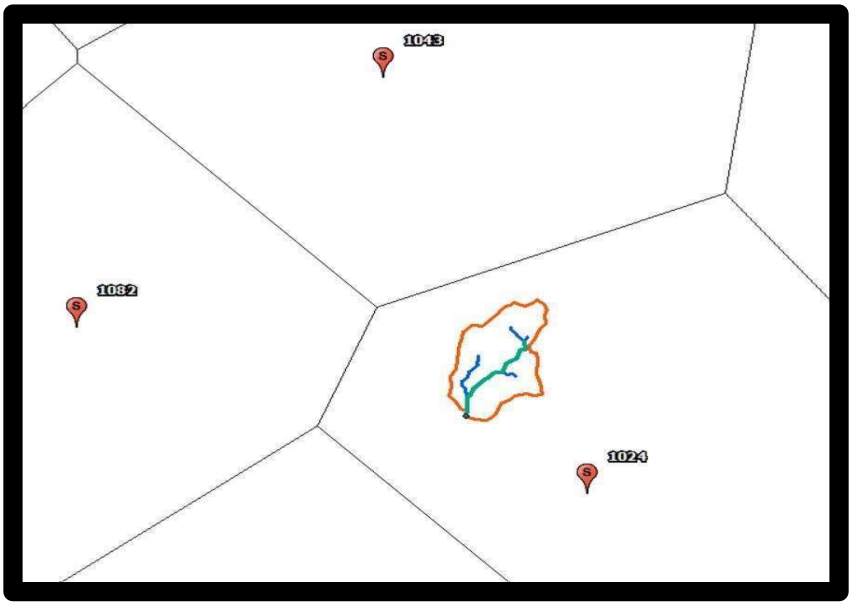


Figure 4.1.5 Thiessen polygon of watershed at Chandeshwori

Flood Estimation:

Rational Method

Area = 2.7515 km2 = 275.15 ha

Length of Concentration (Lc) = 2556.7 m

Head(H) = $H_2 - H_1$ = 1841 – 1455 = 386 m

Slope(S) = $\frac{H_2 - H_1}{Lc} = \frac{386}{2556.7}$ = 0.1509

Coefficient of runoff (C)= **0.3577**

tc = $0.01947L^{0.77}\, S^{-0.385}$

t= tc = 16.96 min

Sample calculation:

Discharge $(Q) = \frac{CiA}{360} = \frac{0.3577*119.96*275.15}{360}$ = 32.79 m^3/s

The Table 4.1.6 show intensity and discharge of flood at various return periods. Hence 100-year flood was found to be **50.55m^3/s.**

Table 4.1.6 Flood estimation using mononobe's intensity

Return Period	Intensity(I) (mm/hr)	Q(m^3/s)
10	119.96	32.79
20	139.97	38.27
33	154.13	42.14
50	165.52	45.25
100	184.91	50.55
200	204.00	55.77
300	215.39	58.89
500	229.56	62.76

Chardobato- Outlet

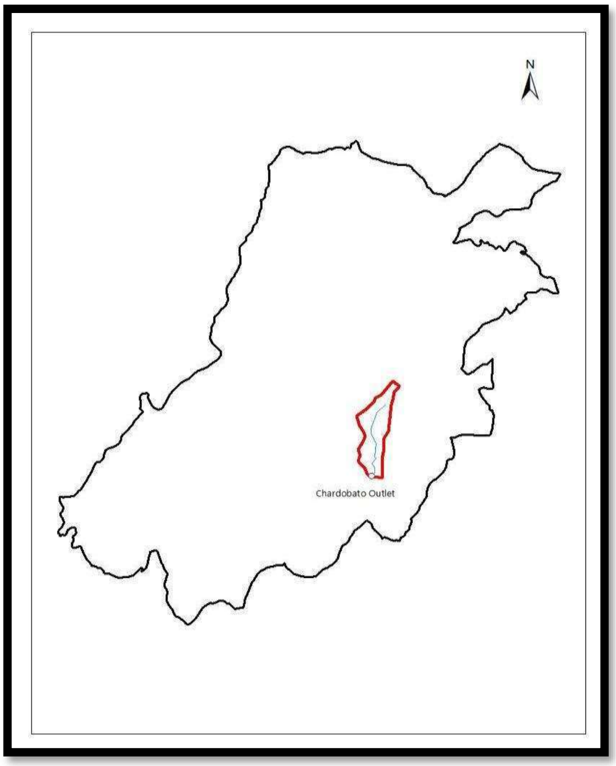


Figure 4.1.6 Catchment area of stream at Chardobato outlet

The delineation of catchment area using ArcGIS at Chardobato is shown in Figure 4.1.6. The data of various parameters obtained are listed below.

Catchment area = 0.8828 km^2 = 88.28 ha

Longest stream length = 1760.8 m

Head difference = H2-H1 = 1729 – 1460 = 269 m

Slope(S) = $\frac{H_2 - H_1}{Lc} = \frac{269}{1760.8} = 0.1528$

tc=0.01947$L^{0.77}$ $S^{-0.385}$

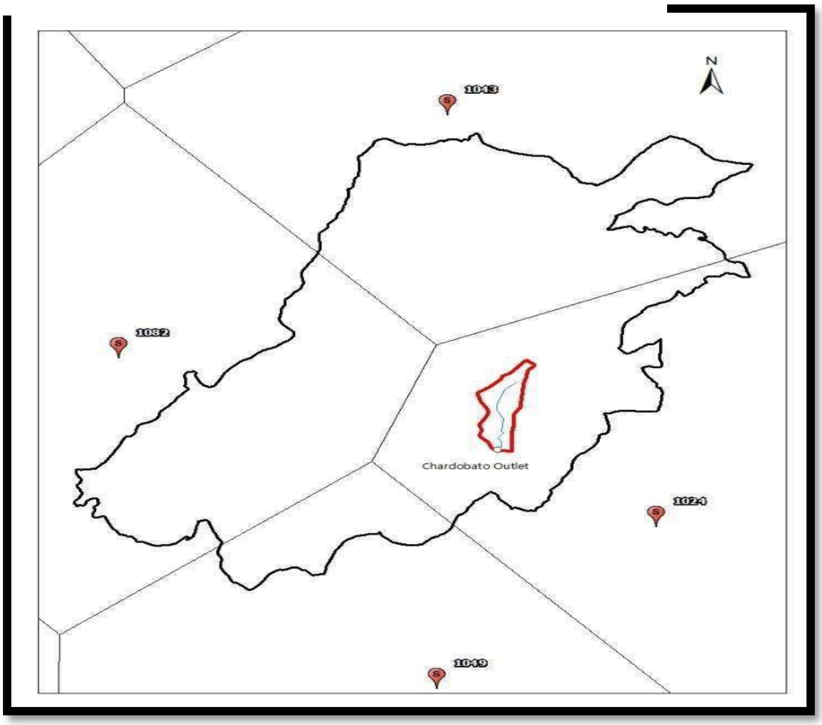
Time of concentration = 12.66 min

Coefficient of runoff:

The calculation of weighted coefficient of runoff for various land use is given in Table 4.1.7

Table 4.1.7 Coefficient of Runoff using Monolobe's intensity

Figure 4.1.7 Thiessen Polygon of Chardobato catchment



	Area(km2)	% Area(A)	Runoff coefficient (C)	AxC/100
Forest	0.315	35.69	0.2	0.0714
Agricultural	0.3766	42.67	0.35	0.1493
Residential	0.1912	21.63	0.9	0.1947
			Total	**0.4154**

The weighted runoff coefficient of Chardobato catchment is found to be 0.4154.

Thiessen Polygon of rainfall station

The Figure 4.1.7 shows that the total runoff in the Pulbazar catchment is due to the contribution of station 1024 only.

The Thiessen Polygon area contributed by this station is:

Station 1024= 0.8828 km^2

Flood Estimation

Rational Method:

Area = 0.8828 km^2= 88.28 ha

Length of Concentration (Lc) = 1760.8 m

Head(H) = $H_2 - H_1$ = 1729-1460 = 269 m

Slope(S) = $\frac{H_2 - H_1}{Lc} = \frac{269}{1760.8}$ = 0.1528

Coefficient of runoff (C)= **0.4154**

tc=0.01947$L^{0.77}$ $S^{-0.385}$

tc= 12.66 min

Sample calculation:

Discharge $(Q) = \frac{CiA}{360} = \frac{0.4154*145.75*275.15}{360}$ = 14.85 m^3/s

The Table 4.1.8 show intensity and discharge at various return period.

Table 4.1.8 Flood estimation using mononobe's intensity

Return Period	Intensity(I) (mm/hr)	Q(m^3/s)
10	145.775	14.85
20	170.091	17.33
33	187.307	19.08
50	201.148	20.49
100	224.712	22.89
200	247.914	25.25
300	261.754	26.66
500	278.970	28.42

Hence 100-year flood was found to be **22.89m^3/s.**

Design flow estimation in side drain

The flow in the side drain can be estimated using following procedure.

Sample calculation:

Approach: Kathmandu to Dhulikhel

Lane = left

Catchment area(A) = 18.1224 ha

Runoff coefficient (c) = 0.3

Length of concentration (L) = 1340 m

time of concentration (tc)= 14.2 min

Table 4.1.9Design Flood Estimation in Side Drain

Return Period(T)	Intensity for tc = 14.2 min (mm/hr)	Discharge(m^3/s)
10	134.98	2.039
20	157.501	2.379
33	173.442	2.619
50	186.258	2.813
100	208.078	3.142
200	229.562	3.467
300	242.379	3.66.
500	258.32	3.901

The Table 4.1.9 shows calculation of intensity and discharge at various return periods. The 100-year intensity and flow is calculated as 208.078 mm/hr and 3.142 m^3/s.

The remaining calculation of design flood inside drain is shown in Appendix B.

Estimation of cross section of bridge at various locations

1. **Punyamati River**

Flood (Q)=139.805 m^3/s.

Bridge width (b)= 25 m

Manning's coefficient(n)= 0.023

Slope(S)=0.03%

Using Manning's equation:

$$Q = \frac{A}{n} R^{2/3} S^{1/2}$$

139.805 = (1/0.023) * ((25 * y) ^ 5/3) * (1 / (25+2 *y)) * (0.0003 ^ ½)

Flow depth (y) =3.67 m

Consider freeboard of 1m,

Depth of bridge from bed level of river = 3.67+1 = 4.67m

2. **Chandeshwori River**

Flood (Q)=50.55 m^3/s.

Bridge width (b)= 5 m

Manning's coefficient(n)= 0.023

Slope(S)=1%

Using Manning's equation:

$$Q = \frac{A}{n} R^{2/3} S^{1/2}$$

50.55 = (1/0.023) * ((5 * y) ^ 5/3) * (1 / (5+2 *y)) * (0.01 ^ ½)

Flow depth (y) =2.12 m

Consider freeboard of 1m,

Depth of bridge from bed level of river = 2.12+1 = 3.12 m

3. **Chardobato outlet**

Flood (Q)=22.89 m^3/s.

Bridge width (b)= 5 m

Manning's coefficient(n)= 0.023

Slope(S)=0.05%

Using Manning's equation:

$$Q = \frac{A}{n} R^{2/3} S^{1/2}$$

22.89 = (1/0.023) * ((5 * y) ^ 5/3) * (1 / (5+2 *y)) * (0.0005 ^ ½)

Flow depth (y) =3.62 m

Consider freeboard of 1m,

Depth of bridge from bed level of river = 3.62+1 = 4.62 m

Estimation of cross section of side drain

The rainfall data of station Dhulikhel (1024) was used for analysis as the other nearby rainfall station did not introduce any changes in the Thiessen polygon.

Table 4.1.10 Cross section of Side Drain

Chainage	Lane	Discharge (Q) m^3/s	Drain Section	Bottom width (b)	Depth of flow (d)	Bed slope (S)	Manning's coefficient (n)	Area (A) (m^2)	Discharge capacity (Qc)	Velocity (v) (m/s)	Velocity range (m/s)
0+000 to 0+900	Left Service Lane	2.813	D1	0.9	0.9	2%	0.017	0.81	3.019	3.47	2.5-5
0+900 to 1+100	Left Service Lane	1.542	D2	0.7	0.7	3%	0.017	0.49	1.727	3.14	2.5-5
1+100 to 1+190	Left Service Lane	0.129	D3	0.4	0.3	2%	0.017	0.12	0.242	1.07	2.5-5
1+190 to 1+300	Left Service Lane	0.921	D4	0.7	0.5	2.5%	0.017	0.35	1.135	2.6	2.5-5
1+300 to 1+540	Left Service Lane	1.74	D5	0.8	0.7	2.5%	0.017	0.56	2.09	3.11	2.5-5
0+000 to 0+900	Right Service Lane	0.772	D6	0.5	0.6	2%	0.017	0.3	0.785	2.57	2.5-5
0+900 to 1+100	Right Service Lane	0.277	D7	0.4	0.4	2%	0.017	0.16	0.347	1.73	2.5-5

Chainage	Lane	Discharge (Q) m^3/s	Drain Section	Bottom width (b)	Depth of flow (d)	Bed slope (S)	Manning's coefficient (n)	Area (A) (m^2)	Discharge capacity (Qc)	Velocity (v) (m/s)	Velocity range (m/s)
1+100 to 1+190	Right Service Lane	0.192	D8	0.4	0.3	2%	0.017	0.12	0.243	1.59	2.5-5
1+190 to 1+300	Right Service Lane	0.546	D9	0.4	0.6	2.5%	0.017	0.24	0.63	2.27	2.5-5
1+300 to 1+540	Right Service Lane	0.804	D10	0.6	0.5	2.5%	0.017	0.3	0.914	2.67	2.5-5
0+000 to 0+900	Freeway both lane	0.53	D11	0.6	0.5	1%	0.017	0.3	0.578	1.92	2.5-5
0+900 to 1+100	Freeway both lane	0.071	D12	0.4	0.2	2%	0.017	0.08	0.143	1.79	2.5-5
1+100 to 1+300	Freeway both lane	0.118	D3	0.4	0.2	3%	0.017	0.08	0.17	2.1	2.5-5
1+300 to 1+540	Freeway both lane	0.142	D14	0.4	0.2	3.5%	0.017	0.08	0.189	2.37	2.5-5

The Table 4.1.10 shows the calculation of the cross section of the side drains at various chainages. In the left service lane, from chainage 0+000 to 0+900, side drain of width 0.9m and depth 0.9 m is taken.

Comparison of flood and flow depth at Punyamati river, Chardobato Outlet and Chandeswari river with the results from DPR

1) Punyamati River

As per DPR:

Flood (Q)=115.87 m^3/s.

Bridge width (b)= 25 m

Manning's coefficient(n)= 0.023

Slope(S)=0.03%

Using Manning's equation:

$$Q = \frac{A}{n} R^{2/3} S^{1/2}$$

Flow Depth y= 3.27m

But the flow depth is calculated as 2.815 m which is incorrect.

- As per NEPAL BRIDGE STANDARDS-2067 a minimum of 1m of freeboard is to beprovided so the total clear height below the bridge would be 3.27+1=4.27m

From Analysis

Flood (Q) =139.805 m^3/s.

Keeping all other factors constant,

Flow depth y= 3.69 m

- Considering freeboard, the required total clear height below the bridge would be 3.69+1=4.69m
- According to the design of the road profile, the height of the bridge from the bottom of the riverbed is about 4.4 m.

So, it is recommended to increase the height of the bridge from the bottom of riverbed to 5 m.

2) Chardobato Outlet

As per DPR,

Q=15.83m^3/s.

Bridge width (b)= 5 m

Manning's coefficient(n)= 0.023

Slope(S)=0.05%

Depth of flow= 1.657 but actual depth should be 2.73 m

Clear depth= 2.73+1=3.73m

From Analysis,

Q=22.89m^3/s.

Keeping other factors constant,

Depth of flow = 3.62 m

Actual Clear depth= 3.62+1=4.62m

Proposed structure = 5*3 box culvert which is insufficient

Recommend (5x5) m box culvert.

3) Chandeswari River

As per DPR,

Q=30.03m^3/s.

Bridge width (b)= 5 m

Manning's coefficient(n)= 0.023

Slope(S)=1%

Depth of flow= 2.369 m but actual depth should be 1.46 m

Clear depth= 1.46+1=2.46m

From Analysis,

Q=50.55m^3/s.

Keeping other factors constant,

Depth of flow= 2.12 m

Actual Clear depth= 2.12+1=3.12m

Proposed structure = 5*5 box culvert which is sufficient.

4.2 Crash STUDIES

Six Basic Steps for Crash Studies

1. Obtain adequate vehicle crash records
2. Select high crash frequency location in order of severity
3. Prepare collision diagram and physical condition for each selected location
4. Summarize the facts
5. Supplement crash data with field observations during which more crashes have been reported
6. Analyze summarized facts and field data and prescribe remedial measures.

1. Vehicle crash records

Vehicle Crash records were obtained from Banepa traffic police.

The detail converted crash record obtained from traffic police is given in Appendix section.

The collision and condition diagram is as follows as obtained from traffic police records- Figure 4.2.1

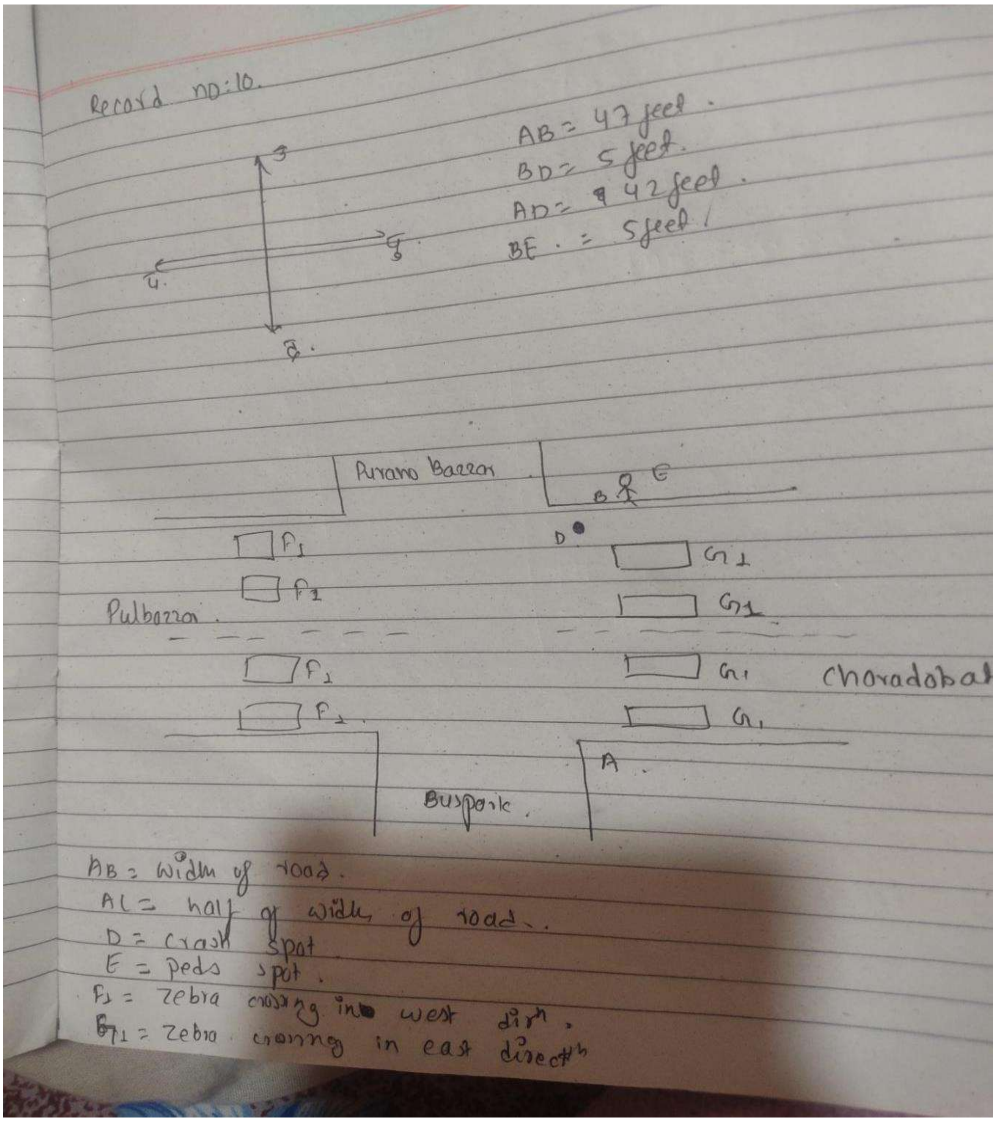


Figure 4.2.1 Samples of collision diagram from traffic police (copied)

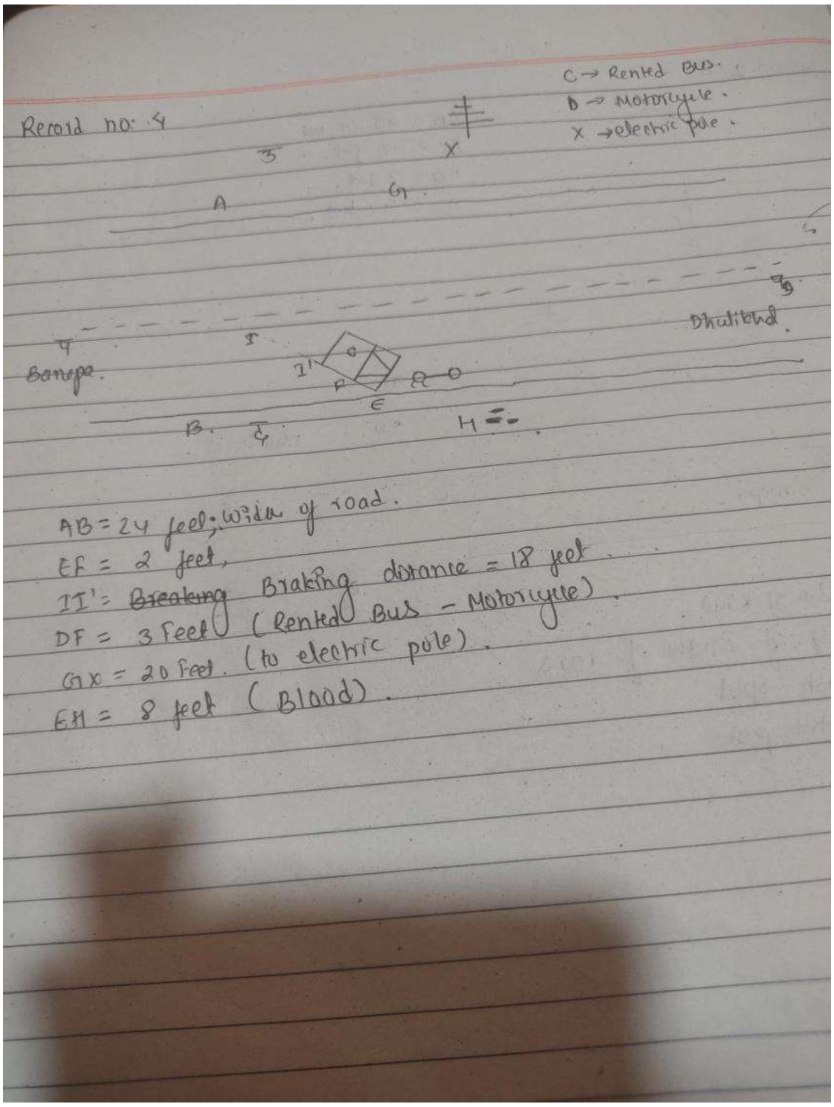


Figure 4.2.2 Samples of collision diagram from traffic police (copied)

