PLANNING CRITERIA FOR INTERCHANGES

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ABSTRACT

PLANNING CRITERIA FOR INTERCHANGES

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When two or more highways cross each other, an intersection is formed. Depending upon the traffic volume of crossing highways; some arrangements may be necessary on the intersection to provide the traffic flow without any problem. Some of the problems may be named as reduction on vehicular speed resulting in delays, decrease of capacity, increase in accidents and increase in cost of operation. The above mentioned problems become more important when the crossing roads are high standard highways such as freeway. In order to avoid from such problems, proper intersection types should be selected by the planners. In this study the various types of intersections are throughly examined with possible uses in relation with the traffic and operation, site conditions, safety and economy. Planning criteria were applied as a case study to an existing interchange on a freeway.

The application of grade-separated intersections on urban roads were also studied. İsmet İnönü Intersection was chosen as the case study to apply the planning criteria of an urban GSI. For this intersection traffic volume counts and signal timing were obtained from EGO and TŞM respectively which were used with actual field measurements. Capacity, level-of-service and economic evaluation were made and the benefits derived through such an investment were evaluated.

Key words: Transportation, Intersections, Interchanges, Economic Evaluation.

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ÖZ

ÇOK KATLI KAVŞAKLARIN PLANLAMA KRİTERLERİ

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Kavşaklar iki veya daha çok sayıda yolun kesişmesi ile oluşurlar. Bu kavşak noktalarında trafik akışının problemsiz olarak sağlanması için, kesişen yollardaki trafik hacmine bağlı olarak bazı düzenlemeler gerekli olabilir. Araç hızlarındaki düşme ve bunun sonucu ortaya çıkan gecikmeler, kesişen yolların kapasitelerinin azalması, kaza olasılığının yükselmesi ve taşıt işletme giderlerinin artışı kavşaklarda ortaya çıkabilecek belli başlı problemlerdir. Kesişen yolların, otoyollar gibi yüksek standartlı olması, ortaya çıkabilecek problemleri daha önemli kılar. Kavşak tipinin doğru seçimi ise, ortaya çıkabilecek problemleri azaltır. Bu amaçla bu çalışmada kavşak tipleri ile ilgili geniş bir araştırma yapıldı. Çok katlı kavşakların tipleri ve uygulanabilirlikleri; trafik, arazi şartları, emniyet ve ekonomik faktörler açısından incelendi. Planlama kriterleri otoyol üzerinde bir katlı kavşak örneği ile desteklenerek açıklandı. Katlı kavşakların kent içindeki yollarda uygulanışları araştırıldı. Çok katlı kavşakların planlama kriterlerinin kent içindeki uygulaması İsmet İnönü Kavşağı üzerinde yapıldı. Kavşaktaki araç sayımları EGO'dan, sinyalizasyon şeması ise Trafik Şube Müdürlüğünden temin edildi. Diğer gerekli ölçümler ise çalışmalar sırasında arazide gerçekleştirildi. İsmet İnönü Kavşağı için yapılan kapasite, hizmet seviyesi ve ekonomik analizler sonucunda bu uygulamadan elde edilen faydalar araştırıldı.

Anahtar Kelimeler: Ulaşım, Kavşaklar, Çok Katlı Kavşaklar, Ekonomik Değerlendirme.

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AADT	Average annually daily traffic
C	Cycle length
c_i	The capacity of lane group or approach
d	Average delay per vehicle for the lane group
d_1	First term delay
d_2	Second term delay
d_A	Delay for approach A
d_i	Delay for lane group i on approach A
d_I	Average delay per vehicle for the intersection
f_a	Adjustment factor for area type
f_{bb}	Adjustment factor for the blocking effect of local buses stopping within the intersection area
f_g	Adjustment factor for approach grade
f_{HV}	Adjustment factor for heavy vehicles
f_{LT}	Adjustment factor for left turns in the lane group
F_n	The n^{th} year annual benefits
f_p	Adjustment factor for the existence of a parking lane adjacent to the lane group
f_{RT}	Adjustment factor for right turns in the lane group
f_w	Adjustment factor for the width
g	Effective green time
g/C	Green ratio for a lane group or approach
G_p	Minimum green time
HDT	Hourly design traffic
i	Reduction rate
I_n	The n^{th} year annual investment
L	Lost time
LT	Left
m	The termination year of the investments
n	Useful life
N	Number of lanes in the lane group
pc	Passenger car
pcph	Passenger car per hour

pcphgpl	Passenger car per hour of green per lane
PF	Progression factor
PHF	Peak hour factor
P_{LT}	The proportion of left turns in lane group
P_{RT}	The proportion of right turns in lane group
PTG	Percentage of the cycle length that is green for the movement
PVG	Percentage of all vehicles in the movement arriving during the green phase
R_p	Platoon ratio
RT	Right
r	Rate of return
s	Saturation flow rate for the subject lane group
s_o	Ideal saturation flow rate per lane
TH	Through
U	Lane utilization factor
V	Hourly volume
v_A	Adjusted flow for approach A
v/c	The ratio of flow rate to capacity
v_g	Lane group flow rate
v_i	Adjusted flow rate for the lane group i
v_{LT}	Left-turn flow rate
v_p	Flow rate during peak 15 min. period
v_{ph}	Vehicle per hour
v_{RT}	Right-turn flow rate
$(v/s)_{ci}$	The flow ratio for critical lane group
w	Distance from the curb to the nearest pedestrian refuge island
x	The ratio of flow rate to capacity
x_c	The critical (v/c) ratio
Y	Change interval