2. Collision diagram and High frequency location

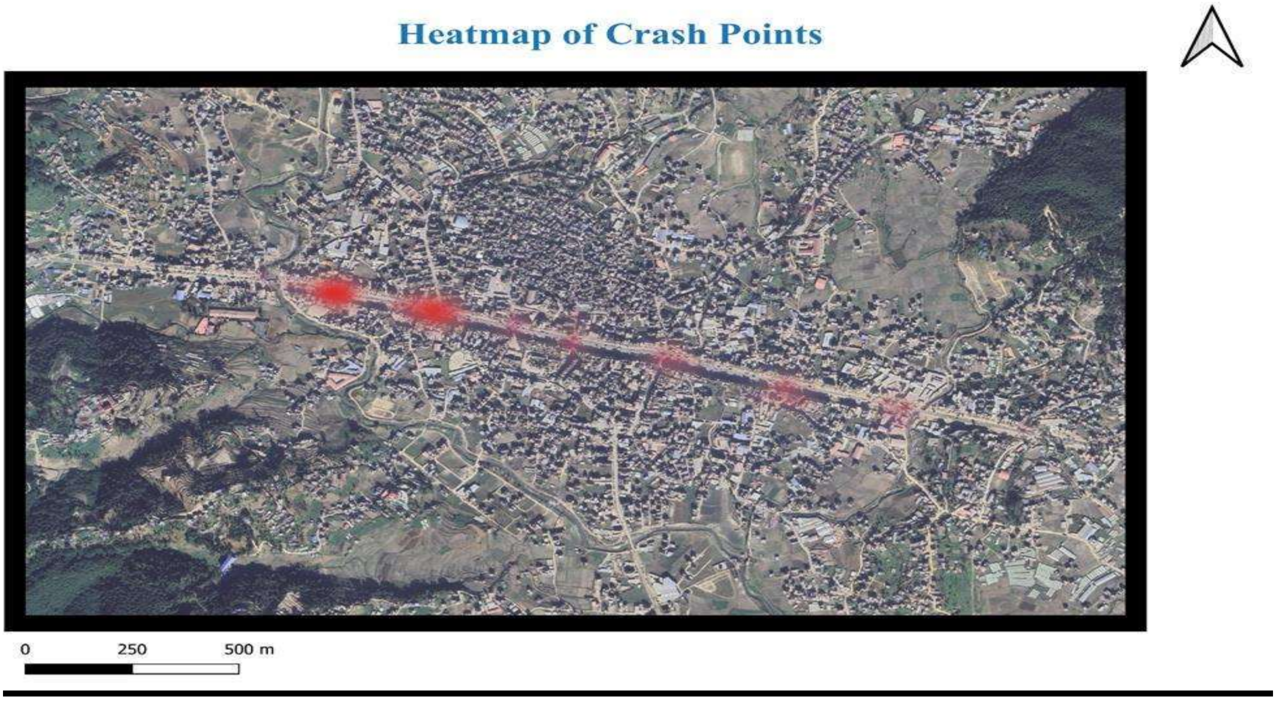
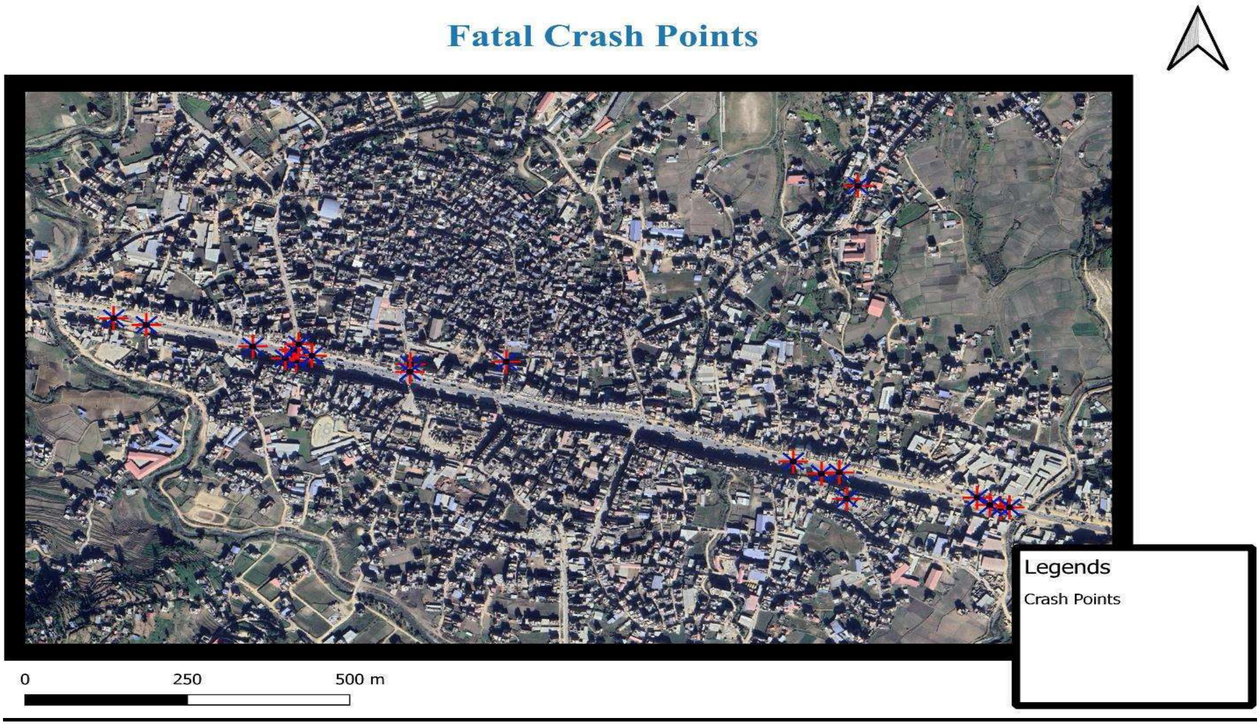


Figure 4.2.3 Heatmap of Crash Points



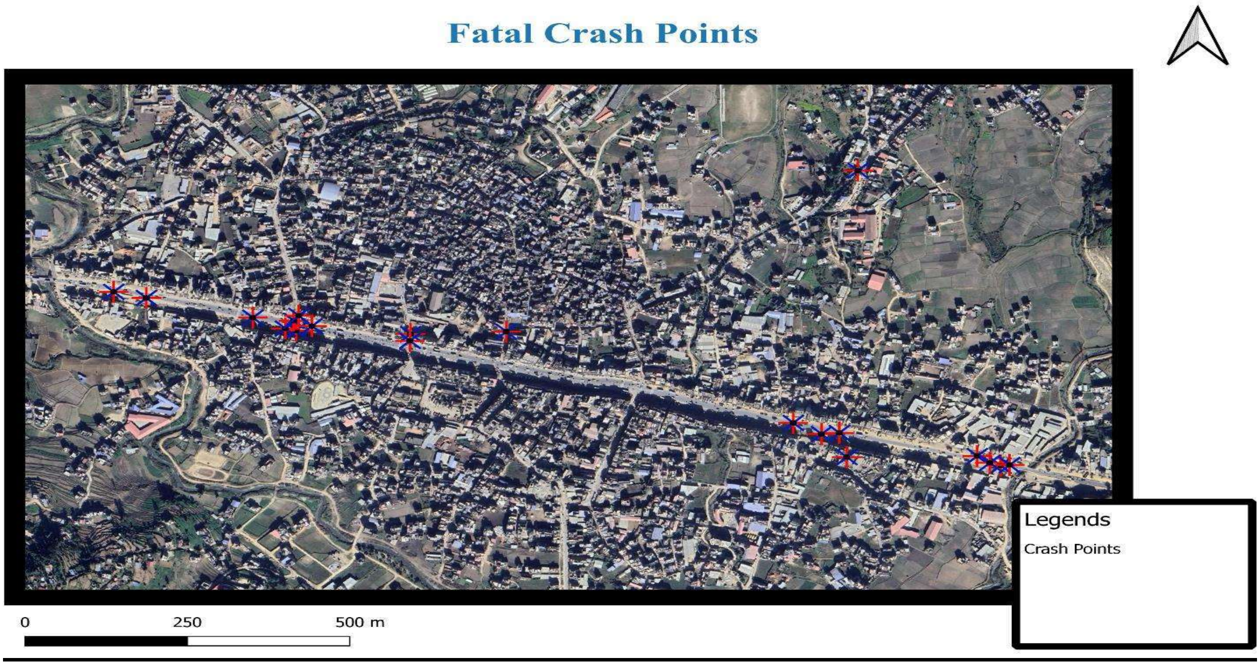
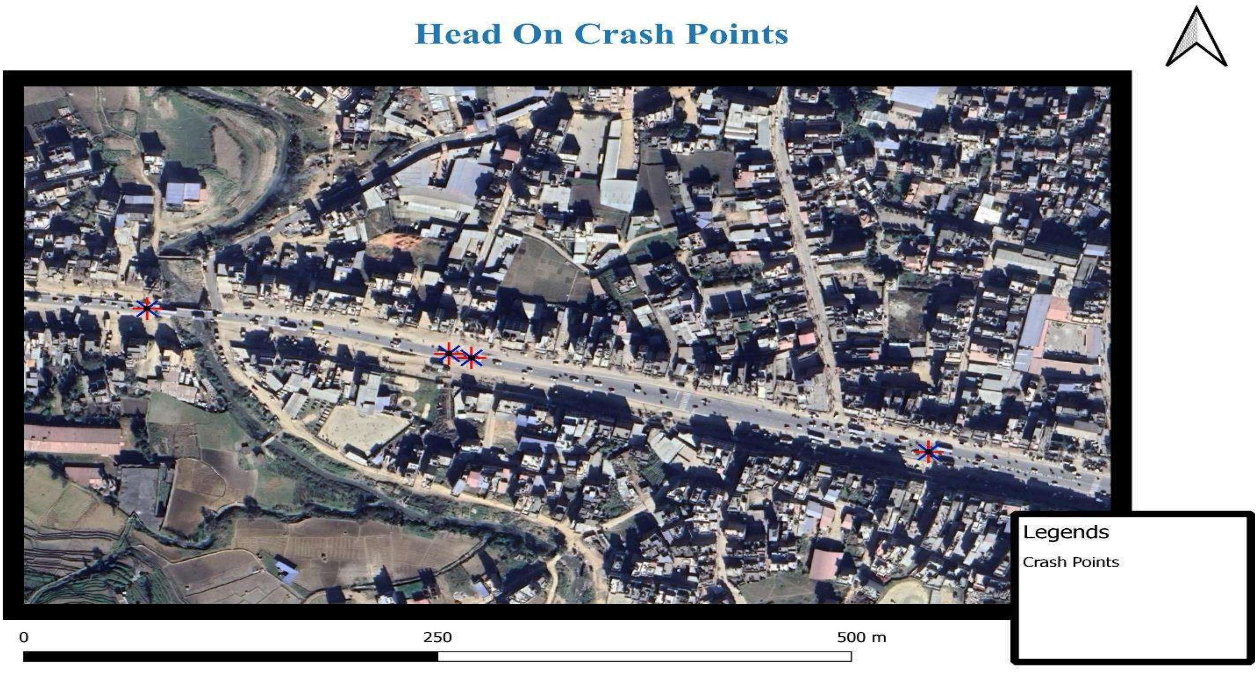
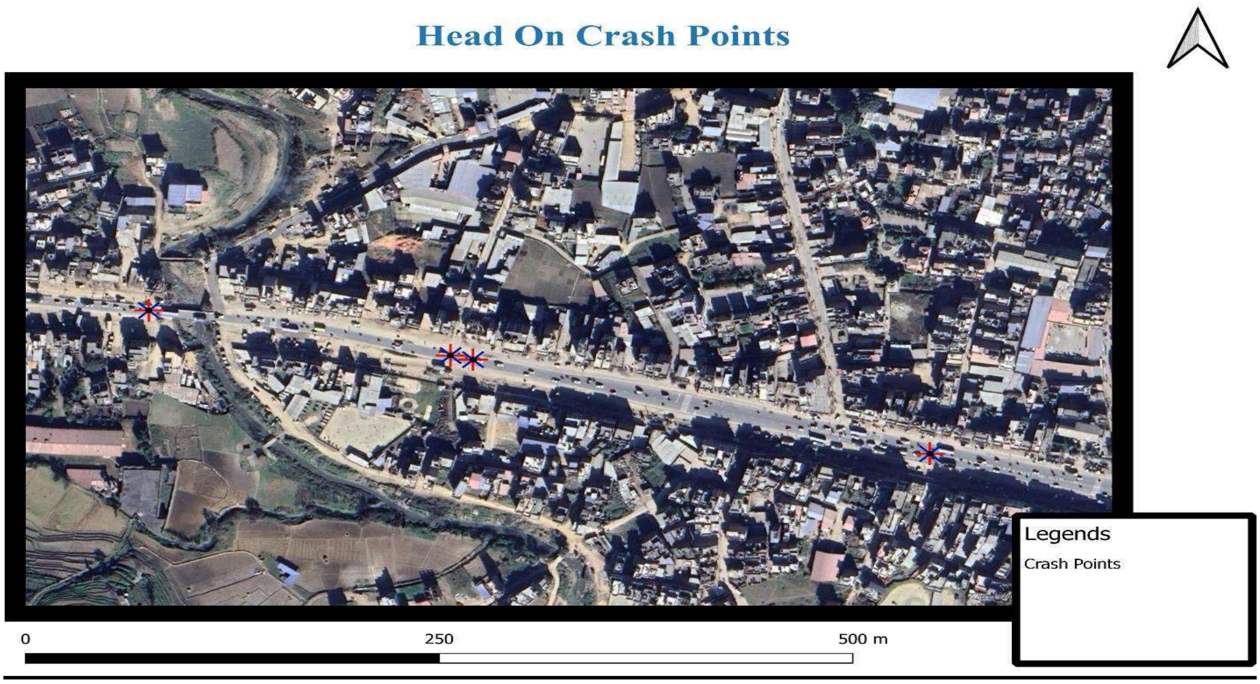


Figure 4.2.4 fatal crash points





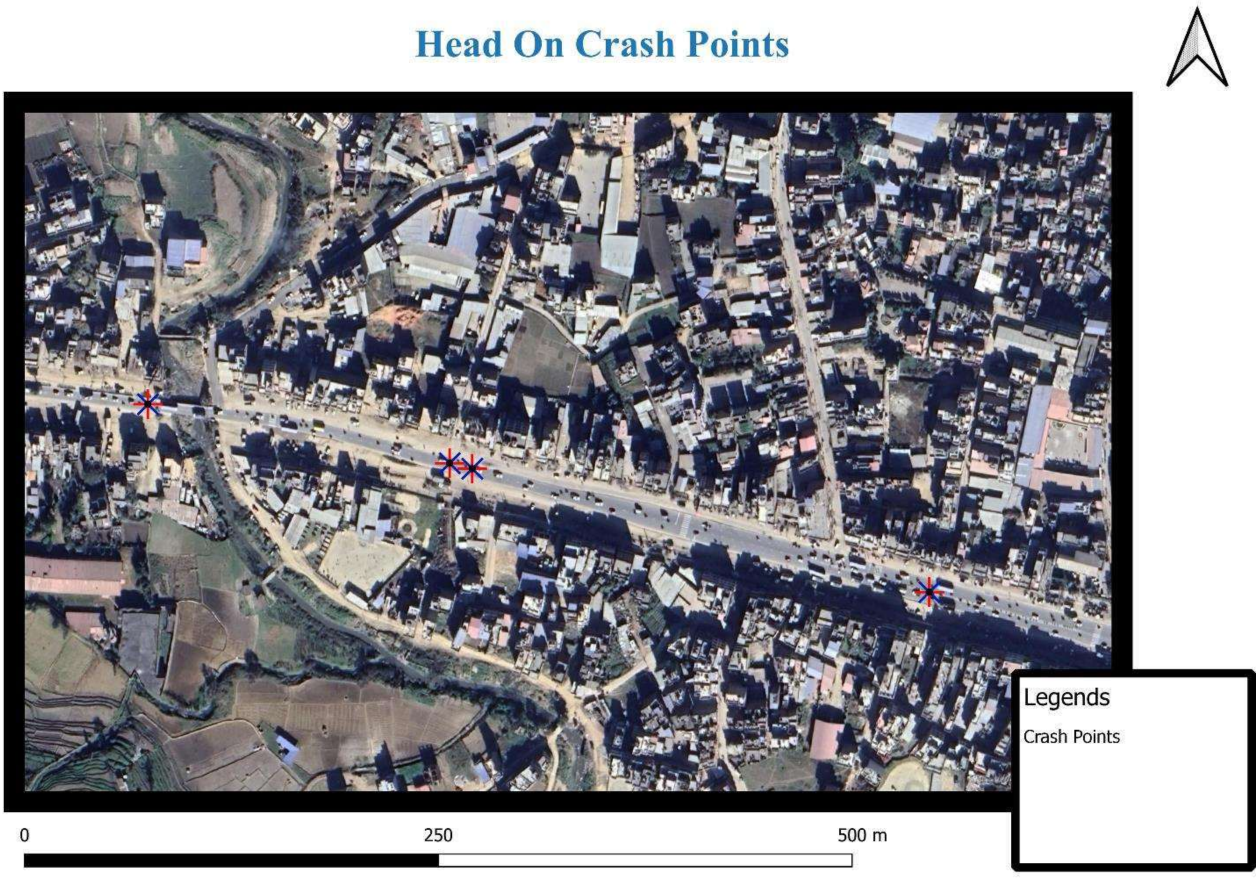


Figure 4.2.5 head on crash point

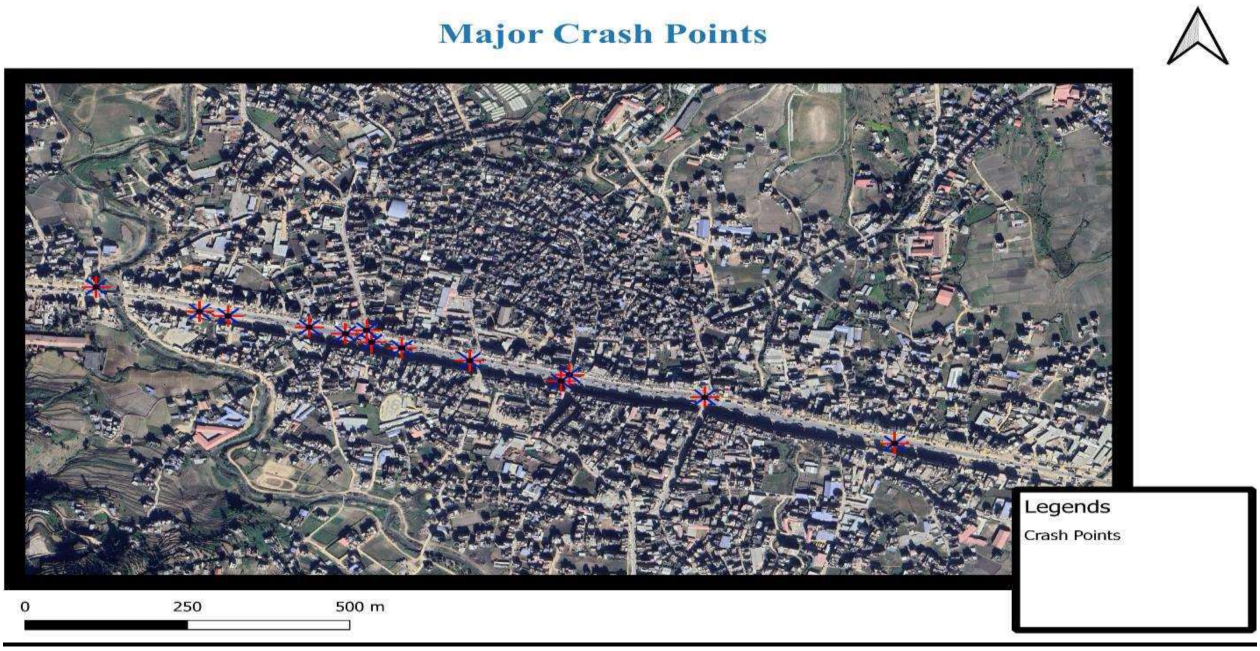


Figure 4.2.6 major crash points

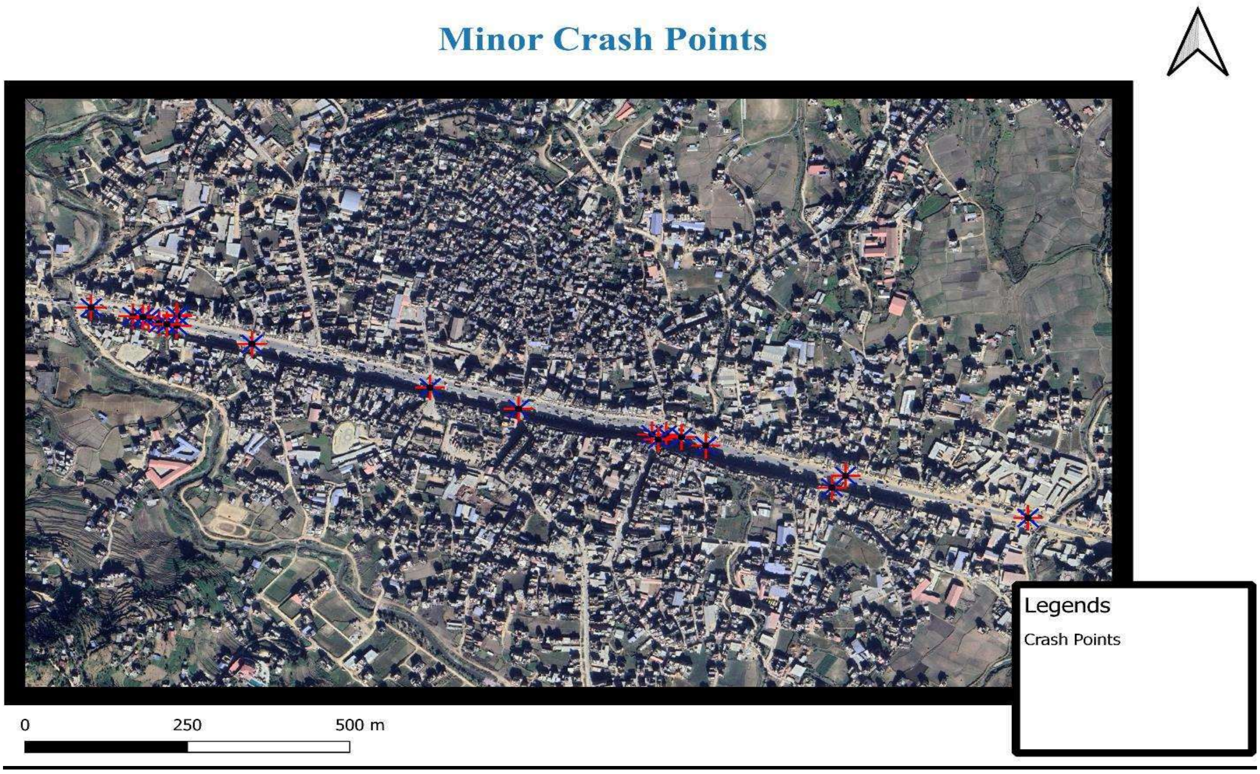


Figure 4.2.7 minor crash points

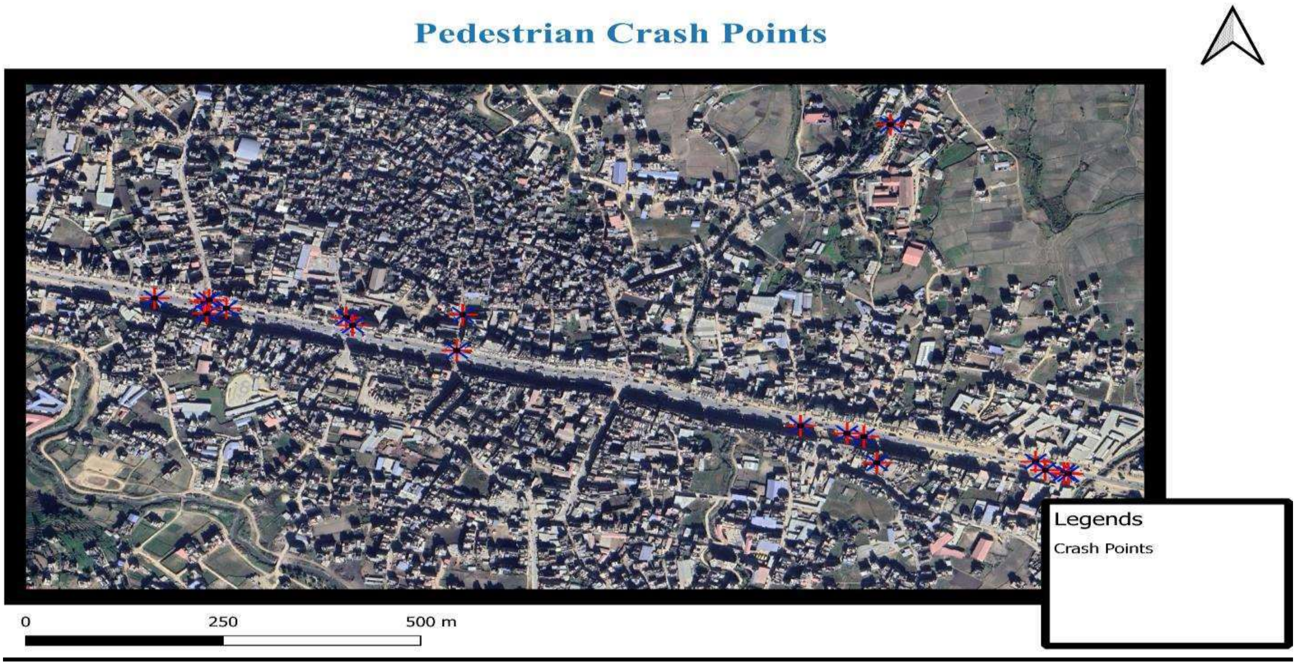


Figure 4.2.8 pedestrian crash points

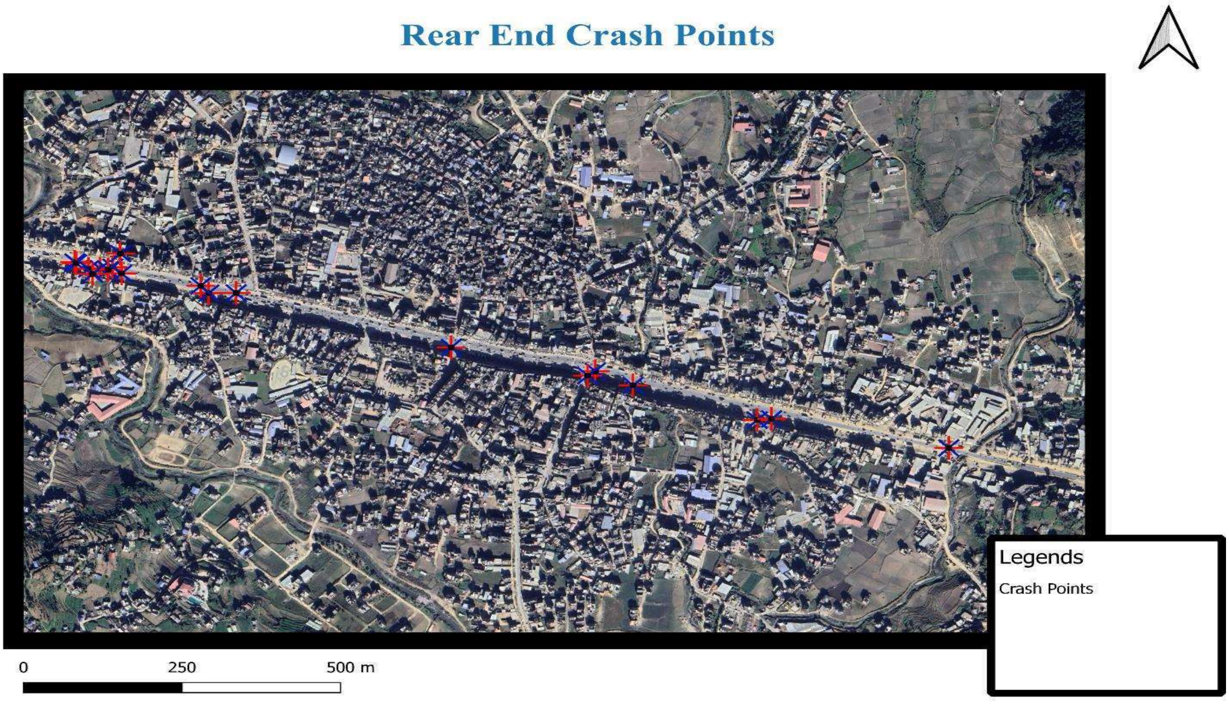


Figure 4.2.9 rear end crash points

Analysis of crash data and graphs

Table 4.2.1 Cause of Crashes

Cause of Crash									
Fiscal year	Driver's fault	Alcohol	overtake	over speed	overload	use of mobile phone	machine fault	road condition	weather	total Crashes
076/077	5	1	0	0	0	1	1	1	1	10
077/78	13	1	1	0	1	1	1	2	2	22
078/079	22	3	0	0	0	3	1	1	1	31

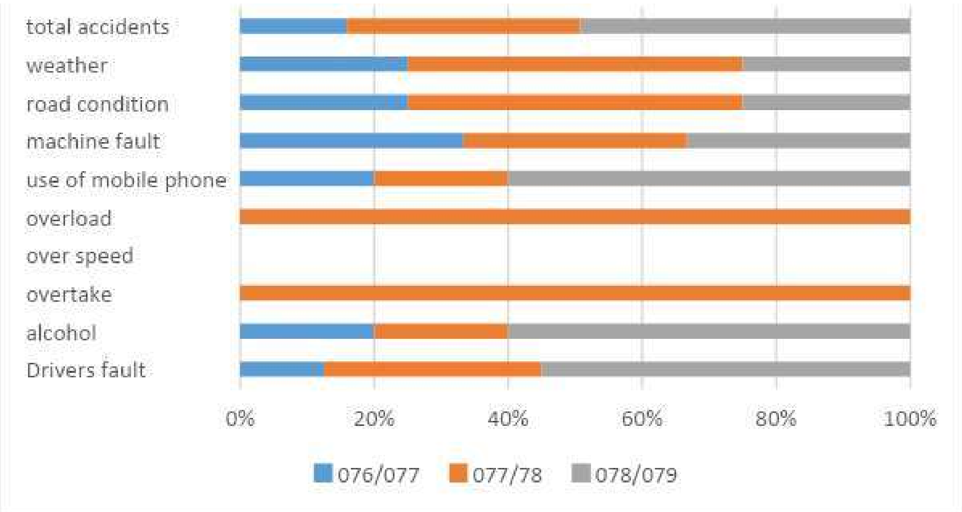


Figure 4.2.10 Graph depicting cause of Crash

Table 4.2.2 Vehicles Involved in Road Crash

Fiscal year	Tanker	truck	tipper	microbus	car/ jeep/ van	tractor	motorcycle	tempo	bus	Cycle	pedestrian	total
076/077	0	2	3	1	7	1	1	1	4	1	0	21
077/078	0	1	9	0	8	0	17	0	4	1	6	46
078/079	0	1	5	1	18	0	14	0	6	0	13	58

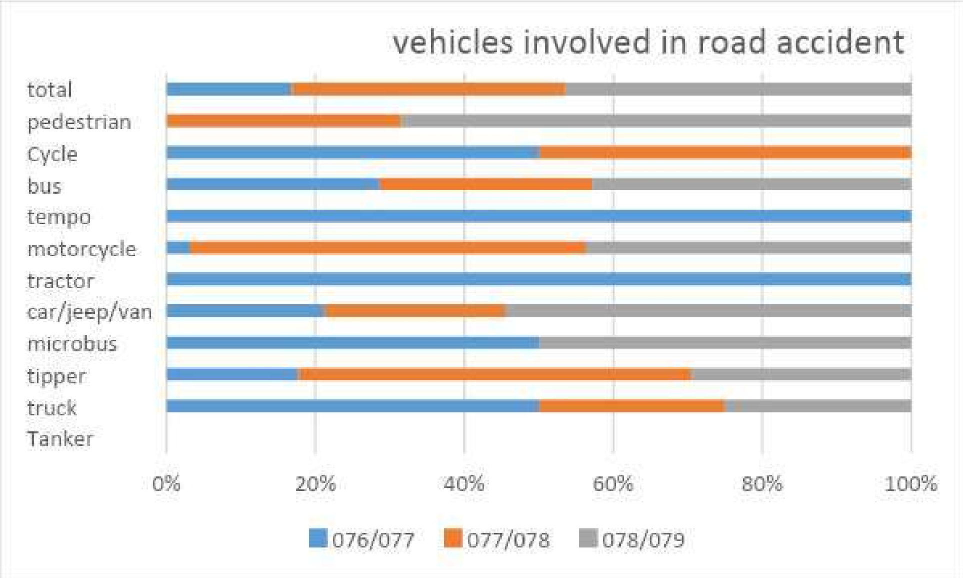


Figure 4.2.11 Graph depicting Vehicle involved in Road Crash

Table 4.2.3 Crash Severity

Fiscal Year	No.of crashes	Death	Major Injury	Minor Injury	Property Damages
076/077	10	1	0	9	0
077/078	22	9	6	6	1
078/079	31	12	8	7	4

Figure 4.2.12 Graph depicting Crash Severity in past 3 years

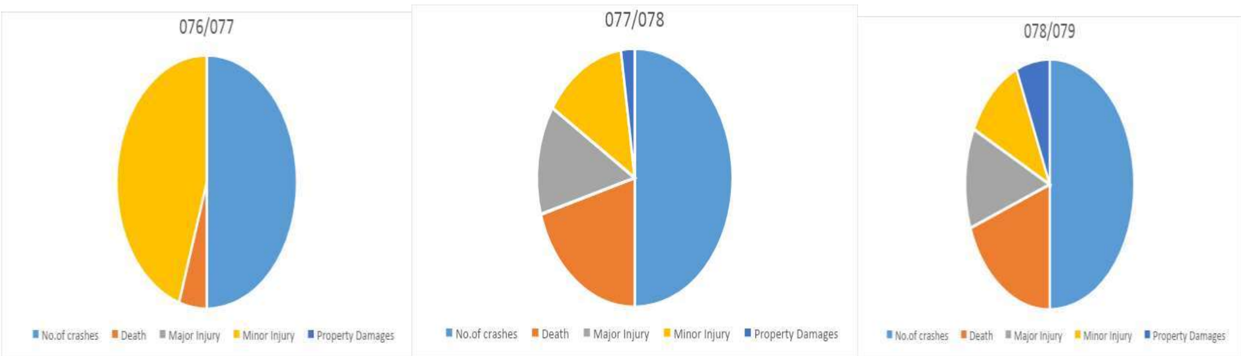


Table 4.2.4 Crash Time

Fiscal year	6:00-12:00	12:00-18:00	18:00-24:00	24:00-06:00
076/077	6	3	0	1
077/078	11	4	5	2
078/079	9	13	5	4

Figure 4.2.13 Graph depicting Crash Time of the day in 3 years

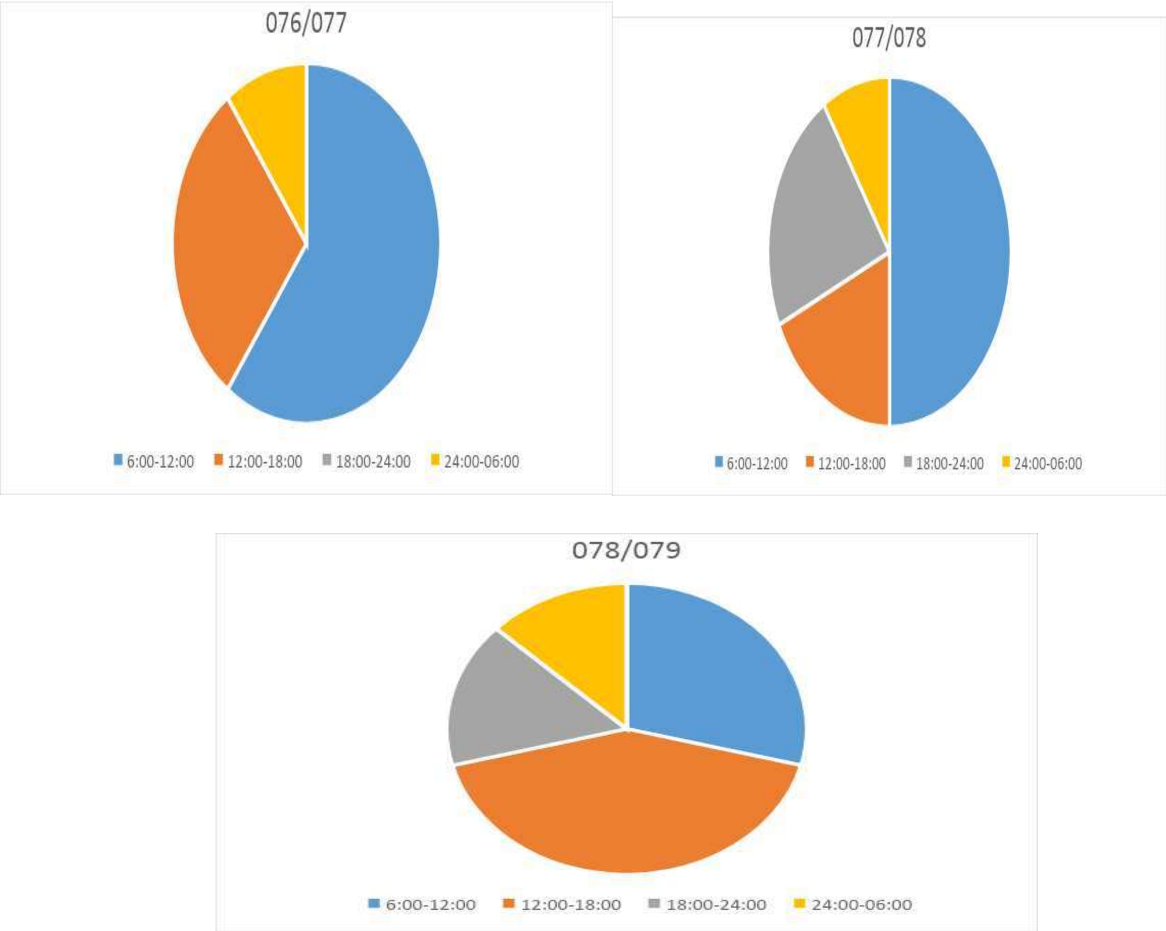
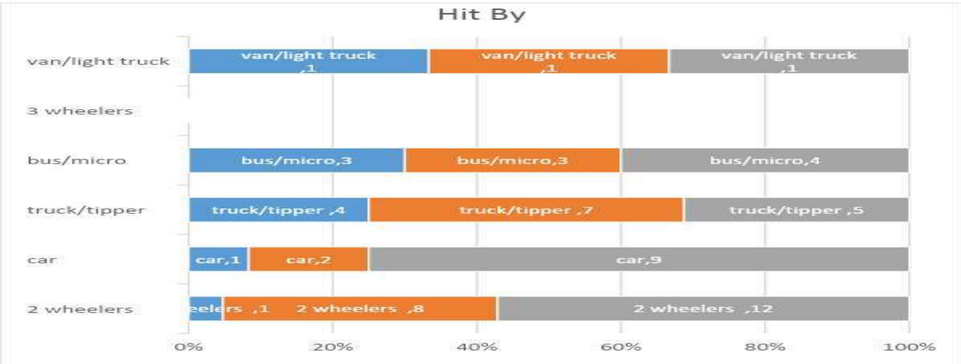


Table 4.2.5 Crash by vehicle Class (Hit By)

Fiscal year	2 wheelers	car	truck/tipper	bus/micro	3 wheelers	van/light truck
076/077	1	1	4	3		1
077/078	8	2	7	3		1
078/079	12	9	5	4		1

Figure 4.2.14 Graph depicting Crash by Vehicle Class



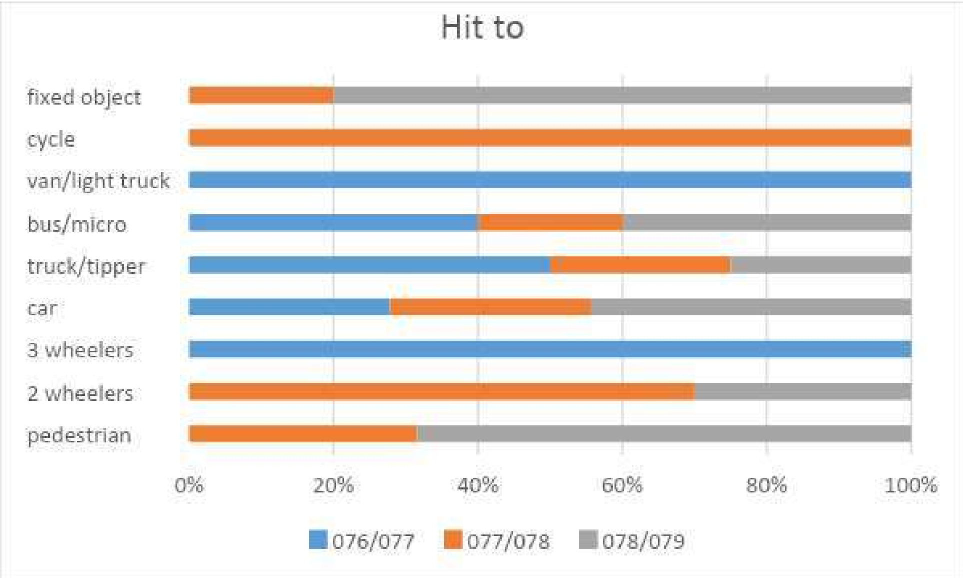


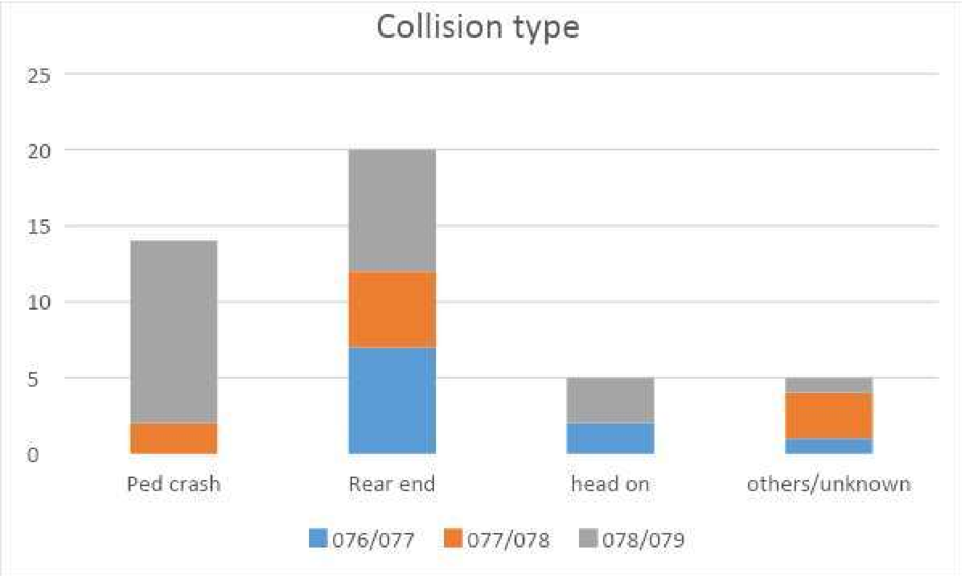
Table 4.2.6 Crash by Vehicle Class (Hit to)

Fiscal year	pedestrian	2 wheelers	3 wheelers	car	truck/tipper	bus/micro	van/light truck	cycle	fixed object
076/077			1	5	2	2	1		
077/078	6	7	0	5	1	1		1	1
078/079	13	3	0	8	1	2			4

Table 4.2.7 Type of Collision

Fiscal year	Ped crash	Rear end	head on	others/unknown
076/077		7	2	1
077/078	2	5		3
078/079	12	8	3	1

Figure 4.2.15 Graph depicting Crash by collision type



4.3 SAFETY AUDIT USING iRAP DEMONSTRATOR

The star rating at various section of the road is shown below:

At chainage 0+500m to 0+600B, Ratings:

Table 4.3.1 Vehicle Star Rating at Chainage 0+500

Vehicle type	Star	Points
Car	4	6.27
Motorcycle	4	7.49
Pedestrian	2	55.57
Bicycle	3	14.17

Figure 4.3.1 Sample of Star Rating of Chainage 0+500 to 0+600B along with Georeferenced image

Changes Introduced

At chainage 0+500-0+600 B,

For Pedestrian:

Table 4.3.2 Pedestrian Star Rating at Chainage 0+500

Properties	Initial Condition	Initial Rating	Initial Star	Final Condition	Final Rating	Final Star
Delineation	Poor	55.57	2	Adequate	55.12	2
Street Lighting	Not Present	55.12	2	Present	44.1	2
Pedestrian crossing facilities - inspected road	Unsignalized marked crossing without refuge	44.1	2	Unsignalized raised marked crossing without refuge	17.49	3

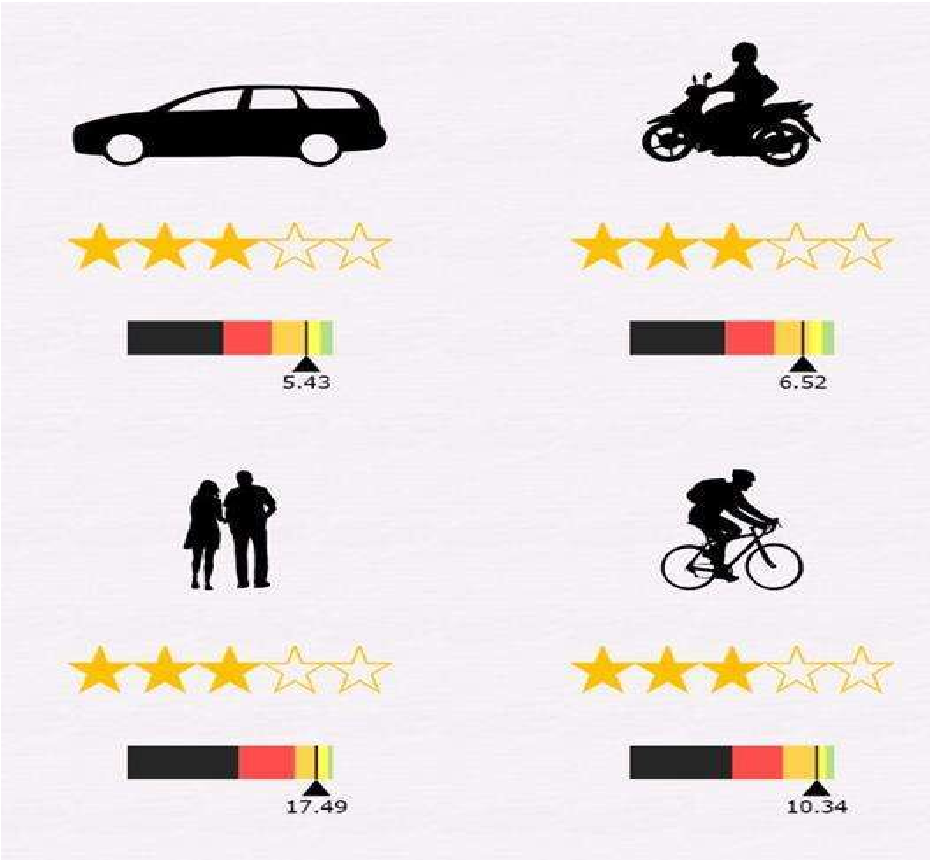


Figure 4.3.2 Star rating of road section 0+500 to 0+600 B after safety changes

Because of restriction on size of the report and to enhance comprehensiveness a sample of analysis is only presented here.