ABBREVIATIONS

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway Transportation Officials
AGI	At-Grade Intersection
ALT	Alternative
B/C	Benefit Cost Ratio
CE	Civil Engineering
EB	East Boundary
EGO	Elektrik Gaz Otobüs (General Directorate of Electricity, Gas and Bus Service of Ankara)
GAZİKARÖ	Gazi Üniversitesi Kazaları Araştırma Enstitüsü (Gazi University, Institute of Traffic Accident Research)
GSI	Grade-Separated Intersection
HCM	Highway Capacity Manual
IIP	International Incorporation Parsons
IRR	Internal Rate of Return
KGM	Karayolları Genel Müdürlüğü (General Directorate of Turkish State Highways)
LOS	Level-of-Service
NB	North Boundary
NPV	Net Present Value
METU	Middle East Technical University
SB	South Boundary
TEK	Türkiye Elektrik Kurumu (Turkish Electricity Authority)
TŞM	Trafik Şube Müdürlüğü (Branch Directorate of Traffic)
WB	West Boundary

CHAPTER I

INTRODUCTION

An intersection for overland traffic systems is defined as the area where two or more highways join or cross. A highway radiating from an intersection and forming part of it is called as an intersection leg. It is better to avoid from using intersections more than four legs.

The efficiency, safety, speed, cost of operation and capacity on a highway depend on the intersections on that highway. It is very important to use appropriate intersection wherever it is necessary.

Since delays generally occur on intersections and it is also delays which determine the level-of-service of the highway, intersections are important components of highways and traffic circulation.

On an intersection, delays begin to occur when the traffic volume approaches capacity. The implication of these delays is not only the loss of time but also monetary loss.

There are three general types of intersection, intersections at grade, grade separation without ramps and grade-separated intersections. This study is generally focused on AGI and GSI. It is observed that grade-separated intersection and interchange has the same meaning in literature. Hence, in this study not only GSI but also interchange is used in various places.

A traffic interchange is a combination of ramps and grade separation at the intersection of two or more highways. Interchanges increase capacity, safety and reduce traffic conflicts.

There exists no strict rules or procedure for planning the intersections. AGI and GSI have different field of application. For example access to freeways should be made by GSI. On the other hand, interchange type determination is another problem which is effected from several factors out of which traffic, economical factors, site conditions and safety considerations are the most pronounced ones.

In Turkey, although each above-mentioned factor seems to have the same significant effect on deciding the interchange type; economic factors have frequently the strongest effect on the decision.

The aim of this study is; to put together the ways of planning interchanges, to make it clear where to use and what kind of interchanges should be used for different cases.

The study presents a brief summary of intersections. Then advantages and disadvantages of using interchanges are shown and the general principles of interchange planning adopted by KGM are summarized. Next, an "interchange alternative comparison" study which was made during the preliminary planning studies on Ankara ring road is explained. Meanwhile, the application of GSI on urban roads is searched. Finally the planning criteria of urban GSI is applied to an existing congested intersection as a case study and the benefits from the implementation of an urban GSI are briefly explained and discussed.

CHAPTER II

INTERSECTIONS

2.1. Classification of Roads

In order to provide the traffic flow properly, it is necessary to join roads within an order. Roads which carry fast and dense traffic should be connected to roads in neighbouring areas.

In order to understand the hierarchical order of roads, one should know the general classification of roads.

Roads are classified into four categories;

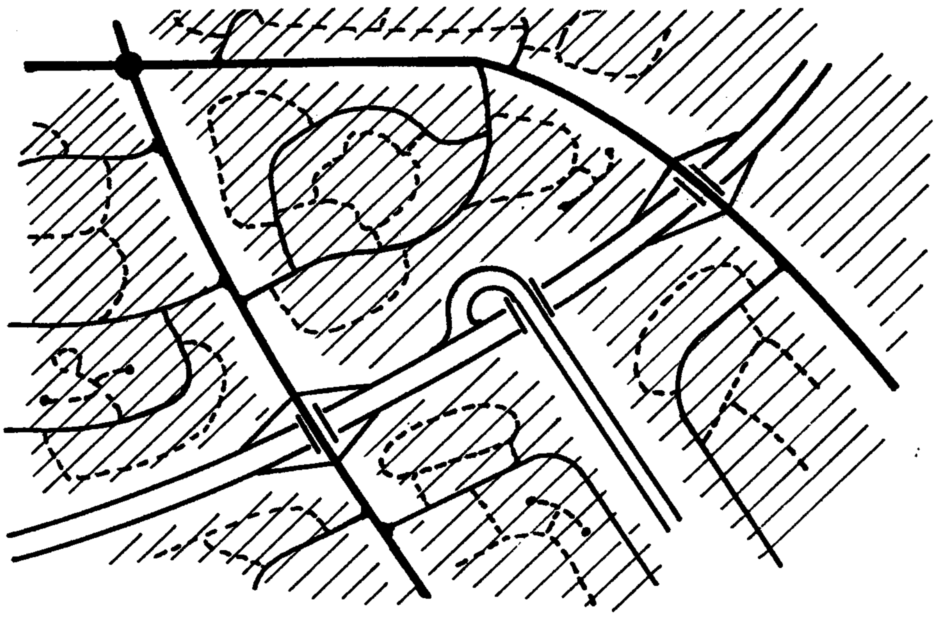
a) Primary distributor roads

b) District distributor roads

c) Local distributor roads

d) Access roads

Figure 2.1 is given to describe the differences between above-mentioned road classification by the aid of following definitions:



▬ Primary Distributors

▬ District Distributors

—— Local Distributors

--- Access Roads

/// Environmental Areas

Figure 2.1. Classification of Roads (7)

i) Primary Distributors: Roads between metropolitan areas should be linked to primary distributor roads. At-grade intersections, may cause congestions on these roads.

ii) District Distributors: These roads connect the primary distributors with districts. Districts are classified as industrial, business or administrative zones.

iii) Local Distributors: These roads distribute traffic within environmental areas and provide the link between district distributor with access road.

iv) Access Roads: Access to buildings are accomodated by these roads.