4.4 Detailed intersection design and analysis-EXISTING AND Proposed DESIGN

Data Collection and Analysis

Description of data collection methods and data sources is given as:

1. Intersection Geometry:

   Previous Design Reports for the proposed design and Google Earth Pro for the existing case was used to obtain Intersection Geometry data.

2. Traffic and Pedestrian Volume Survey:

   Individual Smartphones mounted in convenient rooftops using tripods were used for Video graphic survey of both the Sites (Tindobato and Chardobato). The video graphs so obtained were replayed for a number of times in order to obtain **classified count** vehicular and pedestrian volume for each of the movements separately. Tally marks system was employed to note down the observation on the observation sheet.

A total of 5.5hrs (2hrs in the morning peak, 1.5 in the day and 2 in the evening peak) of video was taken with each sub-count period lasting 15 minutes. Cross-checking of a sample of observation of volume count was done during the analysis period to ascertain accuracy and precision.

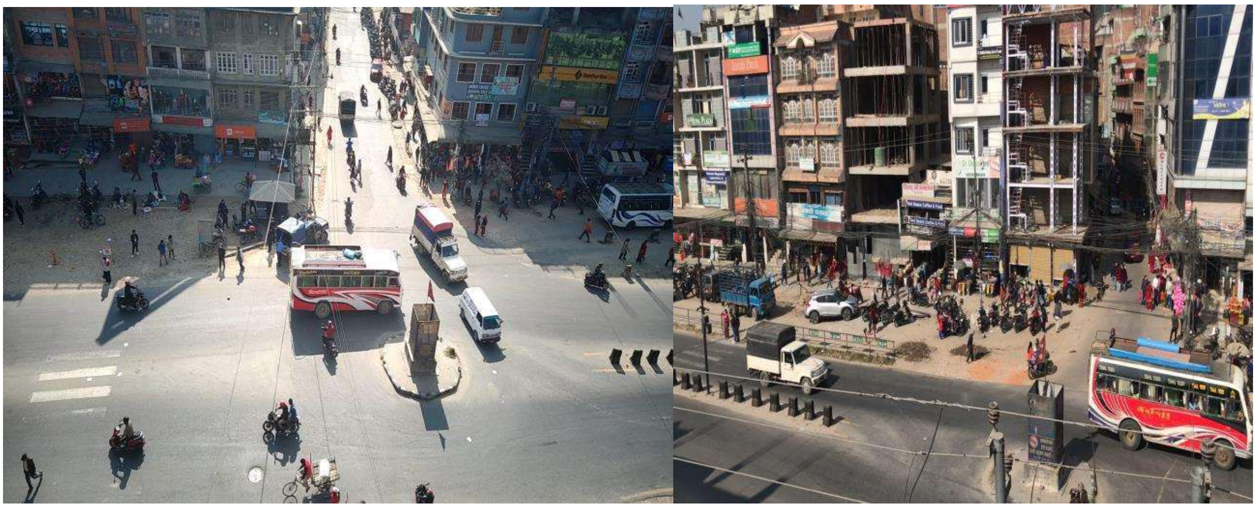


Figure 4.4.1 Video Footage Field View of Chardobato (Both Cameras)

The speed of video playback for the count varied depending on the flow in the approach. For example: Vehicular volume in Tindobato was generally counted at the speed of 2x.

A sample of Blank Observation Sheet is shown as below:

Figure 4.4.2 Sample Observation Sheets (Filled and Unfilled)

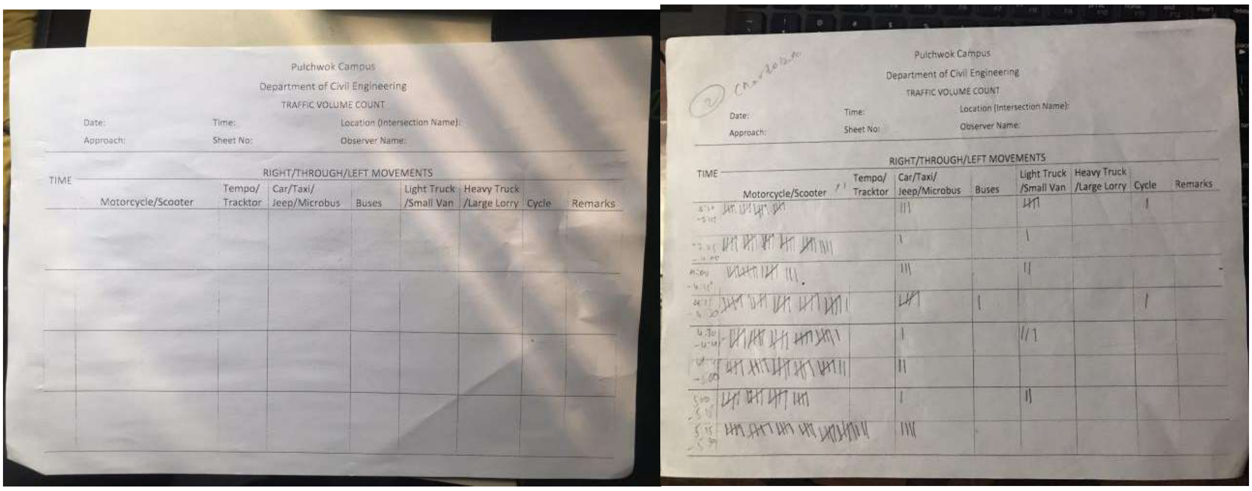


Table 4.4.1 Traffic Volume Count Sample

Pulchowk Campus

Department of Civil Engineering

Traffic Volume Count

Date:-2079/09/07 Location:-Tindobato

Approach- East to West

Time	B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
8:45-9:00	135	0	29	18	18	2	0
9:00-9:15	153	1	36	16	23	0	0
9:15-9:30	120	0	42	14	18	3	2
9:30-9:45	145	0	43	11	13	5	0
9:45-10:00	155	1	53	20	20	3	3
10:00-10:15	164	3	42	15	17	4	1
10:15-10:30	0	0	0	0	0	0	0
10:30-10:45	0	0	0	0	0	0	0

Here, B/S refers to Motorcycles and Scooters

tem/tract refers to Tempo and Tractors

T/J/M refers to Car, Taxi, Jeep and Microbuses

LT refers to Light Trucks

HV refers to Heavy Vehicles

The Sample Excel Sheet for the observations is as follows: (See Appendix C (Tindobato) and D (Chardobato) for entire observation in Tabulated form)

APPENDIX D- TRAFFIC VOLUME COUNT OF TINDOBATO (SAMPLE)

VEHICLE COUNT

Approach- East to West

Time	B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
8:45-9:00	135	0	29	18	18	2	0
9:00-9:15	153	1	36	16	23	0	0
9:15-9:30	120	0	42	14	18	3	2
9:30-9:45	145	0	43	11	13	5	0
9:45-10:00	155	1	53	20	20	3	3
10:00-10:15	164	3	42	15	17	4	1
10:15-10:30	0	0	0	0	0	0	0
10:30-10:45	0	0	0	0	0	0	0

Hourly Volume for Each Vehicle Class

B/S	tem/tract	T/J/M	Buses	L/T/Van	HV	Cycle
553	1	150	59	72	10	2
573	2	174	61	74	11	5
584	4	180	60	68	15	6
464	4	138	46	50	12	4
319	4	95	35	37	7	4

Here, 584 two-wheeler count is the summation of four consecutive 15 min count from 9:15 to 10:15 am

V15 is the maximum 15 min count within the Peak Hourly Traffic (164 units for two-wheelers in this case)

Peak Hourly Traffic

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
584	4	180	61	74	15	6

V15

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
164	3	53	20	23	5	3

PHF = Peak Traffic / (4*V15)

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
0.8902439	0.33333	0.849056	0.7625	0.804348	0.75	0.5

Time	B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
12:30-12:45	193	2	44	20	13	9	0
12:45-1:00	178	2	43	17	26	4	0
1:00-1:15	158	5	53	19	14	2	0
1:15-1:30	170	3	46	17	19	4	0
1:30-1:45	180	2	57	18	19	0	4
1:45-2:00	172	2	42	21	19	5	0

Hourly Volume for Each Vehicle Class

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
699	12	186	73	72	19	0
686	12	199	71	78	10	4
680	12	198	75	71	11	4
522	7	145	56	57	9	4
352	4	99	39	38	5	4

Peak Hourly Traffic

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
699	12	199	75	78	19	4

V15

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
193	5	57	21	26	9	4

PHF = Peak Traffic / (4*V15)

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
0.90544041	0.6	0.87280701	0.8928571	0.75	0.5277777	0.25

Time	B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
3:30-3:45	187	2	82	20	32	5	0
3:45-4:00	217	2	75	22	44	5	0
4:00-4:15	212	1	75	23	41	1	4
4:15-4:30	239	2	70	20	26	3	0
4:30-4:45	180	1	68	22	33	1	0
4:45-5:00	158	2	65	22	30	4	1
5:00-5:15	0	0	0	0	0	0	0
5:15-5:30	0	0	0	0	0	0	0

Hourly Volume for Each Vehicle Class

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
855	7	302	85	143	14	4
848	6	288	87	144	10	4
789	6	278	87	130	9	5
577	5	203	64	89	8	1

338	3	133	44	63	5	1

Peak Hourly Traffic

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
855	7	302	87	144	14	5

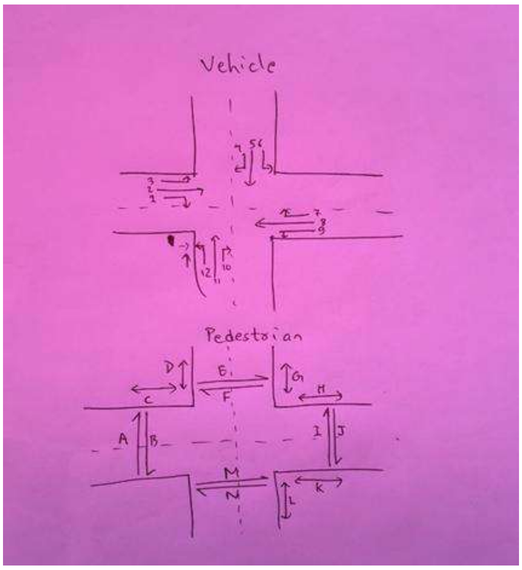
V15

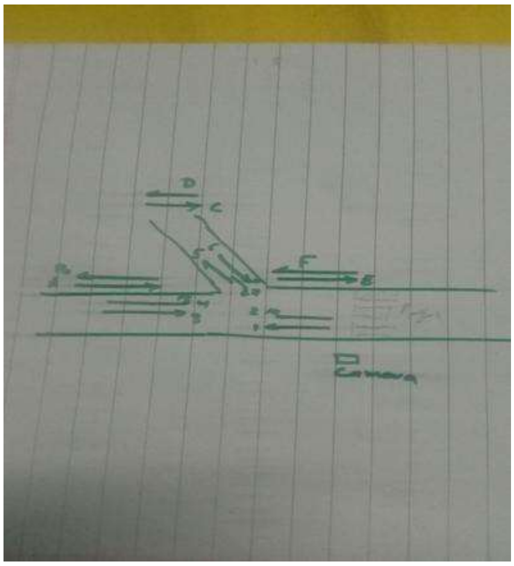
B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
239	2	82	23	44	5	4

PHF = Peak Traffic / (4*V15)

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
0.894351464	0.875	0.9207317	0.9456521	0.818182	0.7	0.3125

The notations used for the volume count of pedestrian and vehicular traffic is as follows:


Vehicle



PEDESTRIAN COUNT (Sample)

Approach: - GH (bi-directional)

Time	G(in)	H(out)
8:45-9:00	60	50
9:00-9:15	60	60
9:15-9:30	70	60
9:30-9:45	105	95
9:45-10:00	110	85
10:00-10:15	75	85
10:15-10:30	65	65
10:30-10:45	55	60

Time	G	H
12:30-12:45	35	30
12:45-1:00	35	35
1:00-1:15	40	45
1:15-1:30	50	40
1:30-1:45	43	38
1:45-2:00	35	35

Time	G	H
3:30-3:45	90	100
3:45-4:00	90	80
4:00-4:15	110	90
4:15-4:30	110	135
4:30-4:45	140	140
4:45-5:00	130	120
5:00-5:15	160	100
5:15-5:30	125	110

PHF = 0.924107

APPENDIX C –TRAFFIC VOLUME COUNT OF CHARDOBATO (SAMPLE)

VEHICLE COUNT

Approach: - From Tindobato to Panauti road (right turn-1)

Time	B/S	Tem/trac	T/J/M	Buses	LT/van	HV	Cycle
8:45-9:00	40	0	11	1	0	0	2
9:00-9:15	50	0	15	3	4	0	1
9:15-9:30	58	0	12	4	11	0	1
9:30-9:45	52	0	14	2	4	0	1
9:45-10:00	45	0	17	3	3	1	1
10:00-10:15	46	0	9	2	3	0	0
10:15-10:30	30	1	16	4	2	0	1
10:30-10:45	58	0	10	2	4	0	1

Hourly Volume for Each Vehicle Class

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
200	0	52	10	19	0	5
205	0	58	12	22	1	4
201	0	52	11	21	1	3
173	1	56	11	12	1	3
179	1	52	11	12	1	3

Peak Hourly Traffic

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
205	1	58	12	22	1	5

V15

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
58	1	17	4	11	1	2

PHF = Peak Traffic / (4*V15)

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
0.8836207	0.25	0.8529412	0.75	0.5	0.25	0.625

Time	B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
12:30-12:45	39	0	4	3	6	2	2
12:45-1:00	48	0	12	2	5	1	1
1:00-1:15	55	0	10	4	6	3	2
1:15-1:30	45	0	11	4	11	0	0
1:30-1:45	45	1	18	3	8	1	2
1:45-2:00	60	1	8	4	6	1	1

Hourly Volume for Each Vehicle Class

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
187	0	37	13	28	6	5
193	1	51	13	30	5	5
205	2	47	15	31	5	5
150	2	37	11	25	2	3
105	2	26	7	14	2	3

Peak Hourly Traffic

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
205	2	51	15	31	6	5

V15

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
60	1	18	4	11	3	2

PHF = Peak Traffic / (4*V15)

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
0.8541667	0.5	0.7083333	0.9375	0.7045455	0.5	0.625

Time	B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
3:30-3:45	51	2	19	4	5	3	1
3:45-4:00	70	1	15	6	1	1	0
4:00-4:15	72	1	11	5	1	0	3
4:15-4:30	61	0	20	5	4	2	1
4:30-4:45	55	0	12	9	1	0	0
4:45-5:00	70	3	18	4	4	0	1
5:00-5:15	90	0	17	8	0	1	2
5:15-5:30	78	1	22	4	2	3	0

Hourly Volume for Each Vehicle Class

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
254	4	65	20	11	6	5
258	2	58	25	7	3	4
258	4	61	23	10	2	5
276	3	67	26	9	3	4
293	4	69	25	7	4	3

Peak Hourly Traffic

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
293	4	69	26	11	6	5

V15

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
90	2	22	9	5	3	3

PHF = Peak Traffic / (4*V15)

B/S	tem/tract	T/J/M	Buses	LT/van	HV	Cycle
0.8138889	0.5	0.7840909	0.7222222	0.55	0.5	0.4166667

PEDESTRIAN COUNT

Time	APPROACH															
	A	B	C	D	E	F	G	H	I	J	K	L	M	N	P	Q
8:45-9:00	85	63	233	89	95	60	91	224	108	65	194	165	117	73	180	110
9:00-9:15	121	66	279	11	95	67	70	207	79	80	232	159	135	130	300	190
9:15-9:30	125	84	300	135	107	72	101	239	105	74	225	155	119	92	250	160
9:30-9:45	118	103	388	100	122	108	174	345	131	102	299	212	171	116	235	150
9:45-10:00	168	87	390	141	128	161	185	386	178	137	360	255	152	197	210	190
10:00-10:15	150	99	339	162	135	132	167	353	133	106	296	175	170	194	270	140
10:15-10:30	138	94	361	126	143	98	139	340	104	116	312	169	183	116	230	150
10:30-10:45	107	80	310	124	136	106	133	303	101	93	329	165	180	136	180	170

A	B	AB	E	F	EF	I	J	IJ	M	N	MN
449	316	765	419	307	726	423	321	744	542	411	953
532	340	872	452	408	860	493	393	886	577	535	1112
561	373	934	492	473	965	547	419	966	612	599	1211
574	383	957	528	499	1027	546	461	1007	676	623	1299
563	360	923	542	497	1039	516	452	968	685	643	1328

Hourly Volume

Peak Hourly Pedestrian

Time	APPROACH															
	A	B	C	D	E	F	G	H	I	J	K	L	M	N	P	Q
12:30-12:45	123	110	326	174	94	115	183	331	132	84	195	182	145	94	120	90
12:45-1:00	112	118	342	182	107	110	166	352	116	108	222	196	146	132	125	85
1:00-1:15	108	99	315	226	121	127	153	369	103	101	242	218	149	139	135	90
1:15-1:30	101	88	289	239	122	129	178	383	119	121	218	240	154	143	140	112
1:30-1:45	116	108	322	235	128	132	184	348	138	119	220	205	146	148	160	120
1:45-2:00	125	105	339	215	104	126	172	329	123	102	215	186	152	147	175	128

Hourly Volume

A	B	AB	E	F	EF	I	J	IJ	M	N	MN
444	415	859	444	481	925	470	414	884	594	508	1
437	413	850	478	498	976	476	449	925	595	562	1157
450	400	850	475	514	989	483	443	926	601	577	1178
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0

Peak Hourly Pedestrian

AB	EF	IJ	MN
859	989	926	1178

Time	APPROACH															
	A	B	C	D	E	F	G	H	I	J	K	L	M	N	P	Q
3:30-3:45	102	109	366	164	140	157	230	438	141	146	365	208	145	151	370	160
3:45-4:00	108	152	392	171	148	149	206	364	126	132	340	164	155	139	460	220
4:00-4:15	120	160	403	225	145	162	182	348	106	121	315	132	160	144	520	230
4:15-4:30	132	122	417	240	151	168	189	383	134	140	332	121	169	128	480	210
4:30-4:45	140	98	433	249	152	142	213	411	112	113	330	134	161	119	500	220
4:45-5:00	135	102	444	233	165	148	222	432	135	122	342	146	162	160	400	230
5:00-5:15	129	94	420	220	160	140	218	401	110	118	346	139	172	146	440	230
5:15-5:30	127	91	403	215	148	134	213	364	121	114	357	158	178	147	460	238

Hourly Volume

A	B	AB	E	F	EF	I	J	IJ	M	N	MN
462	543	1005	584	636	1220	507	539	1046	629	562	1191
500	532	1032	596	621	1217	478	506	984	645	530	1175
527	482	1009	613	620	1233	487	496	983	652	551	1203

536	416	952	628	598	1226	491	493	984	664	553	1217
531	385	916	625	564	1189	478	467	945	673	572	1245

Peak Hourly Pedestrian

AB	EF	IJ	MN
1032	1233	1046	1245

Peak Hourly Flow

AB	EF	IJ	MN
0.921428571	0.96630094	0.911149826	0.957692308

Hence, The classification of the vehicles were done as:

Movement Definitions - Included Movement Classes

Name	ID	Model Designation	Type
Light Vehicles	LV	Light Vehicle	Standard
Heavy Vehicles	HV	Heavy Vehicle	Standard
Buses	B	Heavy Vehicle	Standard
Bicycles	C	Light Vehicle	Standard
Large Trucks	TR	Heavy Vehicle	Standard
Tempo	U1	Light Vehicle	User Defined
Motorcycle	U2	Light Vehicle	User Defined

Table 4.4.2 Classification by Movement Class

Movement Class	PCU per Vehicle
Two-wheelers	0.25
Tempo	1.3
Car	1
Bus	2.19
Light Truck	1.5
Heavy Vehicles	3

Source: (Shrestha, 2014) and KVSTP 2022 (for the last two factors

Table 4.4.3 Sample Pictures of all Vehicle Class

Vehicle Class	Figure
Car (T/J/M)	   
Two-wheelers (B/S)	 
Tempo and Tractors	 
Bus	
Light Truck and Van	  
Heavy Vehicles	

Approach and Exit Cruise Speed: Cruise speed refers to the average speed of a vehicle during uninterrupted travel, not including delays at intersections. When traffic flow is continuous, cruise speed is considered the free-flow speed and a reduced speed is determined based on flow rate to account for in-stream delays. For regular movements, the specified speed value is used without any modifications. The speed limit may also be a suitable value to use. (SIDRAsolutions.com)

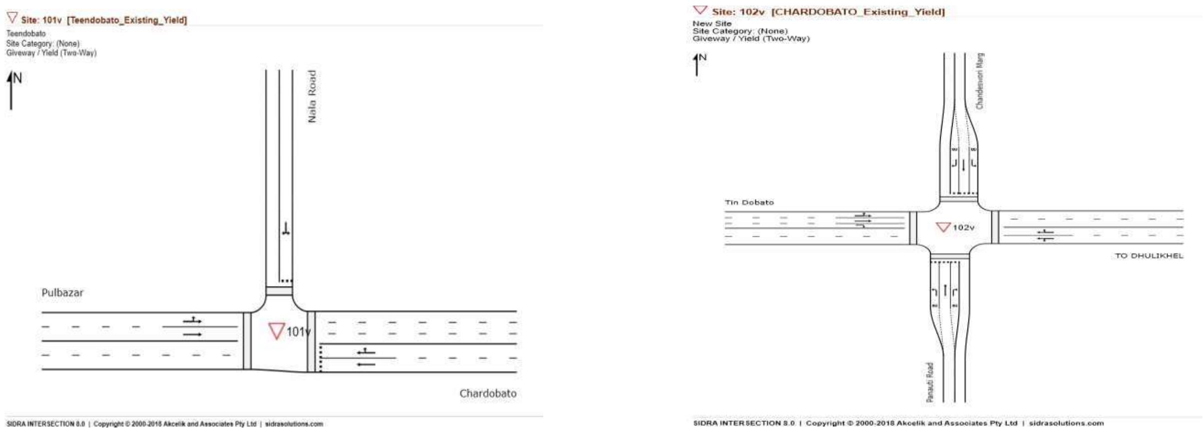
The speed limit of the highway section was 50kmph. Speed survey was also carried out in all the approaches and exits during off-peak hours. Spot speed observation method with short base length of about 40 meters was marked approximately using pacing for the survey. Time was marked using Stopwatch in smartphones.