2.2. The Hierarchical Order of Roads

Through traffic on an area is the traffic which has an origin and destination out of this area. If the through traffic must use the local roads of a city, both the city traffic and the through traffic effect each other. Roads should be linked each other within an hierarchical order in order to prevent this influence.

Both the capacity of the road and that of the junctions on a road should be balanced. If the junction has a higher capacity than the intersected roads that would mean waste of money, meanwhile if the junction has inadequate capacity, than it will create bottleneck in traffic operations.

It is very important to plan junctions with sufficient capacity to meet the requirements also effecting its design; such as composition of traffic, speed, distribution and future growth of traffic. Where it is necessary to make some arrangements on a junction; signalization, channelization, over and under passes are necessary to meet the free flow of traffic.

2.3. Definition of Intersection

Intersection is the area where roads cross each other in order to provide changes in travel direction for defined destination. Each intersection has through or cross traffic movements on one or more of the highways. Turning movements between these highways can be implemented by many methods. The chosen method is a very important factor effecting the type of intersection. In general terms, intersections can be classified as shown in Fig.2.2.

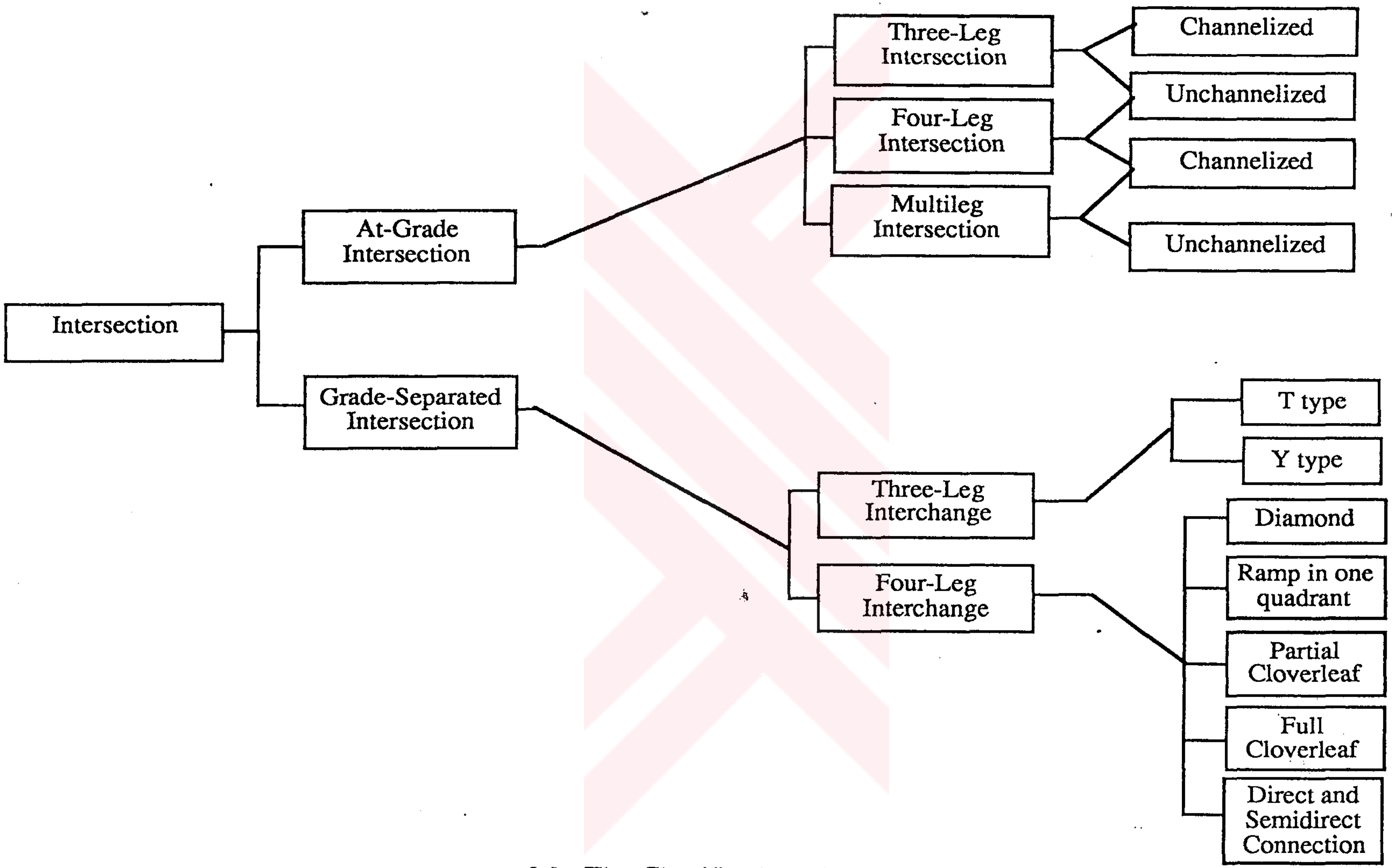


Figure 2.2. The Classification of Intersections

2.4. Types and Examples of At-Grade Intersections

By the use of intersections, the conflicts between pedestrian and vehicular flows decrease and the comfort of road users is improved.