Back of Queue: Back of Queue is the maximum backward extent of the queue relative to the stop line during a signal cycle (Akcelik and Associates, 2018). The GNSS system, particularly, SW Maps application was used to pin-point Back of Queue location at the intersection with general accuracy of about 3 meters

4.4.1 LOS Study of Existing Case (Unsignalized)

Site Layout modeled in SIDRA Intersection is as follows:

Figure 4.4.3 Site Layout in SIDRA



The Network Linkage made in SIDRA is as follows:

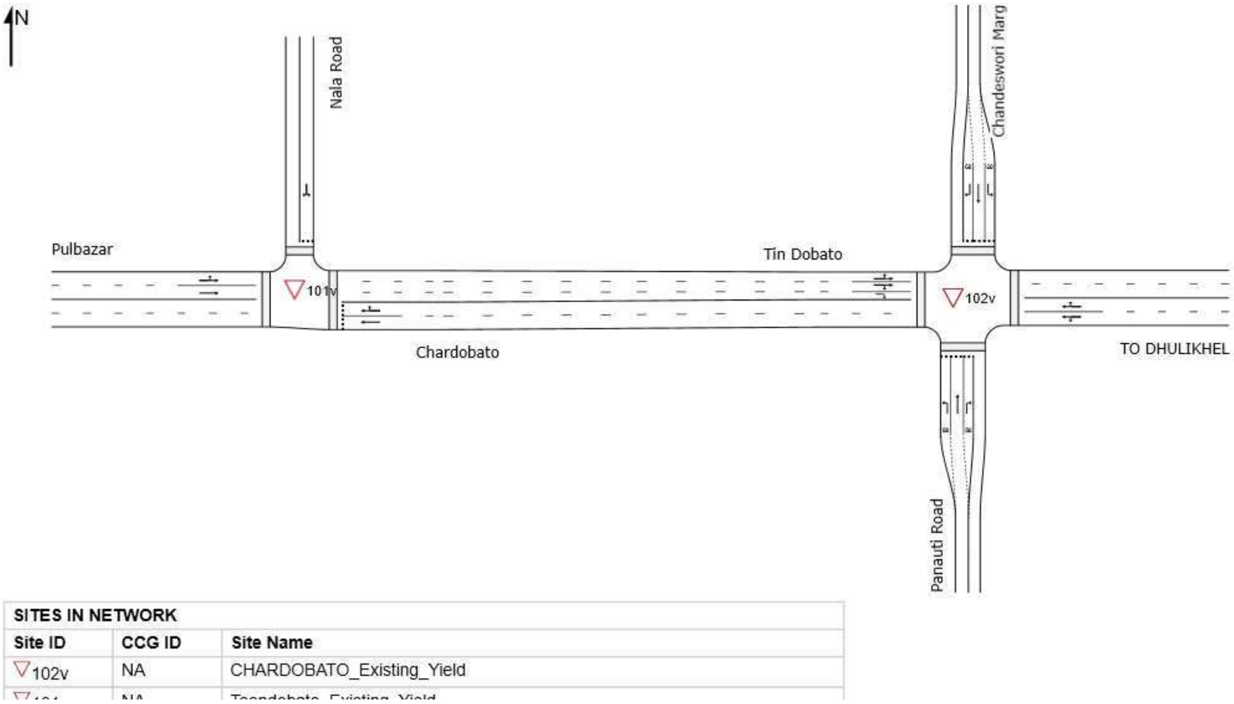


Figure 4.4.4 Network Linkage in SIDRA

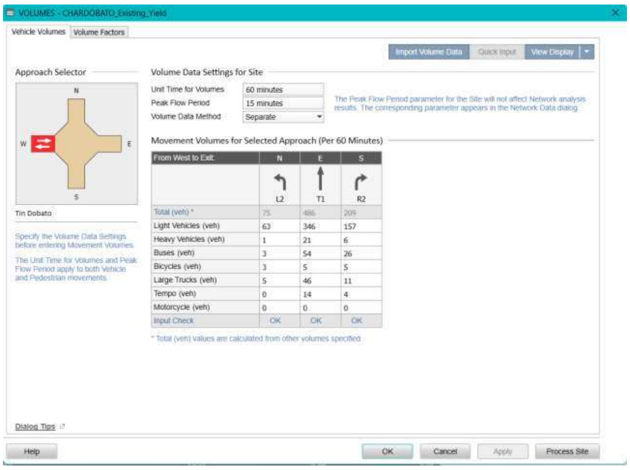
Data Input in SIDRA

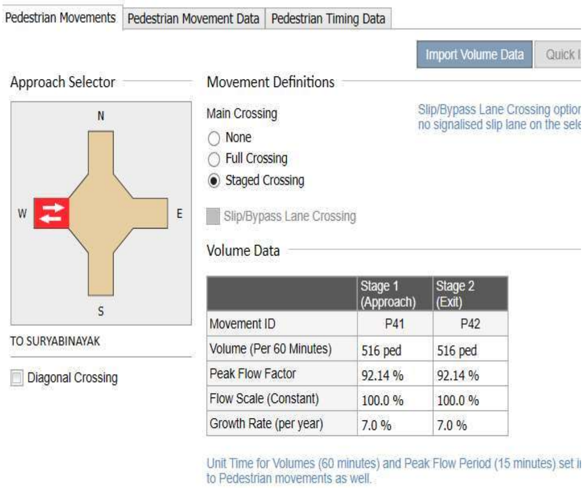
Major Input parameters for Unsignalized case are: Flow rates and Flow factors, Priorities, Critical gap and follow up headway

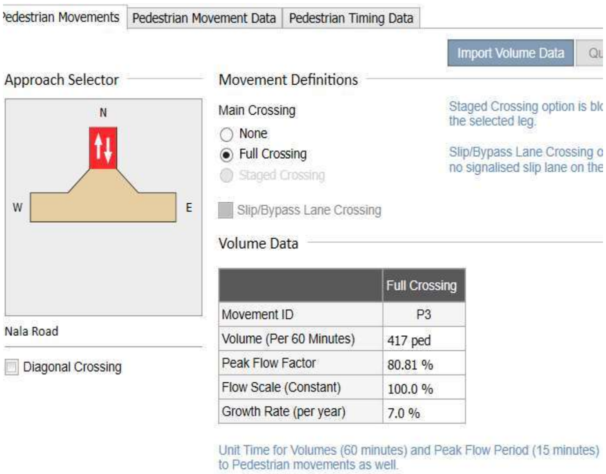
The input sample of each parameter in SIDRA is given below:

Classified Vehicle Volume and Pedestrian Volume was input in the SIDRA Intersection as shown in the sample below:

Figure 4.4.5 Classified Vehicle and Pedestrian Volume Input







The volume factors were given as input for each vehicle class as shown below:

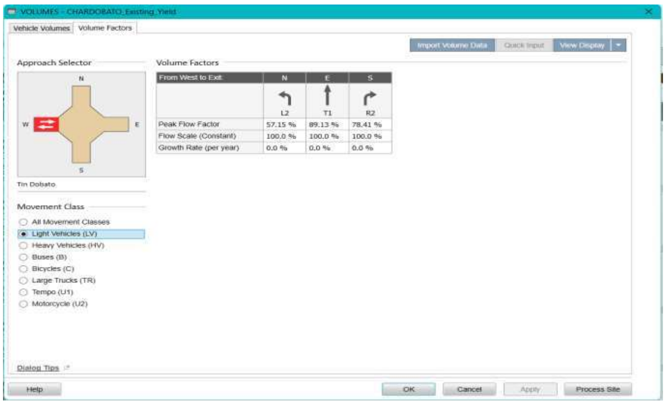


Figure 4.4.6 Volume Factor Input

Priorities were set as per Indo-HCM provisions (See Theory Section) and field condition such as the sample shown below:

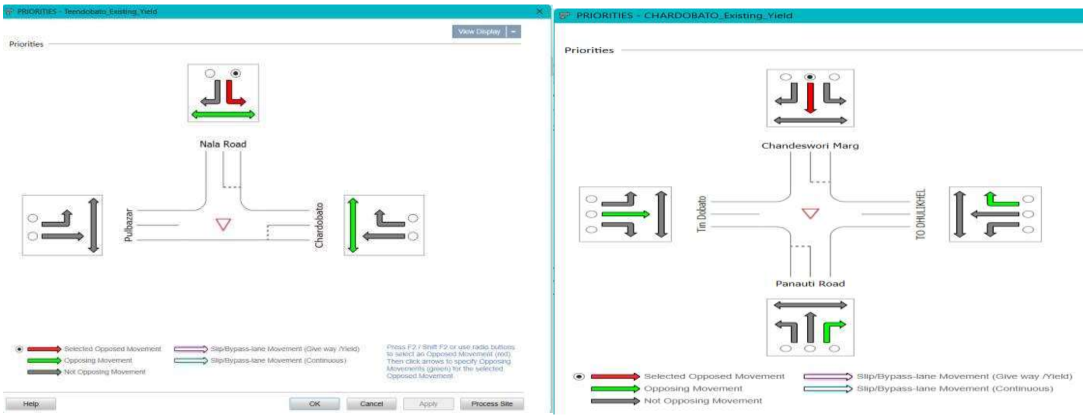


Figure 4.4.7 Priorities Input

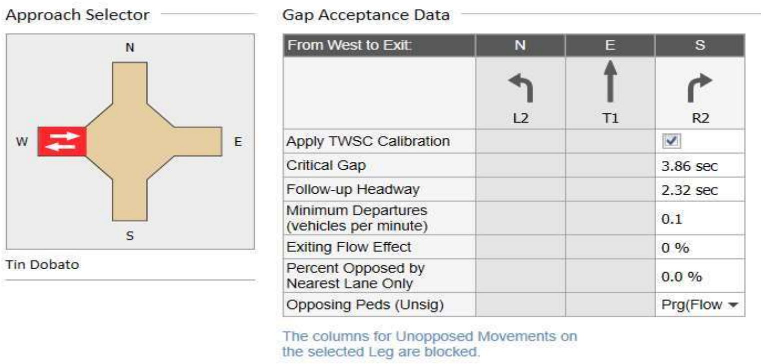


Figure 4.4.8 Critical Gap and Follow-up Time Input

Approach and Exit Cruise speed is input as follows:

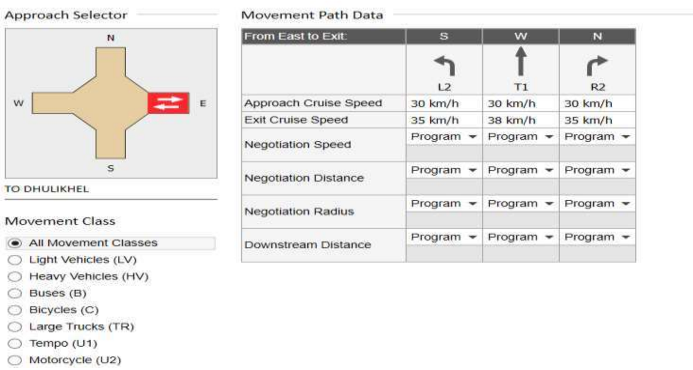


Figure 4.4.9 Approach and Exit Cruise speed input Vehicle Length and Queue space (Other data being irrelevant was set as default)

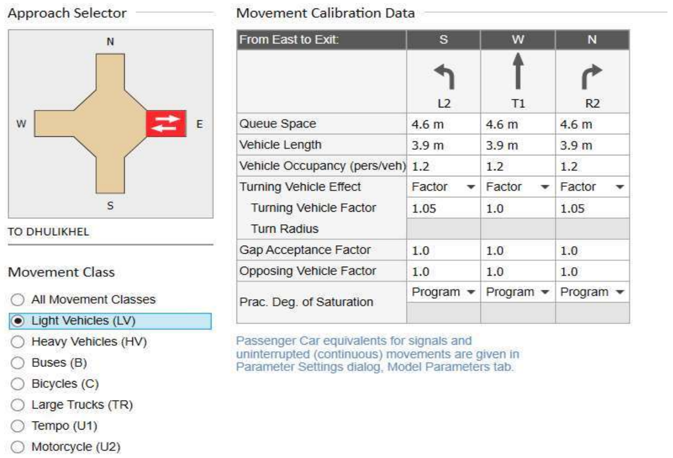


Figure 4.4.10 Vehicle Length and Queue Space input

Because of restriction on size of the report and to enhance comprehensiveness only distinctive part of analysis is only presented here.

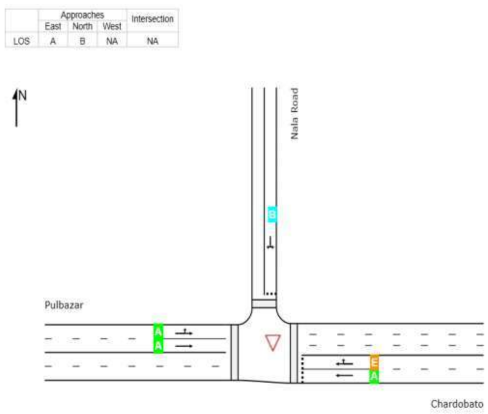
Detailed input parameters could be found in: -

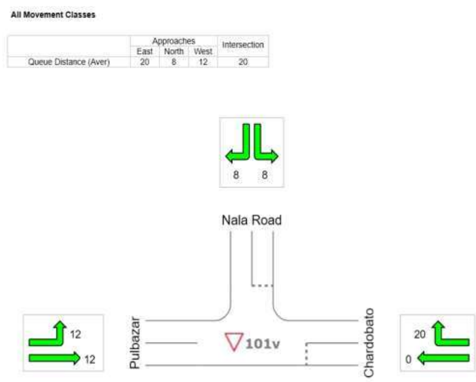
Appendix I- Detailed output in Sidra

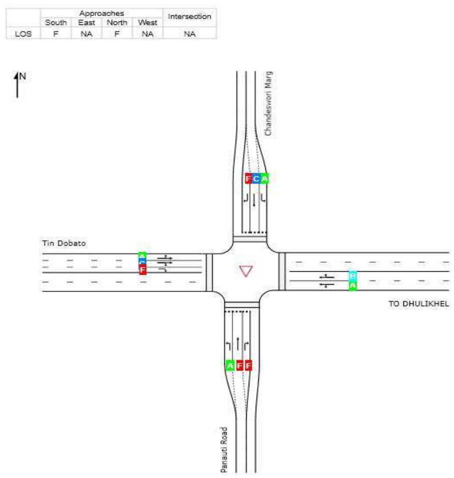
Data Output in SIDRA

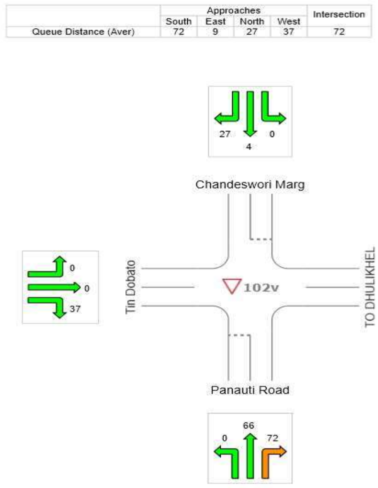
After input of both the sites were complete, the model was run and the output (Level of Service and Queue) of the model is shown as below:

Figure 4.4.11Level of Service and Queue Output



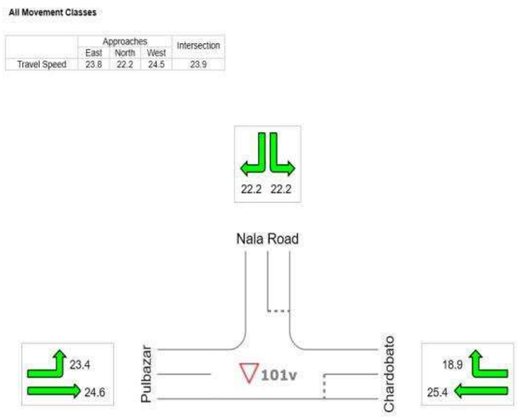


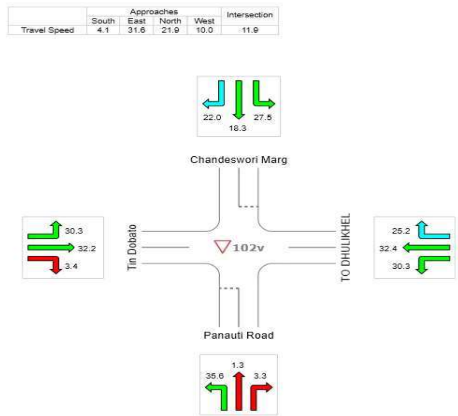




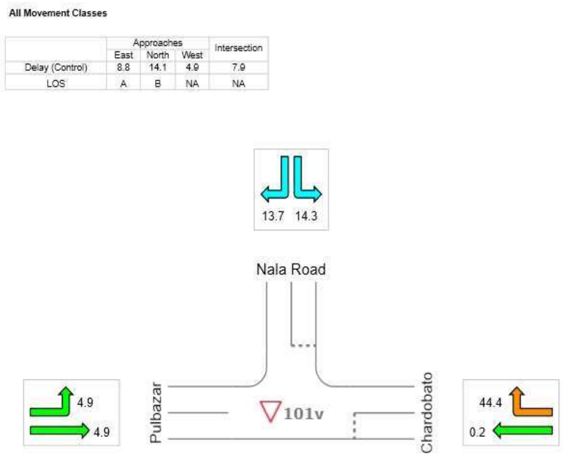
Travel Speed Output from SIDRA is as follows:

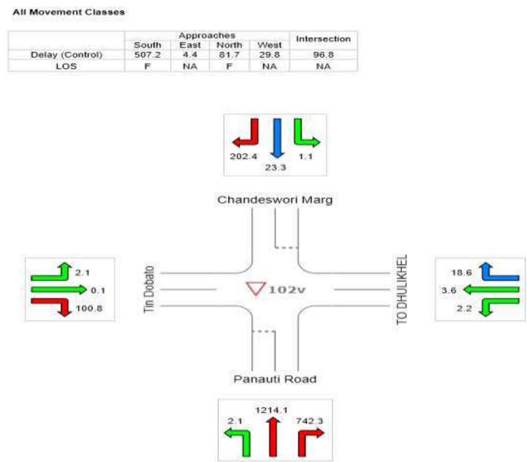
Figure 4.4.12 Travel Speed Output





Control Delay Output from SIDRA is as follows:





4.4.2 Manual Calculation of LOS:

For manual calculation, Indo-HCM 2017 was followed.

A. Capacity Estimation

The capacity of movement for Unsignalized Intersection in PCU per hour is given as:

$$C_x = a \times V_{c,x} \frac{e^{-V_{c,x}(t_{c,x}-b)/3600}}{1-e^{-V_{c,x}t_{f,x}/3600}}$$

Where,

Cx = capacity of movement ‘x’ (in PCU/h),

Vc,x = conflicting flow rate corresponding to movement x (PCU/h),

tc,x = critical gap of standard passenger cars for movement ‘x’ (s),

tf,x = follow-up time for movement ‘x’ (s), and

‘a’ and ‘b’ = adjustment factors based on intersection geometry.

Table 4.4.4 Adjustment Factors for Capacity model

Major Street Configuration	Adjustment Factors	Subject Movement		
		Right Turn from Major	Right Turn from Minor	Through on Minor
Four-lane divided	a	0.80	1.00	0.90
	b	1.30	2.16	5.04
Two-lane undivided	a	0.70	0.80	1.10
	b	-0.11	0.72	0.72

Table 4.4.5 Level of Service at Unsignalized Intersection

Level of Service	Volume-Capacity ratio
A	< 0.15
B	0.16 - 0.35
C	0.36 - 0.55
D	0.56 - 0.80
E	0.81 - 1.00
F	> 1.00

Manual Level of Service Calculation in Unsignalized Intersection:

Table 4.4.6 LOS Calculation of Unsignalized Intersection

For Approach East Right Turn : Chardobato					
Volume	Conflicting flow v2+v4+v5+0.5v11	a	b	tcx	tfx
56	774	0.8	1.3	4.5	3
Capacity					654.6879
For right turn use about capacity of lane=25%					163.672
v/c =					0.342148
			OK	LOS B	<0.36

For Approach West Right Turn: Chardobato					
Volume	Conflicting flow v8+v10+v11+0.5v6	a	b	tcx	tfx
255	1113	0.8	1.3	4.5	3
Capacity					547.7174
For right turn use about capacity of lane=45%					246.4728
v/c =					1.034597
			OK	LOS F	>1

For Approach North Right turn: Tindobato					
Volume	Conflicting flow v2+v3+0.5v1	a	b	tcx	tfx
57	1455	1	2.16	4.5	3
Capacity					804.3655
For right turn use about capacity of lane=40%					321.7462
v/c =					0.177158
			OK	LOS B	>0.16

For Approach East Right turn: Tindobato					
Volume	Conflicting flow v7+v3	a	b	tcx	tfx
176	828	0.8	1.3	5.5	4
Capacity					419.1505
For right turn use about capacity of lane=50%					209.5752
v/c =					0.839794
			OK	LOS E	>0.81

Sample Calculation:

for Chardobato east right turn

volume of right turning vehicle = 56pcu

conflicting flow Vc,x = v2+v4+v5+0.5v11=616+68+61+0.5*58=774pcu

from Indo-HCM table of capacity adjustment factor

constants, a = 0.8

b = 1.3

critical gap tcx = 4.5sec

follow up gap = 3sec

capacity C = 0.8*774*exp (-774*(4.5-1.3)/3600)/ (1-exp (-774*3/3600))

= 654.68pcu/hr

It is shared lane and volume of right turn in that lane is about 15% its increase by 10% (right turn consumes more capacity of lane with respect to through)

so, capacity of lane = 0.25*654.68

= 163.67pcu/hr

volume to capacity ratio

v/c = 56/163.67

= 0.34 <0.36

according to level of service of unsignalized condition, LOS=B

Requirement of signalized intersection in our study location:

As shown in the model output of Operational Analysis above, the queue length of the intersection is about 72 meters and the Level of Service for the intersection (Chardobato) is too low, for example: LOS F for minor approach. In addition to it, the existing operation of the intersection includes:

- The effect of no regard for the priority rules by minor movement to the major movement in the intersection
- Non-existence of stop line in every approaches
- A single lane is used by number of vehicles (especially two-wheelers) at the same time
- The approach lane width widens near intersection accommodating multiple vehicles at the same time

The effect of these parameters results in the under-estimation of the capacity of the intersection which was overcome by the addition of short lanes in the approaches as shown above picture.

If above parameters like stop-line and lane use rules are enforced, the capacity of the intersection reduces significantly as shown in the output below [Without short-lane addition output of LOS and Queue].

Calibration:

For calibration, queue length measured in the field, which was found to be 60 meters, was compared with that obtained from SIDRA intersection (72m) within the tolerance limit of 20%. Also, travel speed was also compared from the field observation for calibration of the model.

MUTCD Guideline for Warrant of Signals in the intersection mentions nine Warrants out of which Warrant 3: Peak hour volume (A) is used to justify signalization:
The three following criteria should be satisfied for the same hour (any four consecutive 15-minute periods) of an average day:

1. The total stopped delay during any four consecutive 15-minute periods on one of the minor-street approaches (one direction only) controlled by a stop sign is equal to or greater than 4 veh-h for a one-lane approach and 5 veh-h for a two-lane approach.

2. The same minor-street approach (one direction only) volume should be equal to or exceed 100 and 150 vehicles per hour for one moving lane of traffic and two moving lanes of traffic, respectively.

3. The total intersection entering volume is equal to or greater than 650 vehicles per hour for three-leg intersections and 800 vehicles per hour for four-leg and multi leg intersections.

For Chardobato:

1. Delay:

    South Approach of the intersection has a delay of 507.2 seconds per vehicle and total vehicle count of 302 vehicles.

    Total Delay of approach = (507.2 * 302)/3600 = 42.54 veh-hr > 5 veh-hr (OK)

2. Volume of the approach of single lane = 302 veh > 100 veh (OK)
3. Total vehicle for intersection = 2149 veh > 800 veh (OK)

For Tindobato:

1. Delay:

    South Approach of the intersection has a delay of 14.1 seconds per vehicle and total vehicle count of 139 vehicles.

    Total Delay of approach = (14.1 * 139)/3600 = 0.54 veh-hr < 5 veh-hr (Not- OK)

2. Volume of the approach of single lane = 139 veh > 100 veh (OK)

Total vehicle for intersection = 1759 veh > 650 veh (OK)

Also, considering the Crash records of both the intersections,

Therefore, Signalization is justified.

Thus, after the expansion of the highway, a suitable traffic management method is required to accommodate this heavy traffic flow. Signalized Intersection for both the sites is recommended. The Design analysis for the same is given below:

4.4.3 LOS Study of Proposed Case (Signalized)

Saturation Flow Calculation

Base Saturation flow rate: Saturation flow rate under stated base conditions of intersection relating to traffic, geometric and control conditions and is expressed in PCU/h. of green. (Indo-HCM 2017)

Using formula,

$$\mathrm{USF_0} = \begin{cases} 630; & for \quad w < 7.0m \\ 1140 - 60w; & for \quad 7.0 \le w \le 10.5m \\ 500; & for \quad w > 10.5m \end{cases}$$

Where,

USFO = Unit base saturation flow rate (in PCU / hour / m)

w = effective width of approach in meters (m).

The prevailing **saturation flow rate** of the intersection approach for the movement group under consideration is then obtained as :

SF = w × USF0 × fbb × fbr × fis

Where,

SF = Prevailing saturation flow rate in PCU/hour

w = effective width of the approach in 'm' used by the movement group

fbb = Adjustment factor for bus blockage due to curbside bus stop

fbr = Adjustment factor for blockage of through vehicles by standing right turning vehicles waiting for their turn.

fis = Adjustment factor for the initial surge of vehicles due to approach flare and anticipation effect.

Manual calculation for input of saturation flow is given as:

Sample Calculation:

Intersection: Chardobato

Approach: West

Lane Number: Lane 1

Here,

Approach width (Aw) = 14.5m > 10.5m

Hence, USF0 = 500 veh/hr

Now,

Lane width(w) = 2.75m

Saturation Flow (SF) = w*USF0 = 2.75 * 500 = 1375 veh/hr

The required saturation flow rate to be input in SIDRA is 1375 veh/hr.

User Guide of SIDRA Intersection states:

"Where measured lane saturation flows are available, they should not be specified as basic saturation flow rates. Instead, the default basic saturation flow values should be adjusted in such a way that the saturation flow rate estimated by SIDRA INTERSECTION is the measured value. This method takes into account the saturation flow adjustments that will be made by the program to account for various factors that affect saturation flow rates. This is important in order to avoid double counting that may lead to significant overestimation or underestimation of saturation flows."

Since, SIDRA only inputs Basic Saturation flow rate as input, the output has to be compared with the required Saturation flow rate in iterations to find the value of base saturation flow input, which would result in acceptable saturation flow rate compared to required value. For example: In this case;

Base Flow Given as Input in SIDRA Intersection = 1570 veh/hr

SIDRA SF Output = 1371 veh/hr

Difference = 1375 – 1371 = 4 veh/hr < 5 veh/hr (OK)

Other Calculations are given below:

Table 4.4.7 Calculation of Saturation Flow Rate

Saturation Flow Rate				
Intersection		Chardobato		
Approach	Width (Aw)	USF0		
West	14.5	500		
East	14.5	500		
South	7	720		
North	6.5	630		
Approach	West			
Lane	Width(w)	SF=w*USF 0 (veh/hr)	SIDRA Base Flow Given (veh/hr)	SIDRA SF (veh/hr)
Lane 1	2.75	1375	1570	1371
Lane 2	2.75	1375	1570	1371
Lane 3	3.5	1750	1930	1752
Lane 4	3.5	1750	1930	1752
Lane 5	2.5	1250	1450	1249
Approach	East			
Lane	Width	SF=w*USF 0	SIDRA Base Flow Given	SIDRA SF
Lane 1	2.75	1375	1570	1373
Lane 2	2.75	1375	1570	1373
Lane 3	3.5	1750	1930	1755

Lane 4	3.5	1750	1930	1755
Lane 5	2.5	1250	1450	1251
Approach	North			
Lane	Width	SF=w*USF 0	SIDRA Base Flow Given	SIDRA SF
Lane 1	2	1260	1470	1261
Lane 2	2.5	1575	1830	1578
Lane 3	2	1260	1470	1261
Approach	South			
Lane	Width	SF=w*USF 0	SIDRA Base Flow Given	SIDRA SF
Lane 1	2.5	1800	2085	1798
Lane 2	2.25	1620	1890	1621
Lane 3	2.25	1620	1890	1621

Saturation Flow Rate

Intersection		Tindobato		
Approach	Width	USF0		
West	13	500		
East	15	500		
North	4.5	630		

Approach	West			
Lane	Width	SF= w*USF0	SIDRA Base Flow Given	SIDRA SF
Lane 1	2.75	1375	1570	1371
Lane 2	2.75	1375	1570	1371
Lane 3	3.75	1875	1930	1752
Lane 4	3.75	1875	1930	1752
Approach	East			
Lane	Width	SF= w*USF0	SIDRA Base Flow Given	SIDRA SF
Lane 1	2.75	1375	1570	1373
Lane 2	2.75	1375	1570	1373
Lane 3	3.5	1750	1930	1755
Lane 4	3.5	1750	1930	1755
Lane 5	2.5	1250	1450	1251
Approach	North			
Lane	Width	SF= w*USF0	SIDRA Base Flow Given	SIDRA SF
Lane 1	2.5	1575	1470	1261
Lane 2	2	1260	1830	1578

Peak hour flow (Vp):

$$Vp = \frac{V}{PHF}$$

Where,

Vp = flow rate during peak 15-min period (veh/h),

V=hourly volume (veh/h), and

PHF=peak-hour factor

Effective green time(gi):

gi = Gi + Yi - tl

where,

gi= Effective green time for phase i (sec)

Gi= Green time for phase i (sec);

Yi= Yellow time for phase i (sec)

Tl=Total lost time for phase i (sec) (HCM 2000)

Pedestrian crossing time (G_{min})

$$G_{min} = 3.2 + \frac{dx}{v_{ped}}$$

Volume input

Volume count of Classified Vehicular movement and pedestrian movement is the same as that of Unsignalized intersection (See Previous Section)

Phasing and Timing Determination

Manual calculation of Cmin

Table 4.4.8 Calculation of Flow ratio

Phase	Movements	v4	v10	
Phase A	volume	68	126	
	SF	1261	1621	
	v/s	0.053926	0.07773	Take 0.1
	Movements	v5	v11	
phase B	volume	61	59	
	SF	1578	1621	
	vls	0.038657	0.036397	Take 0.1
	Movements	v1	v7	
phase C	volume	255	56	
	SF	1249	1251	
	v/s	0.204163	0.044764	Take 0.20
	Movements	v2	v8	
phase D	volume	616	908	
	SF	4875	4880	
	v/s	0.126359	0.186066	Take 0.19

Here, $G_{min} = 3.2 + \frac{dx}{v_{ped}}$

For major approach, Gmin = 3.2 + 22/ 1.3 = 20.12 sec

For minor approach, Gmin = 3.2 + 12/1.3 = 12.43 sec

Sum of Volume = 1350 vehicle/hr

Sum(v/s) = 0.61

Here,

$C_{min} = \frac{N * T_L * Xc}{Xc - \sum v/s}$ = (4*4*0.8)/ (0.8 - 0.61) = 67.37 sec ~ 70 seconds

Effective green time (g)=70- 4 * 4 = 54sec ~ 55 seconds

Now , $g_i = \frac{v_i}{\sum v} * g_{eff}$

Table 4.4.9 Effective green time calculated

g1	5.133333
g2	2.44
g3	10.2
g4	36.32
all red	Take 10 seconds

Here,

Gmin is 12 sec so all calculated green time is increased by about 12 sec so,

Green time input per phase,

Table 4.4.10 Effective green adjusted

Phase	g1	g2	g3	g4
effective green	20	15	25	45

Here, C = 115 seconds

All red = 10 seconds

Finally, the phase sequence and phase time adopted is as shown below:

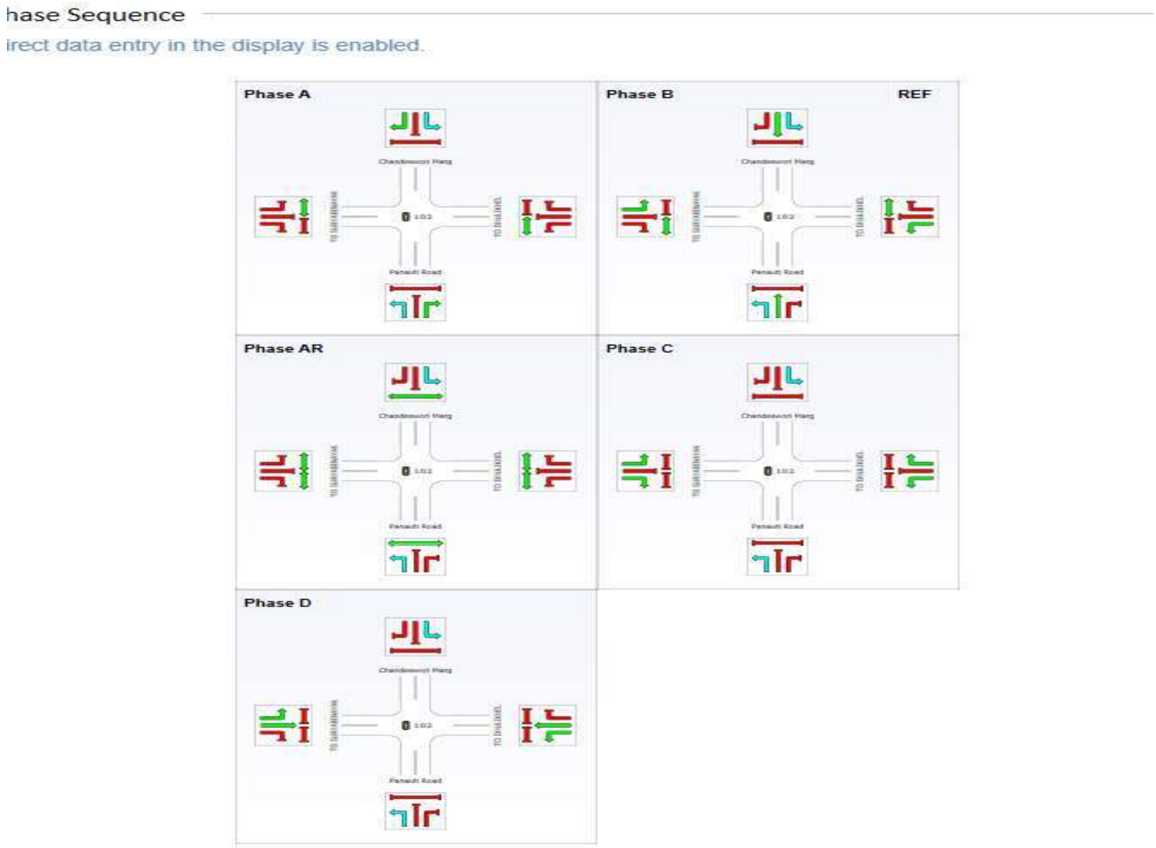


Figure 4.4.13 Phase Sequence of Chardobato

Phase Sequence

Direct data entry in the display is enabled.

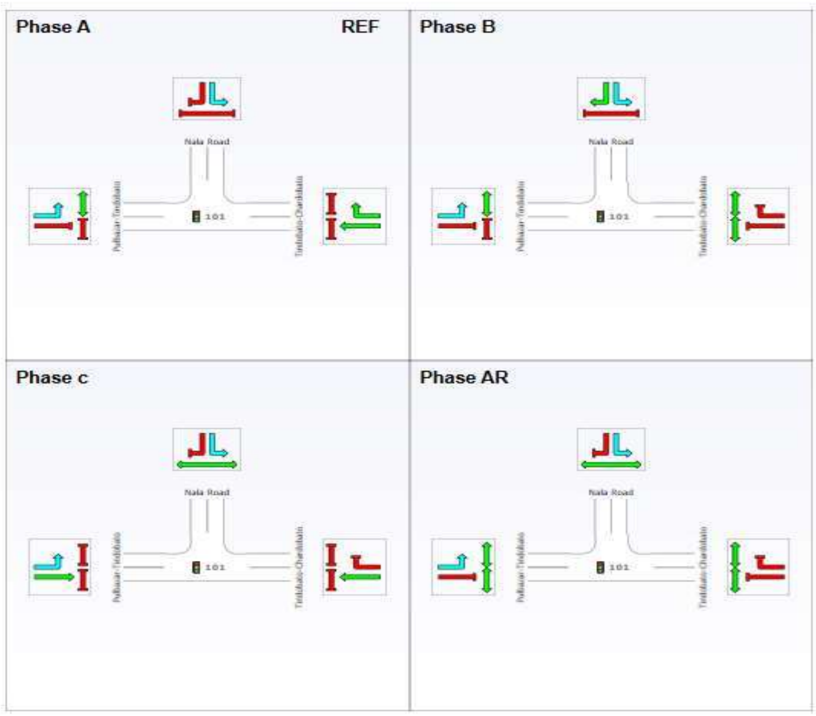


Figure 4.4.14 Phase Sequence of Tindobato

Sequences | Sequence Editor | Phase & Sequence Data | Timing Options | Advanced

Sequence **Leading Right Turn**

Phase Data

Phase:	A	B	AR	C	D
Variable Phase	☐	☐	☐	☐	☐
Reference Phase	○	◉	○	○	○
Phase Time (optional)	20 sec	15 sec	10 sec	25 sec	45 sec
Phase Frequency	Program	Program	Program	Program	Program
Yellow Time	4 sec	4 sec	4 sec	4 sec	4 sec
All-Red Time	2 sec	2 sec	2 sec	2 sec	2 sec
Dummy Movement Data:					

Figure 4.4.15 Phase Data for Chardobato

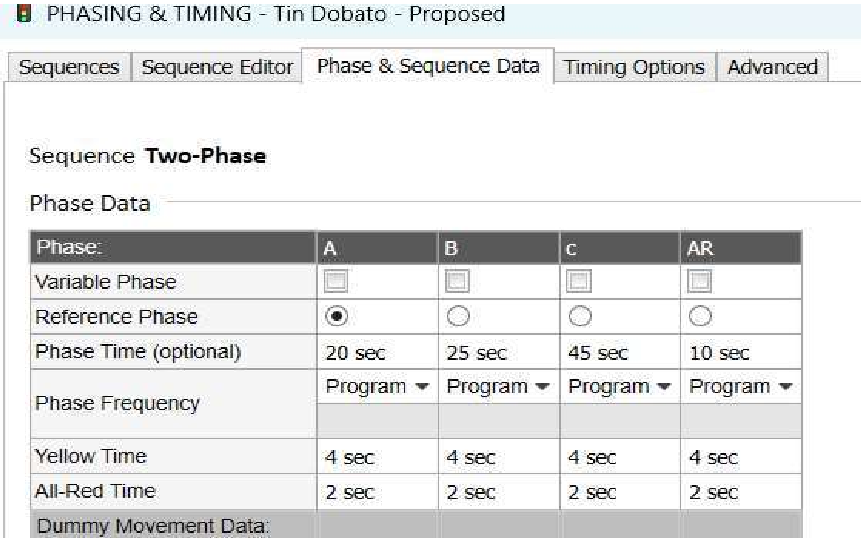


Figure 4.4.16 Phase Data for Tindobato

Data Output in SIDRA

After input of all the parameters was completed for both sites, the model was run and the output of the same is as shown below:

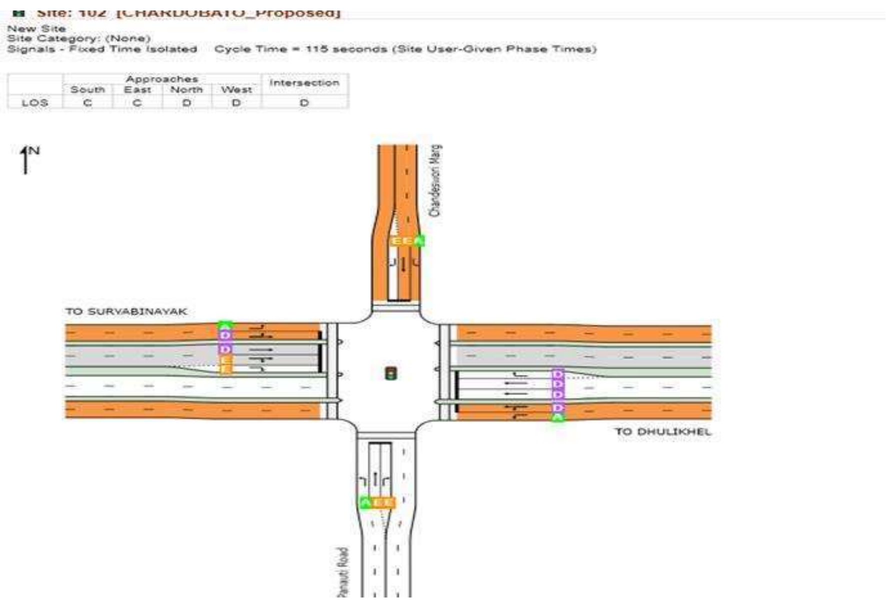


Figure 4.4.17 Level of Service in SIDRA

LANE LEVEL OF SERVICE

Lane Level of Service

Site: 101 [Tin Dobato - Proposed]

New Site
Site Category: (None)
Signals - Fixed Time Isolated Cycle Time = 100 seconds (Site User-Given Phase Times)
Sensitivity Analysis (Critical Gap & Follow-up Headway): Results for Parameter Scale = 80.0 %

	Approaches			Intersection
	East	North	West	
LOS	B	B	C	B

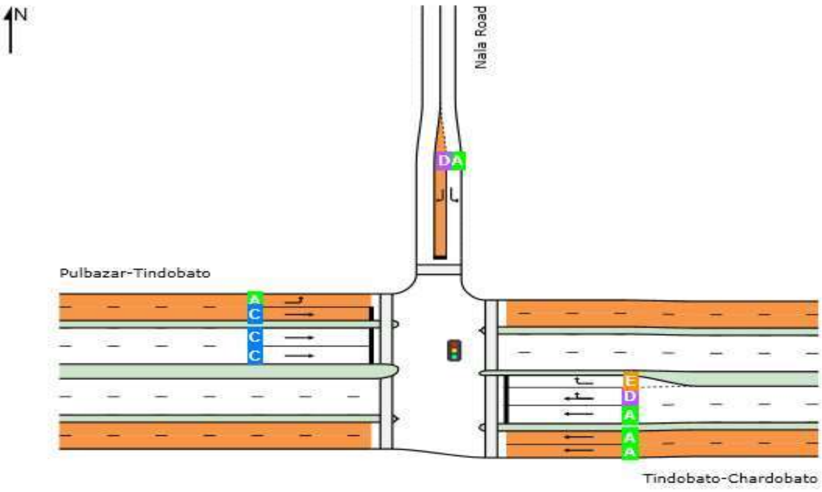
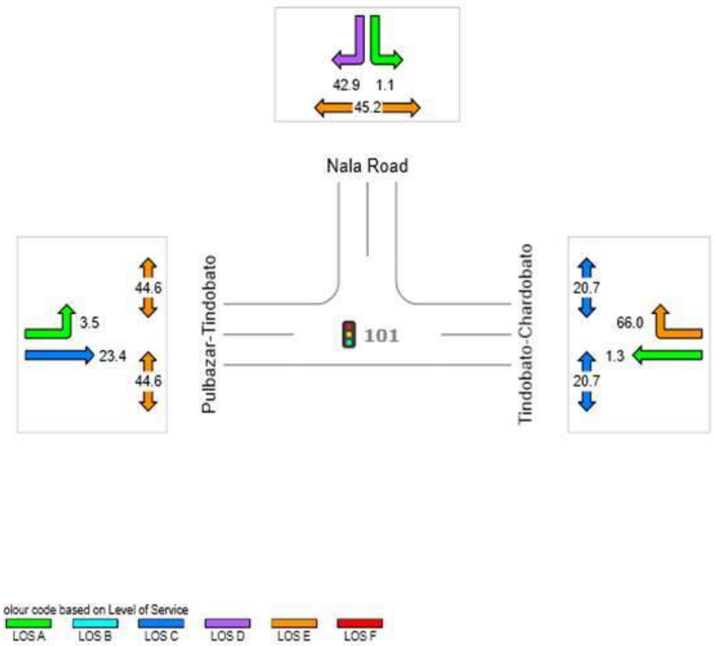