The basic types of at-grade intersections are;

i) Three-Leg intersections

ii) Four-Leg intersections

iii) Multileg intersections (10)

For all these intersection types, different kinds of traffic control such as signalization or stop controls shall have to be implemented for traffic operations.

Although the traffic volume is not generally very heavy to verify the controls, limited sight distances may make it necessary to use stop signs or signals. On the other hand, the peak traffic volumes make it necessary to provide sufficient pavement widths and channelization[1] measures.

If the intersections are close to each other, certain turns may be prohibited at some intersections; but if they are spaced widely, all of the at-grade intersections need to provide crossing and turnings.

2.4.1. Three-Leg Intersections

Three-leg intersections have three intersecting legs. Intersections with three legs may be called as "T" intersection or "Y" intersection depending on which letter more nearly represents the general layout of the intersection as shown on Fig.2.3 (9).

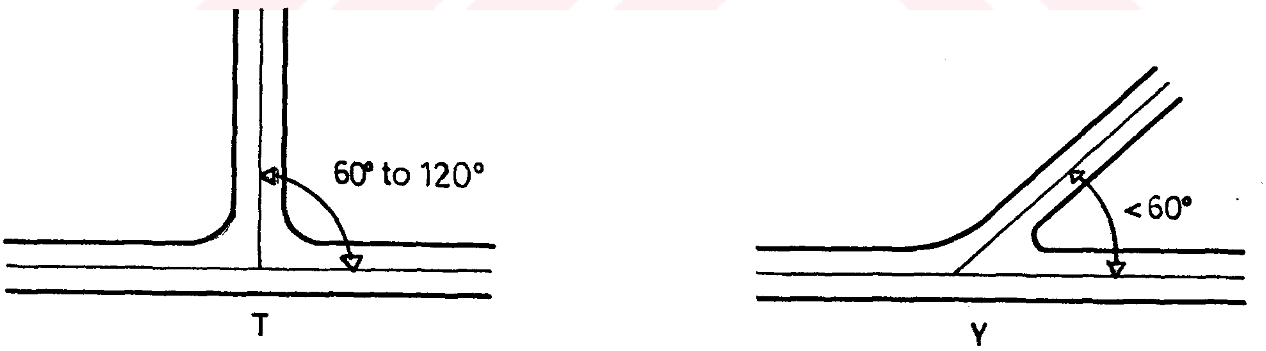
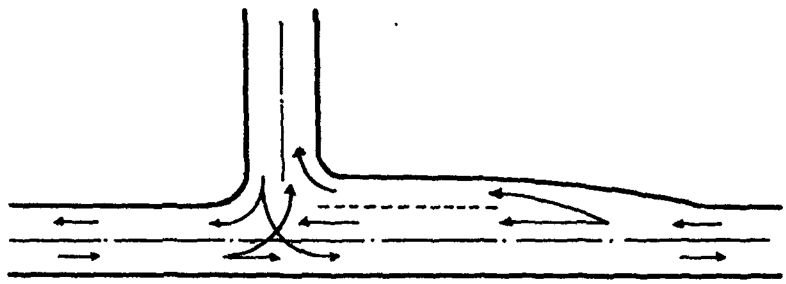


Figure 2.3. Three-Leg Intersections (9)

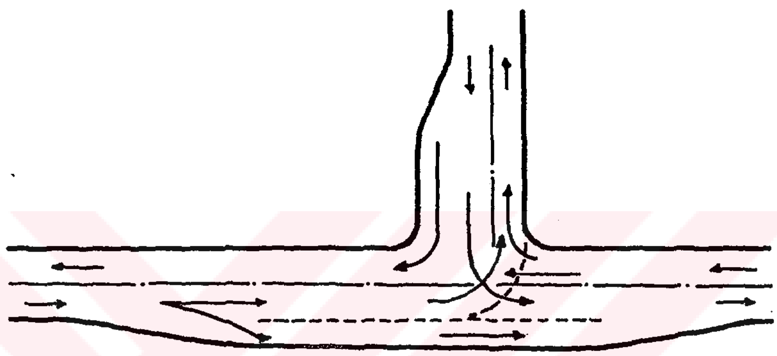
If the speeds are high and number of turnings are high, it is necessary to use flaring for increasing safety. Flaring is defined as the enlarged road section where the number of traffic lanes or the pavement width exceeds those of the normal section of highway.

In Figure 2.4(a), an intersection with auxiliary lane is shown. This auxiliary lane is useful for the right-turning and through traffic. Drivers who decide to turn right must use the added lane so that the through traffic shall not be significantly effected.