Figure 4.4.18 Delay in SIDRA



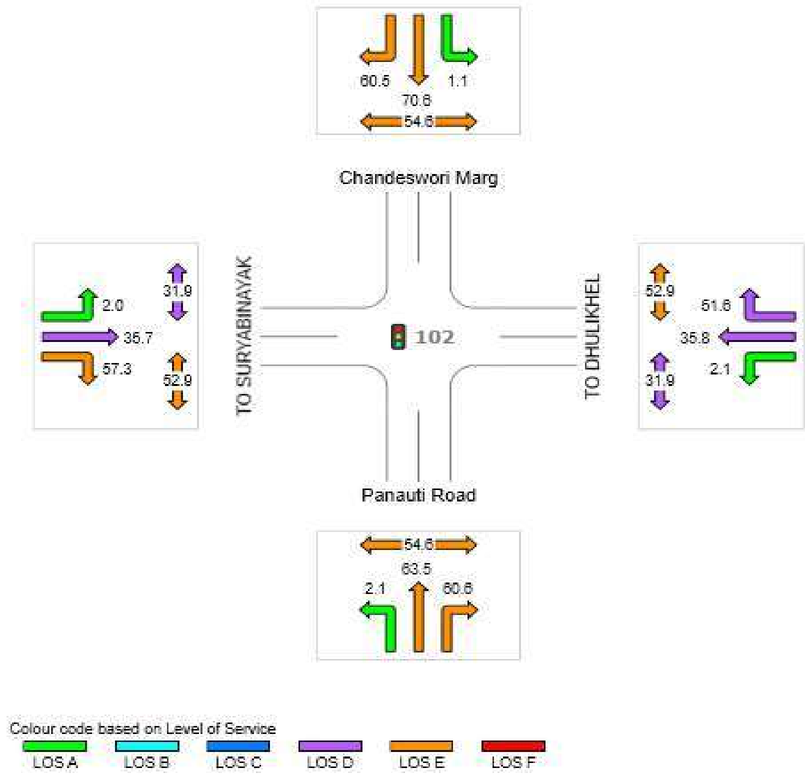


Figure 4.4.19 Queue Distance in SIDRA

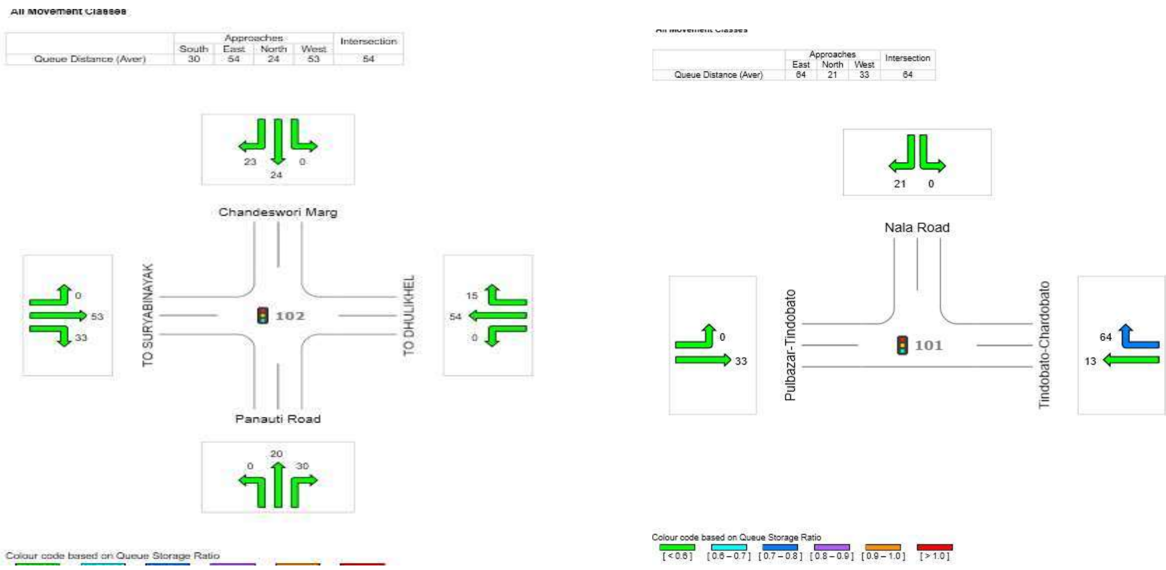
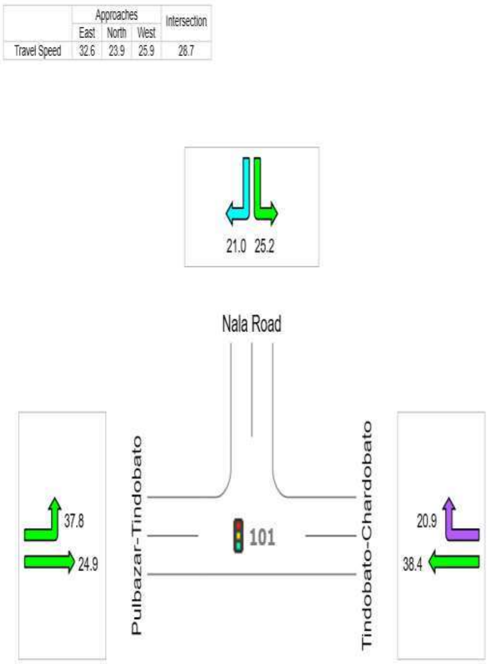
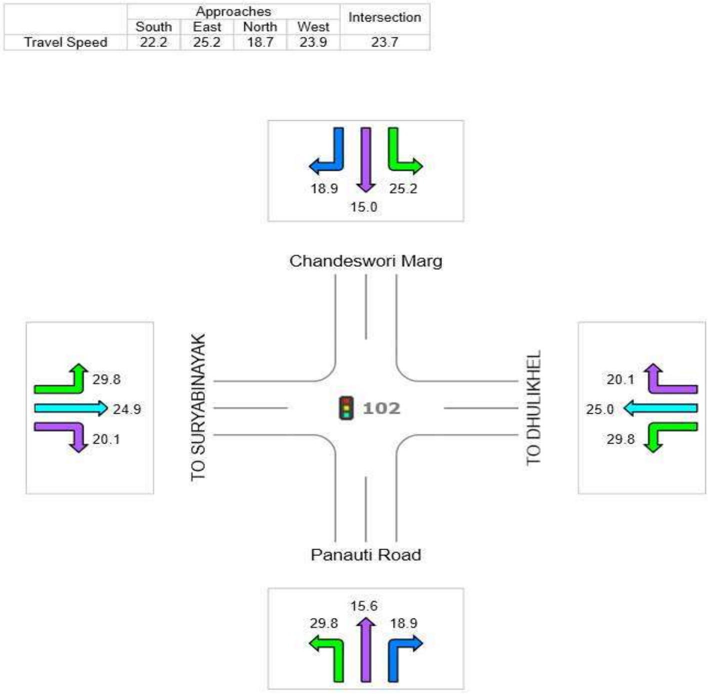


Figure 4.4.20 Travel Speed in SIDRA





4.4.4 Manual Calculation of LOS

Table 4.4.11 Sample Manual Calculation of LOS of Signalized Intersection

CHARDOBATO SOUTH														
G	C	Approach	Si	ci	Vi	Xi	PF	g/C	min of (1,X)	d1	d2	d3	delay sum (di)	vi*di
15	115	**Through**	1621	211.4348	59	0.279046	1	0.13043478	0.279046	45.1205255	3.26226187	0	48.38278739	2854.58446
20	115	**Right**	1621	281.913	125	0.443399	1	0.17391304	0.443399	42.5178011	4.98707721	0	47.5048783	5938.10979
115	115	left	1798	1798	205	0.114016		1	0.114016	0	0.12878955	0	0.128789553	26.4018584
					389								di*vi	8819.0961
													LOS C	22.6711982

Sample Calculation :

Green time for Through movement of the South Approach of Chardobato intersection was found from field observation of the peak hour to be 15 seconds. The cycle time was also found to be 115 seconds.

The Saturation flow rate was 1621 veh/hr as shown in the observation of Basic saturation flow rate at the start of this section.

The Capacity of the movement is found to be:

ci = si $\frac{gi}{C}$ = 1621 * (15/115) = 211.435 veh/hr

The flow rate observed was vi = 59 veh/hr.

The degree of saturation was then calculated to be:

$$X_i = \left(\frac{v}{c}\right)_i = \frac{v_i}{s_i\left(\frac{g_i}{C}\right)} = \frac{v_i C}{s_i g_i}$$

= (59*115)/ (1621*15) = 0.279

Since, the intersection in the upstream signal of south approach in non-existent, the progression of traffic is considered random and hence Progression Factor (PF) =1.

g/C = 15/115 = 0.13

Now,

DETERMINATION OF DELAY

d = d1(PF) + d2 + d3

where,

d = control delay per vehicle (s/veh);

d1 = uniform control delay assuming uniform arrivals (s/veh);

PF = uniform delay progression adjustment factor, which accounts for effects of signal progression;

d2 = incremental delay to account for effect of random arrivals and oversaturation queues, adjusted for duration of analysis period and type of signal control; this delay component assumes that there is no initial queue for lane group at start of analysis period (s/veh); and

d3 = initial queue delay, which accounts for delay to all vehicles in analysis period due to initial queue at start of analysis period (s/veh)

(HCM 2000)

$$d_1 = \frac{0.5C\left(1-\frac{g}{C}\right)^2}{1-\left[\min(1,X)\frac{g}{C}\right]}$$ = 45.1205255 seconds

Where,

d1 = uniform control delay assuming uniform arrivals (s/veh);

C = cycle length (s); cycle length used in pretimed signal control, or average cycle length for actuated control (see Appendix B for signal timing estimation of actuated control parameters);

g = effective green time for lane group (s); green time used in pretimed signal control, or average lane group effective green time for actuated; and

X = v/c ratio or degree of saturation for lane group.

(HCM 2000)

$$d_2 = 900T\left[(X-1)+\sqrt{(X-1)^2+\frac{8kIX}{cT}}\right]$$ = 3.26226187 seconds

Where,

d_2 = incremental delay to account for effect of random and oversaturation queues, adjusted for duration of analysis period and type of signal control (s/veh); this delay component assumes that there is no initial queue for lane group at start of analysis period;
T = duration of analysis period (h);
k = incremental delay factor that is dependent on controller settings;
I = upstream filtering/metering adjustment factor;
c = lane group capacity (veh/h); and
X = lane group v/c ratio or degree of saturation.

(HCM 2000)

Neglecting the initial queue in the approach, d3 = 0 seconds

Hence, d = d1*PF + d2 + d3 = 48.38279 seconds

Also, di * vi = 59 * 48.38279 seconds = 2854.58446

Sum of (di * vi) of all movements in the south approach = 8819.0961

Now, Aggregated Delay:

$$d_A = \frac{\sum d_i v_i}{\sum v_i}$$

= 22.6711982 seconds per vehicle

where

d_A = delay for Approach A (s/veh),
d_i = delay for lane group i (on Approach A) (s/veh), and
v_i = adjusted flow for lane group i (veh/h).

Finally, from Los of service criteria (HCM 2000)

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

Since D(A) = 22.6 seconds per vehicle < 35 seconds per vehicle, the level of service is LOS C

Similarly, LOS of all other approaches were found as:

CHARDOBATO WEST														
G	C	Approach	Si	ci	Vi	Xi	PF	g/C	min of (1,X)	d1	d2	d3	delay sum (di)	vi*di
115	115	Left	1371	1371	81	0.059081	0.9	1	0.059081	0	0.08242251	0	0.082422508	6.67622317
45	115	Through	4875	1907.609	616	0.322917	0.9	0.39130435	0.322917	24.3856998	0.44935847	0	22.39648833	13796.2368
25	115	Right	1249	271.5217	255	0.939151	0.9	0.2173913	1	45	54.618555	0	95.11855499	24255.2315
					952								sum di*vi	38058.1446
													LOS D	39.9770426

TINDOBATO EAST														
G	C	Approach	Si	ci	Vi	Xi	PF	g/C	min of (1,X)	d1	d2	d3	delay sum (di)	vi*di
45	100	Through	6246	2810.7	1018	0.362187	0.9	0.45	0.362187	18.0701511	0.36320259	0	16.62633859	16925.6127
20	100	Right	1251	250.2	176	0.703437	0.9	0.2	0.703437	37.2390698	15.3084743	0	48.82363708	8592.96013
					1194								sum di*vi	25518.5728
													LOS C	21.372339

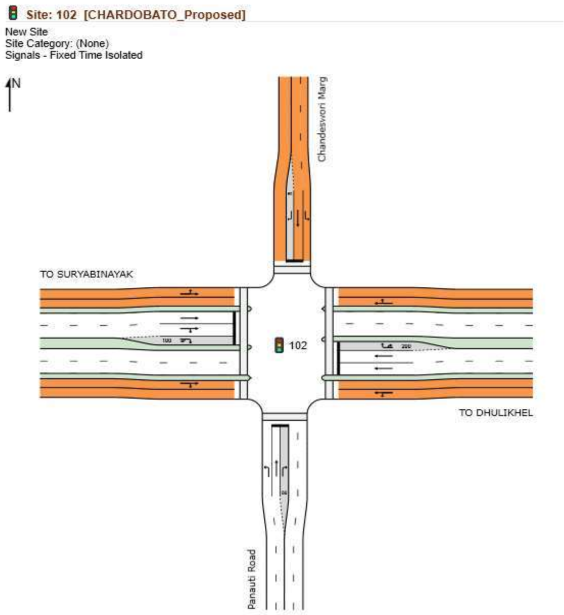
TINDOBATO NORTH														
G	C	Approach	Si	ci	Vi	Xi	PF	g/C	min of (1,X)	d1	d2	d3	delay sum (di)	vi*di
100	100	left	1574	1574	121	0.076874	1	1	0.076874	0	0.09521119	0	0.095211192	11.5205543
25	100	Right	1261	315.25	59	0.187153	1	0.25	0.187153	29.5055116	1.30994487	0	30.81545652	1818.11193
					180								sum di*vi	1829.63249
													LOS B	10.1646249

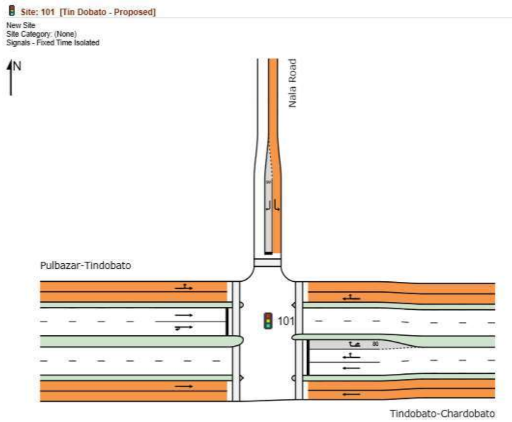
Then, Delay for the intersection as a whole is given as:

$$d_I = \frac{\sum d_A v_A}{\sum v_A}$$

The intersection layout can be further improved to accommodate U-turn flow in major approaches as well as retain the geometric pattern as in the existing Araniko highway of Koteshwor-Suryabinayak Section. The layout of such intersection is shown below:Layout of final signalized intersection in SIDRA:

Figure 4.4.21 Proposed Site Layout of Chardobato and Tindobato





The network model of SIDRA is shown below:

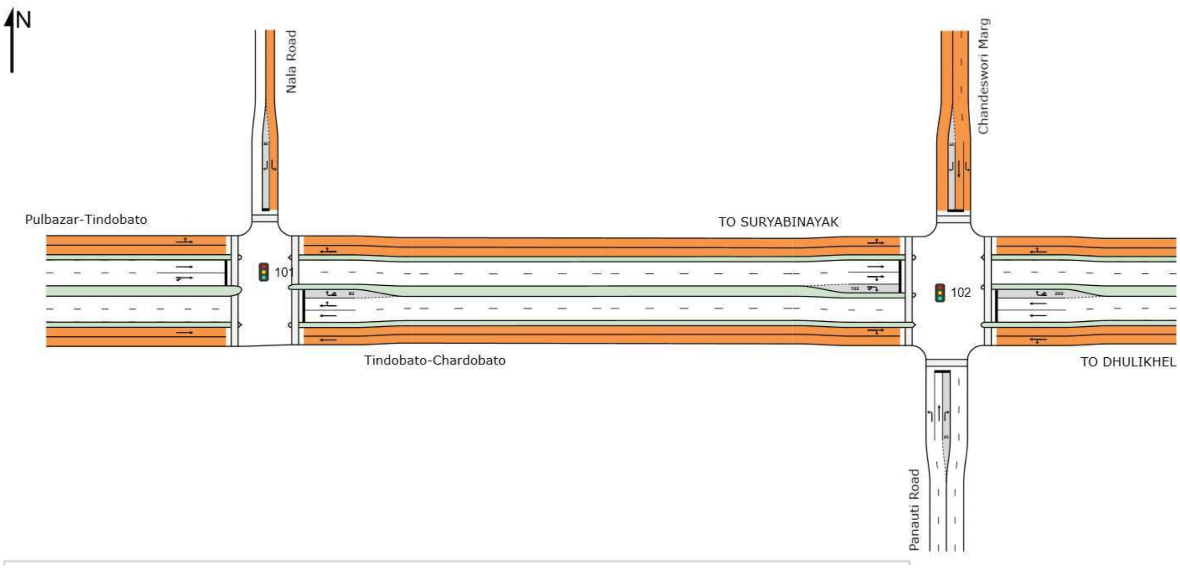


Figure 4.4.22 Network Model of SIDRA Intersection

The proposed case shows significant improvement in the Level of Service of the intersection compared to that of the existing unsignalized case. In the existing case, delay was found to be high and queue length was of about 72 meters which after the design of signal with respect to phase time, it has reduced to 54m in Chardobato intersection. The level of service for Tindobato intersection has improved to LOS B and for Chardobato intersection to LOS D.

Because of restriction on size of the report and to enhance comprehensiveness only distinctive part of analysis is only presented here.

Detailed output of sidra with all remaining results are given in in Appendix J.

4.5 Parking Supply Survey

4.5.1 Off-street parking

The final plot map showing location of the Off- Street parking facilities with images of access the facility is shown by using GIS map print features. Also, description of place like name of street capacity of facility with landmark as well as area is shown in item description section. It also describes either the facility is paid or unpaid. Each map shows plot location on map two photos with direction of north.

Figure 4.5.1 Sample of Off-Street Parking in Banepa

Because of restriction on size of the report and to enhance comprehensiveness a sample of analysis is only presented here.

Detailed in: -

Appendix F- off street parking

4.5.2 On-street parking

The final plot map showing location of the On- Street parking facilities with images of access the facility is shown by using GIS map print features. Also, description of place like name of street, capacity of facility with landmark as well as area is shown in item description section. It also describes either the facility is paid or unpaid. Each map shows plot location on map two photos with direction of north.

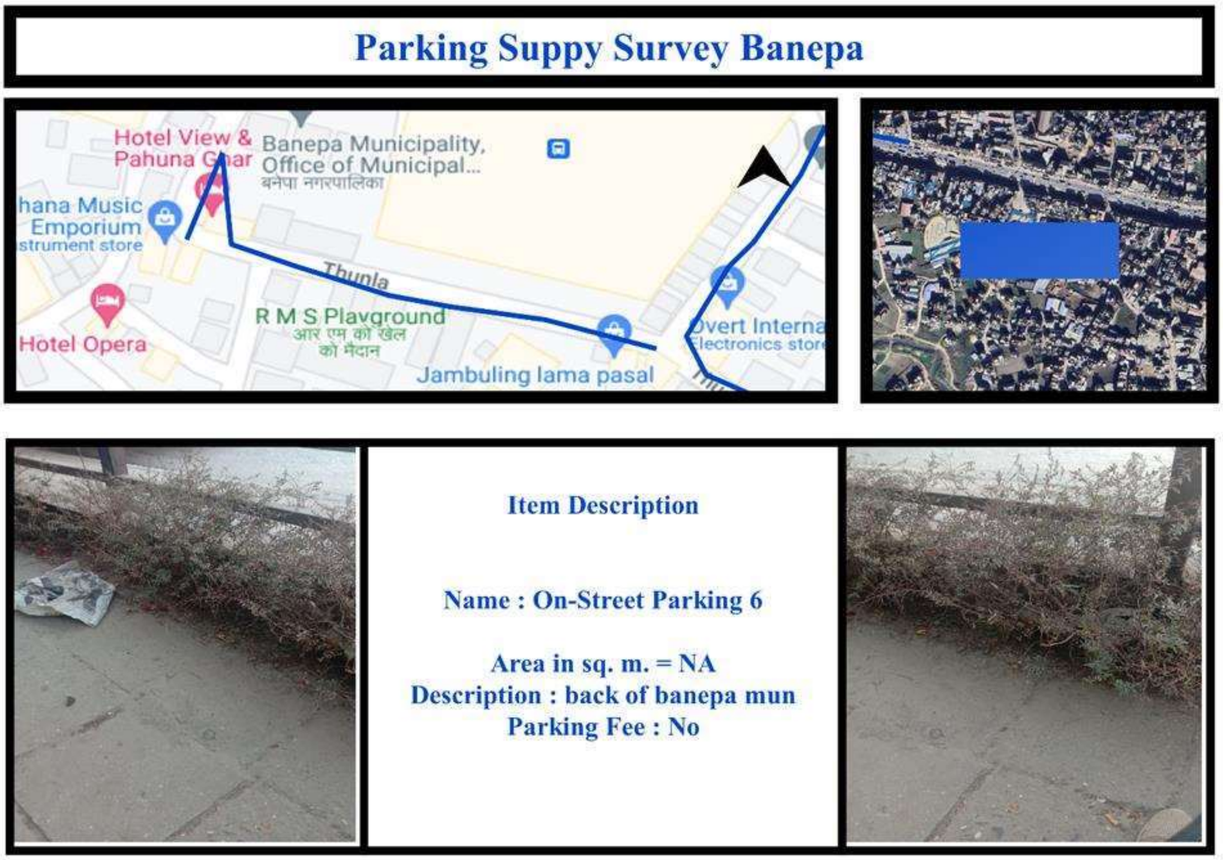


Figure 4.5.2 Sample of On-Street Parking in Banepa

Because of restriction on size of the report and to enhance comprehensiveness a sample of analysis is only presented here.