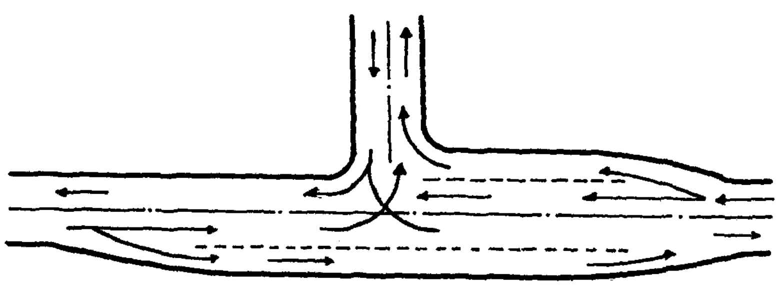
In Figures 2.4(b) and (c) different arrangements of flaring are shown. Figure 2.4(b) must be used where left-turn and through traffic are heavy. The through traffic can continue directly on its way without any deceleration by the use of added lane and the left-turn traffic can be slowed down for turning moments.



(a) "T" intersection with right-turn lane



(b) "T" intersection with right hand passing lane



(c) "T" intersection with right hand passing lane and right-turn lane

Figure 2.4. "T" Intersections with Auxiliary Lane (2)

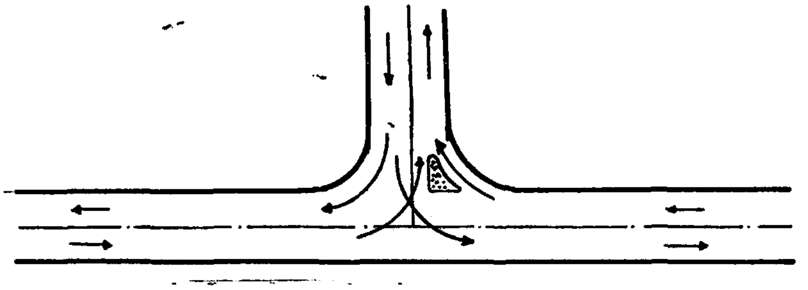
2.4.2. Channelized Three-Leg Intersections

In Figure 2.5 and 2.6, "T" intersections with islands are shown.

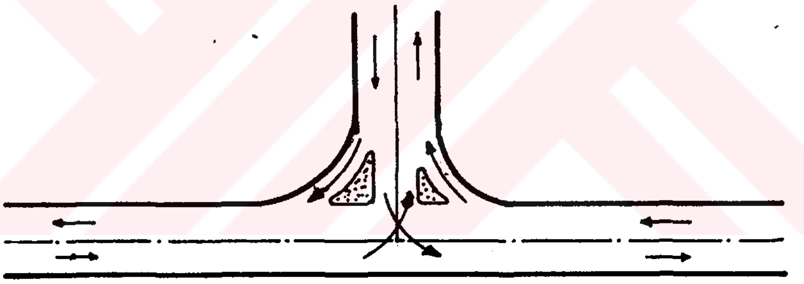
The islands used in Figures 2.5(a) and (b) are directional islands. Directional islands separate right-turning traffic from through traffic. By the help of these islands, right-turning vehicles complete their turnings easily and drivers feel more confident, on the other hand these directional islands do not have any effect on left-turning vehicles.

In Figure 2.6(a) and (b) "T" intersection with divisional and directional islands are shown. Divisional islands are used to separate streams of traffic travelling in the same or opposite direction.

Figure 2.6(b) is suitable arrangement for intersections carrying intermediate and/or heavy traffic flows.

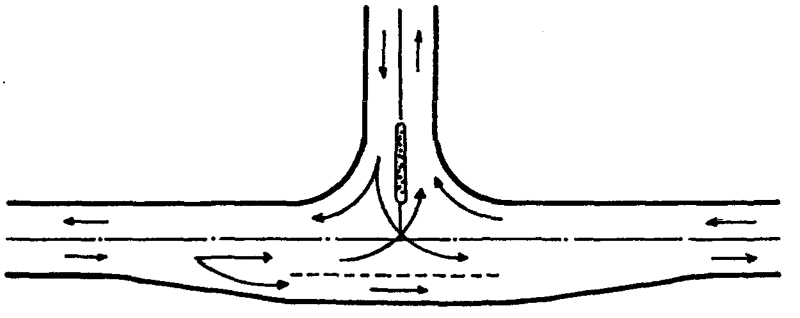


(a) with single turning roadway

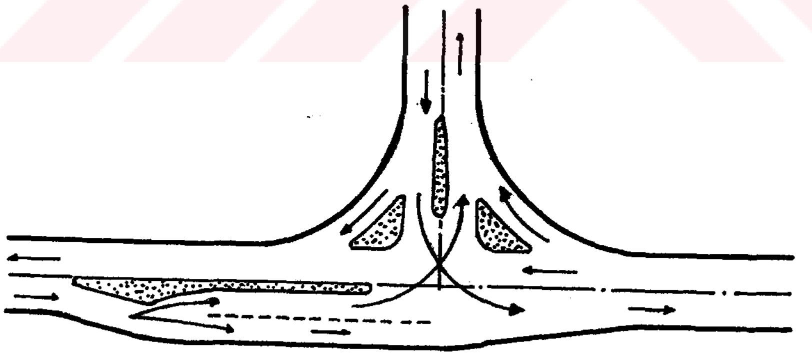


(b) with a pair of turning roadways

Figure 2.5. Channelized "T" intersections (2)



(a) with divisional islands and right passing lane



(b) with divisional island and turning roadways

Figure 2.6. Channelized "T" Intersections (2)

2.4.3. Four-Leg Interseections

Four-leg intersection is the intersection having four intersecting approach legs.

Divisional and directional islands can also be employed for four-leg intersections. Flaring may be applied to increase the capacity wherever necessary.

In Figure 2.7. the simplest form of four leg intersection is shown. This type may be applied where the intersecting legs carry light traffic volumes.

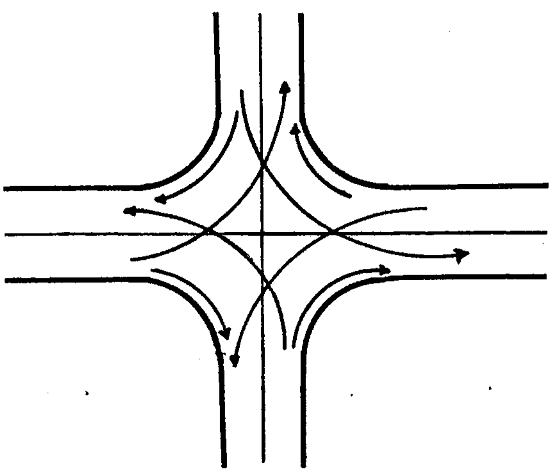


Figure 2.7. Unchannelized Four-Leg Intersection (2)

A typical four-leg intersection is shown in Figure 2.9. The reason of using auxiliary lanes is that one of the intersected highways has heavy through and turning traffic and needs additional capacity.

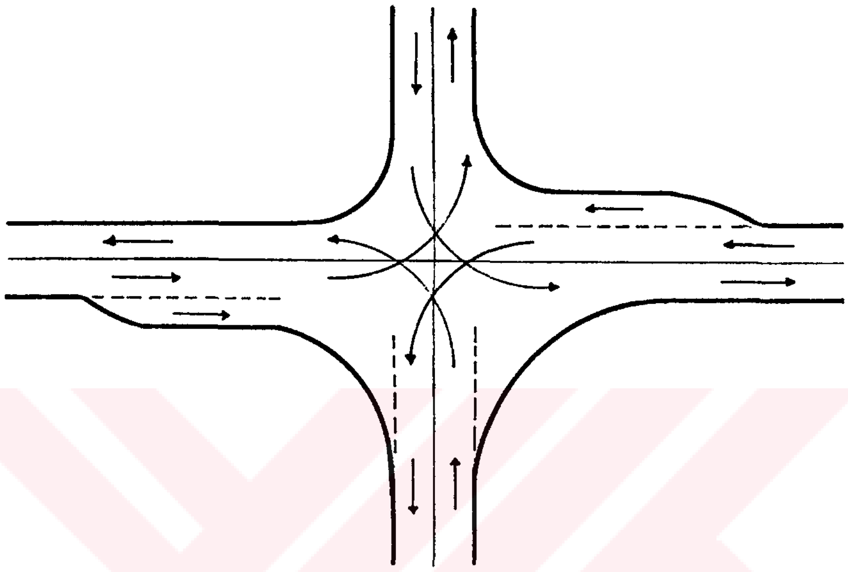


Figure 2.8. Unchannelized Four-Leg Flared Intersection (2)

2.4.4. Channelized Four-Leg Intersections

Figure 2.9 shows a four-leg intersection with directional islands. Since the right-turning traffic is heavy on one of the crossroads, directional island is used. These directional islands do not help to the left-turning traffic.

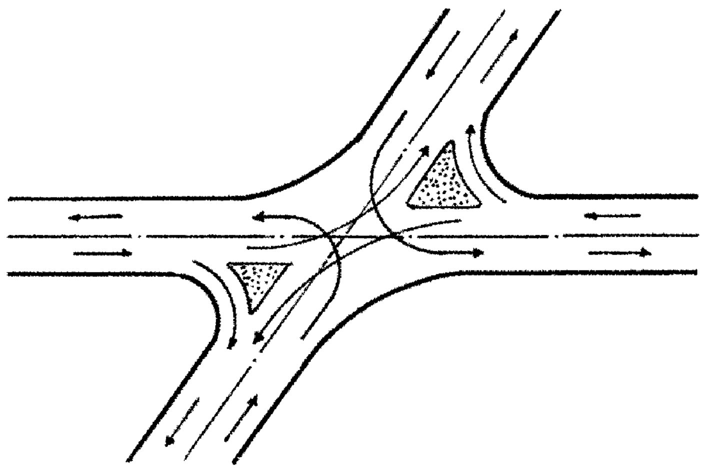


Figure 2.9. Channelized Four-Leg Intersection (2)

Figure 2.10 shows a typical four-leg intersection which carries heavy traffic with high speeds. The use of divisional and directional islands help the turning maneuvers and let drivers feel more confident.

2.4.5. Multileg Intersections

Multileg intersection is an intersection type having more than four intersecting approach legs.

Multileg intersection is preferable if some control devices as

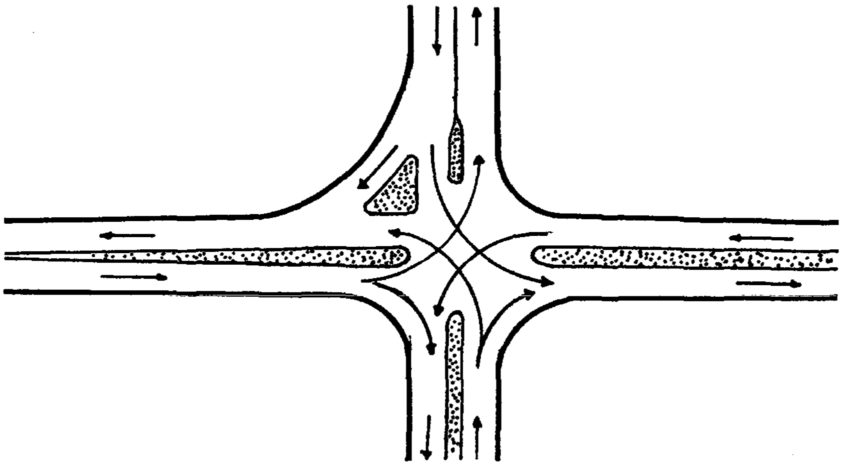


Figure 2.10. Four-Leg Channelized Intersection (2)

signalization, channelization or stop controls are provided. If there are no control devices, some different arrangements must be provided. For example, excessive approach legs can be connected at adjacent intersections in order to provide safety.