Detailed in: -

Appendix G - On street parking

4.6 Design review and overlay

The plan of cad file of Proposed intersection with its element is given below in two sections: -

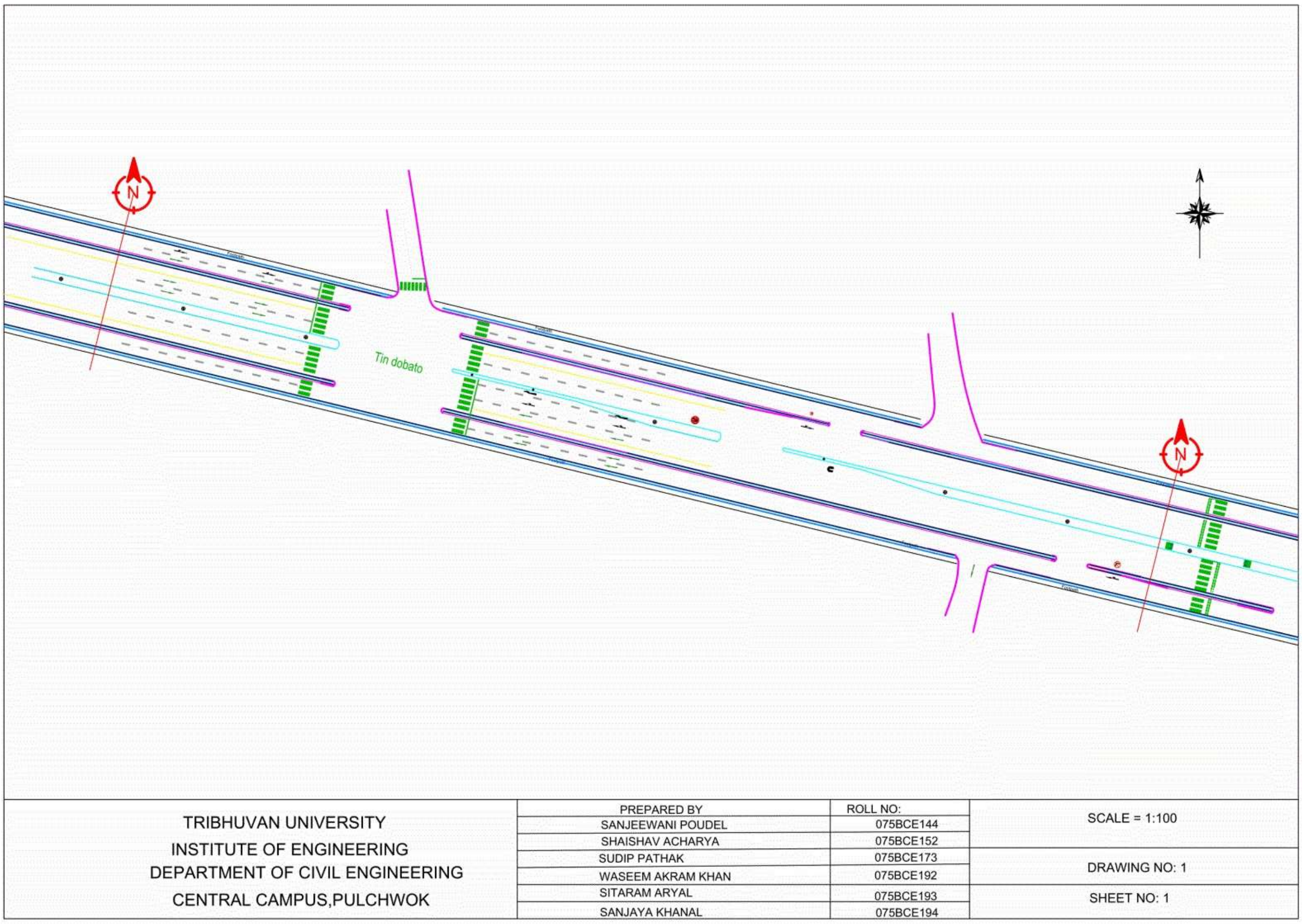


Figure 4.6.1 Proposed AutoCAD Plan for Tindobato Section

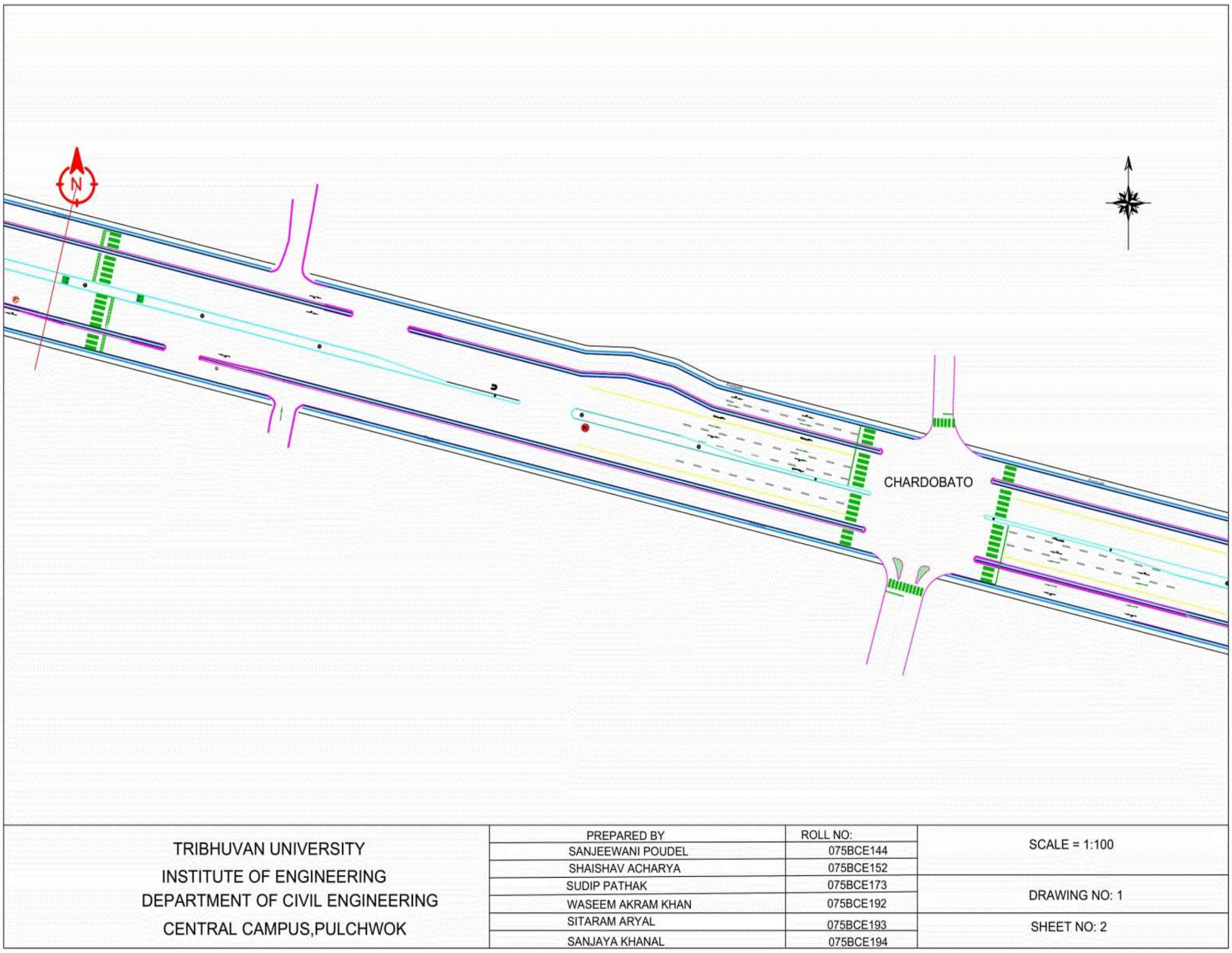


Figure 4.6.2 Proposed AutoCAD Plan for Chardobato Section

A representative model of six laning features of Araniko highway is given below:

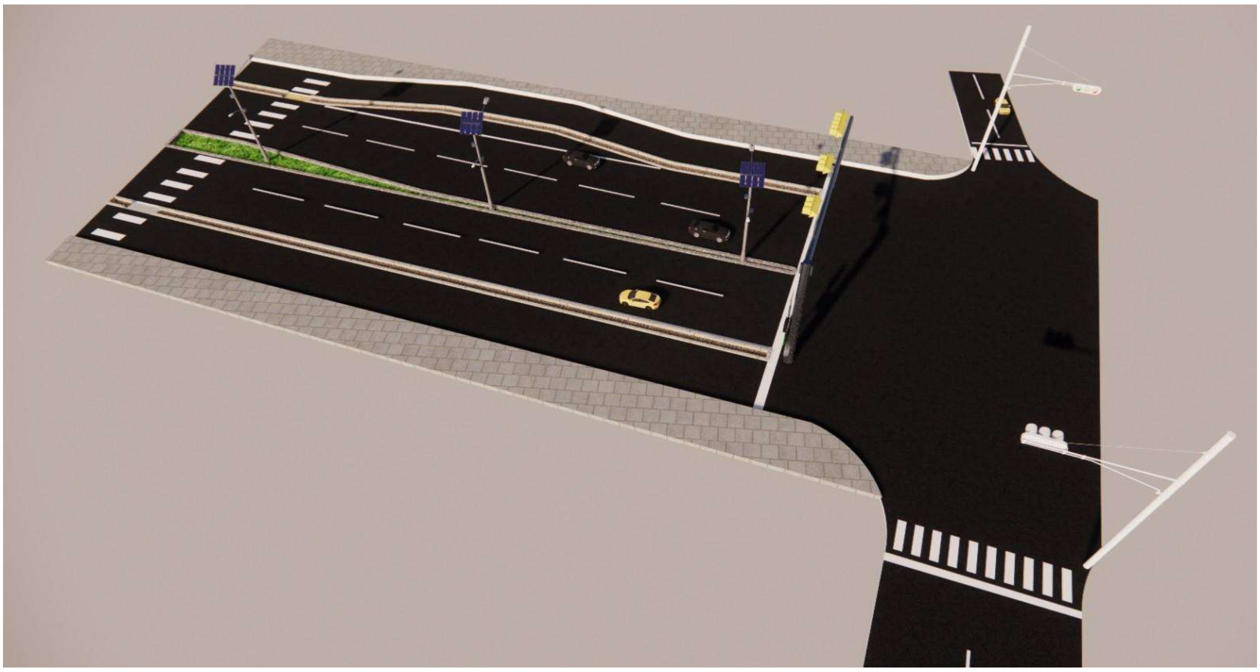


Figure 4.6.3 3D Sample Image of West Approach of Chardobato Intersection made using Sketchup

Detailed AutoCAD and Sketch Up file are attached with the report.

4.7 Rate Estimate of Proposed Works

The following is the rate estimate of Street Lighting and Road Markings as proposed in this project and as shown in the CAD files (Figure 4.6.1) and Sketchup image (Figure 4.6.3) :

For Lighting

Table 4.7.1 Rate Analysis for Single-Mounted Street Lighting

Street lighting (for 1 lamp)-Service lanes- Single mounted						
	Materials	Quantity	Unit	Rate (Rs.)	Total (Rs.)	Remarks
A. Materials						
1	Sodium Vapor lamp	1	nos.	875.7	875.7	(+)5%
2	Steel circular hollow pole	1	nos.	12979	12979	
			Subtotal A		13854.7	
B. Manpower						
1	Skilled	1	nos.	1090	1090	
2	Unskilled	1	nos.	800	800	
			Subtotal B		1890	
			Total (A + B)		15744.7	
C. Hire of tools and plants @ 3% of unskilled labour		3% of unskilled			24	
			Total (A+B+C)		15768.7	
D. Contractors overhead and profit @ 15% of total (A+B+C)		15% of (A+B+C)			2365.305	
			Grand total (A+B+C+D)		18134.005	

For 100 m stretch
Total quantity of lamp with spacing of 40 m = 2.5
Quantity = 2.5

Cost = 45335.0125

Total Cost= 2.5*18134.01= NRs. 45335.013

For two Service lanes
Cost=NRs. 90670.025

The total cost for lighting is 15* 90670 = NRs. 13,60,050

Table 4.7.2 Rate Analysis for Double-Mounted Street Lighting

Street lighting (for 2 lamp)-Freeway lanes- Double mounted						
	Materials	Quantity	Unit	Rate(Rs.)	Total(Rs.)	Remarks
A. Materials						
1	Sodium Vapor lamp	2	nos.	875.7	1751.4	(+)5%
2	Steel circular hollow pole	1	nos.	12979	12979	
			Subtotal A		14730.4	
B. Manpower						
1	Skilled	1	nos.	1090	1090	
2	Unskilled	1	nos.	800	800	
			Subtotal B		1890	
			Total (A + B)		16620.4	
C. Hire of tools and plants @ 3% of unskilled labour		3% of unskilled			24	
			Total (A+B+C)		16644.4	
D. Contractors overhead and profit @ 15% of total (A+B+C)		15% of (A+B+C)			2496.66	
			Grand total (A+B+C+D)		19141.06	

For 100 m stretch
Total quantity of lamp with spacing of 40 m = 2.5
Quantity = 2.5
Cost = 47852.65
Total Cost= 2.5*18134.01= NRs. 47852.65

The total cost for lighting is 15* 47852.65 = NRs. 7,17,789.75

For Road Markings

Marking of Road (painting Lines, Dashes, Arrows) For 10 sqm						
	Materials	Quantity	Unit	Rate (Rs.)	Total (Rs.)	Remarks
A. Materials						
1	Road marking Paint	1.48	Lit	399	590.52	
			Subtotal A		590.52	
B. Manpower						
1	Skilled	1	nos.	1090	1090	
2	Unskilled	2	nos.	800	1600	
			Subtotal B		2690	
			Total (A + B)		3280.52	
C. Hire of tools and plants @ 3% of unskilled labour		3% of unskilled			48	
			Total (A+B+C)		3328.52	
D. Contractors overhead and profit @ 15% of total (A+B+C)		15% of (A+B+C)			499.278	
			Grand total (A+B+C+D)		3827.798	

For 1500 m stretch per 10 sqm,
Total quantity of marking = 1374.4 sq. m.
Total quantity of marking per 10 sq. m= 137.44 sq. m.
Total Cost= NRs. 5,26,092.6

Retro-reflective beads for 10sqm,
Quantity of Road marking = 1.48 lit
Quantity of Retro-reflective bead = 0.8 kg/litre
Quantity of Retro-reflective bead = 1.184 kg
Rate per kg = Rs. 157

For 1500 m stretch
Total quantity of reflector beads marking = 384.407 m2
Total quantity of reflector beads marking per 10 sqm = 38.4407 m2
Total Quantity of Retro-reflective bead per 10 sqm= 45.5137888 kg
Total Cost of Retro-reflective bead NRs = Rs 7145.664842
Grand Total Cost = 526093 + 7146 = NRs. 5,33,239

5 Conclusion and Recommendations

Conclusions

This project was carried out to conduct review of detailed engineering design document of Punyamata-Chandeshwori section of Araniko highway.

The major problem of the study section was the water logging problem, which was often mixed with sewage, during flash floods in the river. Hence, hydrological analysis of the proposed bridge section at Pulbazar, Chardobato, and Chandeshwori were carried out, the results of which are the 100-year flood passing through these bridges, found to be 139.805m^3/s, 22.89m^3/s, and 50.55m^3/s, respectively, which are all greater than the discharge values of 115.87 m^3/s, 15.83 m^3/s, and 30.03 m^3/s as found in the design document.

It is worth mentioning that the analysis in the previous design document of the section was found to consider only two nearby rainfall stations whereas this study considered three rainfall stations namely Dhulikhel (1024), Nagarkot (1043) and Nangkhel (1082).

Further, the existing document only considered the ROW of the highway section as the catchment area for the analysis of side drain in comparison to which this project considered the entire catchment area which includes the runoff from the settlement upstream of the section.

The clear depth of the bridge at Pulbazar Side drains located between Pulbazar and Chandeshwori were designed considering hydrological factors like 50-year flood passing through these side drains and thus its cross section was computed. Since these are side drains, the effect of sewage was not considered.

Apart from the hydrology of the study section, the safety aspects of the design were also reviewed. While the design document mentioned few safety features such as highway lighting, road delineation, etc. in brief, the specifics of the analysis as well as the detailed recommendation however was absent in the provided document. This might have been due to the fact that the document obtained at the time of data collection was in the stage of drafting and further refinement might have added such details. Hence, in the absence of such details, independent analysis was carried out to review the overall safety aspects of the highway section. Firstly, crash analysis of the study section was done using crash records obtained from Banepa traffic police, results of which were the crash hotspots of the region in the form of clusters. These clusters were analyzed to obtain possible issues in those areas mainly from the Vulnerable Road User's (VRU) perspective. These results were further validated using the iRAP Star rating analysis conducted in the same section. The mitigation measures which had the most positive impact validated from both the analysis were found to be highway lighting, road marking and raised pedestrian crossing. Also, cost estimations of the proposed measures were provided where applicable.

Another notable aspect of the design document was the fact that the three-legged Tindobato intersection was not considered as a proper intersection in the design document. Proper intersection treatment in the intersection was found to be essential as Tindobato was important from business perspective, the negligence of which could produce significant economic impact to the revenue of the region.

The existing intersection control was then analyzed for its adequacy to cope with safety and traffic flow challenges. For this, traffic flow data was collected and MUTCD signal warrant guideline, considering traffic and crash, was analyzed which showed the requirement of signal in both the intersections including the Tindobato intersection.

Therefore, signal design was also carried out for both the intersections of the section using SIDRA intersection. The output from the SIDRA Intersection was further tested with manual calculation. The output of Level of Service and the Queue length from SIDRA was verified with Field Observation of Queue length. The proposed case showed significant improvement in the Level of Service of the intersection compared to that of the existing unsignalized case. In the existing case, delay was found to be high and queue length was of about 72 meters which after the design of signal with respect to phase time, it was reduced to 54m in Chardobato intersection. The level of service for Tindobato intersection was improved to LOS B and for Chardobato intersection to LOS D.

Finally, the design document did not factor in the displacement of parking from the highway section which needs to be accommodated elsewhere. Therefore, parking supply survey was also carried out to fill this gap from which it was concluded that ample supply near the highway section is available.

Recommendation

The following are the measures which need to be immediately considered to solve the issues in the study section:

1. Firstly, the bridges of cross-sectional dimensions of width 25 m, 5m, 5m and clear waterway depth 5 m, 5m, 3.5 m are recommended at Pulbazar, Chardobato, and Chandeshwori, respectively.
2. The depth of side drains greater than 450mm should be covered which can be used as extension of the footpath or as a space to accommodate cycle tracks in the section.
3. The river width needs to be maintained by removal of illegal encroachments as well as the siltation at the river banks routinely. Application of bioengineering techniques and principles should be carried out making proper policies for landslide prevention and reducing silt deposition in river basin.
4. The sewer system should be maintained routinely until complete overhaul of the sewer system takes place.
5. The alignment of cross-drain at the water-logging prone Pulbazar area should be through the southern settlement towards the river. However, it should be ensured that the outlet of both side drain and cross drain should run parallel to the river in the river corridor for a certain distance and discharged into the river at an angle to prevent backflow.
6. In order to prevent flood from breaching into the settlement areas, embankment wall around river alignment should be constructed. Further, demolition of damaged bridges needs to be carried out that are acting as bottlenecks in the stream flow.
7. Road Markings, Lightings and Raised pedestrian crossings should be included in the highway expansion. Further, the maintenance of these safety measures in the road section should be undertaken to ensure operation at all times.
8. Removal of roadside and footpath encroachment should be done to ensure sufficient space for pedestrian movement.
9. Parking supply in the form of Off-Street Parking should be provided for the displaced parking demand and proper system should be made to monitor and manage its use.

The following measures are recommended to solve these problems in the long run:

1. Various locations of Soil erosion should be identified and necessary measures like check dams, bio-engineering, etc. should be employed. Further, construction activities that include excavation should be monitored for transportation of silt in the river.

2. The sewer system needs a complete overhaul and a new separate stormwater and sewage system should be designed for the municipality.

3. The runoff into the river should be delayed as much as possible. Proper Land Use Planning including the adoption of Green Stormwater Infrastructure (GSI) which includes longitudinal swales, artificial ponds, soak ways could be employed in the Highway design as well as in the entire catchment area of the river. Measures like rainwater harvesting could also prove useful in the same. These measures reduce the immediate impact of the flood while also improving the aesthetics and economy of the municipality. Alternative solutions like Storage Dams and Levees are expensive and less effective respectively.

4. Detailed analysis of overall design document should be carried out addressing all safety and drainage requirement for safe operation of traffic in highway section

5.1 Remedial Measure in terms of Highway Safety

- Roads should be designed and maintained to ensure good visibility, proper drainage.
- One main cause of crash is over speeding, so installed speed limit signs, installed speed breaker near the intersection and also bumper strip in shoulder to alert the driver during night for smooth flow on main lane.
- Drivers should provide proper education and training on road safety rules and regulation.
- The Crash data should be transferred to a computer database to establish a reliable, well managed and effective implementation of the Central Road Crash Database System.
- Installed collision warning system, visual information should provide entrance of Crash prone zone for alert drivers.

- Channelization, wide median, slip lane for right turning vehicle, for lane separation divider/barriers should provide for smooth flow and minimize collision.
- Optimization of signal timing, encouraging public transport systems.
- Public awareness campaigns, enforcing strict traffic laws and improving the licensing process which help reduce road Crash.
- Well managed emergency rescue and first aid center installed near the prone crash area.

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7 APPENDICES

7.1 **Appendix A - Meteorological Data**

7.2 **Appendix B - Design flood estimation in side drain**

7.3 **Appendix C - Traffic Volume Count of Chardobato**

7.4 **Appendix D - Traffic Volume Count of Tindobato**

7.5 **Appendix E - Crash record of Banepa**

7.6 **Appendix F - Off street parking**

7.7 **Appendix G - On street parking**

7.8 **Appendix H - iRAP results**

7.9 **Appendix I - Detailed input in SIDRA Intersection**

7.10 **Appendix J - Detailed output in SIDRA Intersection**

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