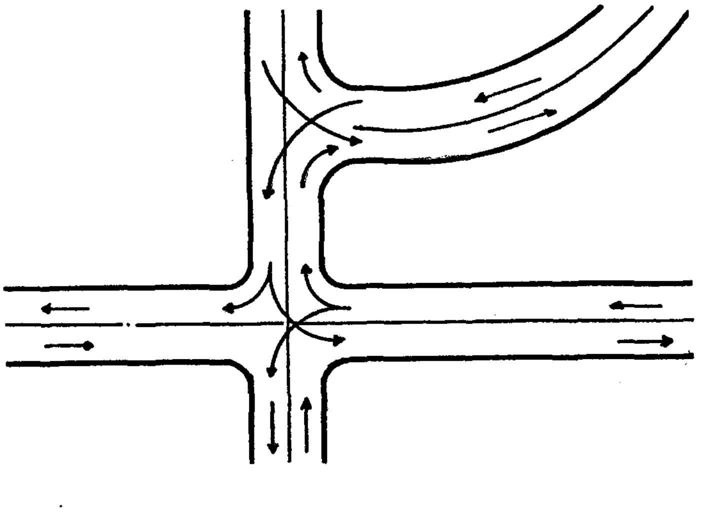


Figure 2.11. Multileg Intersection (2)

Figure 2.11 shows the simplest use of the principle mentioned before.

A rotary intersection can be accepted as a channelized multileg intersection. Although the approach legs do not intersect each other, they are connected by a so-called continuous one-way circular roadway. A rotary intersection is illustrated in Figure 2.12.

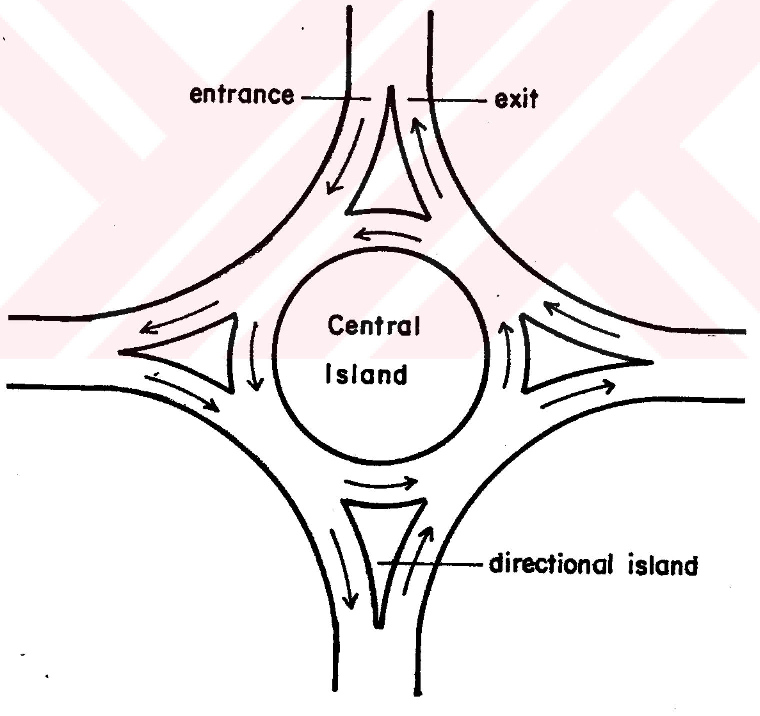


Figure 2.12. Rotary Intersection (9)

NOTES

[1] " Channelization of intersections at grade is the separation of regulation of conflicting traffic movements into definite paths of travel by the use of pavement markings, islands to facilitate the safe and orderly movement of both vehicles and pedestrians" (10). Proper channelization increase capacity, safety and driver confidence.

CHAPTER III

INTERCHANGES

3.1. Definition of Interchange

Interchanges are the intersections of two or more highways on different grades. Ramps are essential components of most of the interchanges. Because they increase the capacity, provide safety and usually remove the turning conflicts, all connections to freeways are to be provided by interchanges. On freeways, at-grade intersections could cause big hazards or bottlenecks therefore, it is better to avoid from their use on high speed highways such as freeways.

3.2. Advantages of Interchanges

The advantages of using interchanges are as follows;

i) Depending on the type of the interchange used, the capacity of the through traveled flows in the interhange is almost equal to the approaching traffic to the interchange.

ii) The safety for through and left turning maneuvers increase and usually travellers who would like to turn right make the same maneuver as they do at the at-grade intersection.

iii) In interchanges, through travellers don't have to stop or change their speed. On the other hand, drivers making turning maneuvers should slow down slightly. Turning maneuvers in interchanges provide comfort, large saving of time and low operating cost.

iv) Interchange designs are flexible and may be adaptable to several intersections. Especially on rolling or hilly topography they are very desirable. An at-grade intersection of a through highway with a steep vertical alignment could cause accidents if the speeds are high.

v) Interchanges are suitable for stage construction. Depending on the planned type, some ramps may be added at later stages. Especially directional, semidirectional and cloverleafs are available for stage construction.

vi) Interchanges are the most important components of freeways.

3.3. Disadvantages of Interchanges

Preliminary disadvantages of interchanges are their cost and the necessary large right-of-way areas. The need of the available terrain for the construction depends on the type of interchange, however it is a well-known principle that GSI needs bigger field than AGI.

The disadvantages of different interchanges are explained below:

i) GSI are expensive. Right-of-way, construction and maintenance costs of such interchanges are more than that for AGI.

ii) Interchanges may not be safe if their patterns are complicated. Especially unfamiliar drivers, feel difficulty using GSI. However, as time passes and users get experience, better practice can be attained. On the other hand, if every measure is provided for the construction of an interchange, even the confusing layout of the interchange may not be considered as an important disadvantage.

iii) The undercrossing grade separation is not suitable for stage construction. Undercross facility must be constructed before hand. If only one structure is enough for undercrossing, than it must be done with its full width. But for overcrossing if two parallel highway is necessary but only one of them is enough for several years, than it is more economical to construct only the first one. The second part can be constructed afterward when it is needed.

iv) Practically in flat topography, it may be necessary to make some crests and sags in the profile of intersecting highways which yields higher expensive costs compared with those constructed in rolling terrain.

v) If there are more than four approach legs on an intersection, a multistructure interchange type may be more costly than combining some of the traffic movements at adjacent intersections.

3.4. General Types of Interchange

The selection of an interchange type is influenced by many factors. These factors are the following;

i) Speed
ii) Volume
iii) Number of intersecting legs
iv) Topography
v) Right-of-way
vi) Cost
vii) Proximity to other interchanges

Interchanges are generally divided into two groups. They are three-leg interchanges and four-leg interchanges.

On all freeways with full access-controlled exits and entrances must be designed with interchanges. Especially on freeway-to-freeway[1] interchanges all traffic movements must be provided by ramps. If the turning traffic is lighter, then the movement may be omitted.

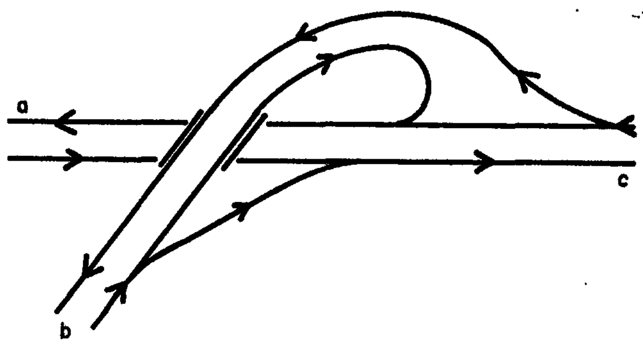
3.4.1. Interchanges With Three Intersecting Legs

Interchanges with three intersecting legs may be of two types, namely, "T" type or "Y" type. The difference between these two types depend on the angle of the intersection. If the intersection with the third approach leg is acute, "Y" type, or otherwise "T" type must be used. Both types could consist of one or two grade separations.

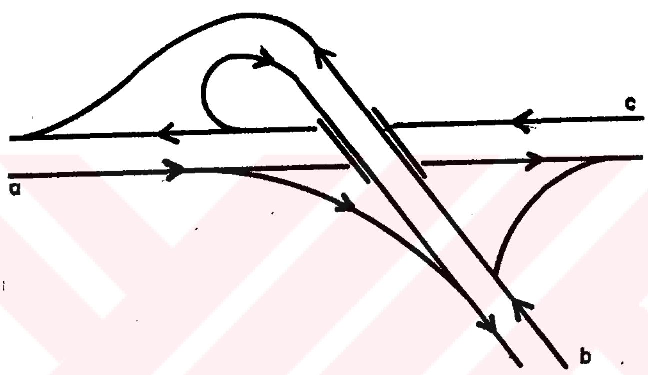
Three-leg interchange with single structure is shown in Figure 3.1.

The interchanges shown in Figure 3.1(a) and (b) are called as trumpet interchanges. These interchanges are especially adaptable to the intersection of primary distributor roads.

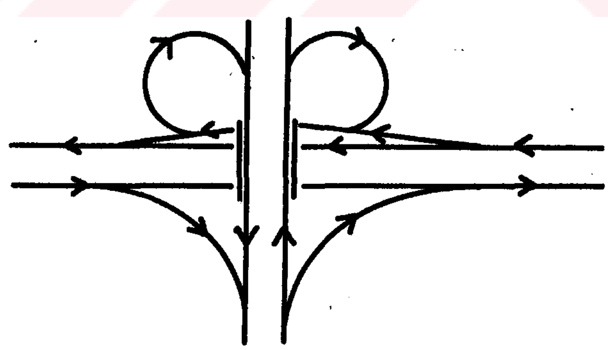
Figure 3.1(a) is desirable whenever the light traffic is on direction "b-a" and the heavy traffic is on "c-b" direction. On the contrary if the heavy traffic is on "b-a" direction and the lighter is on "c-b", the interchange shown in Figure 3.1(b) is more suitable.



(a) "Y" type three-leg interchange



(b) "Y" type three-leg interchange



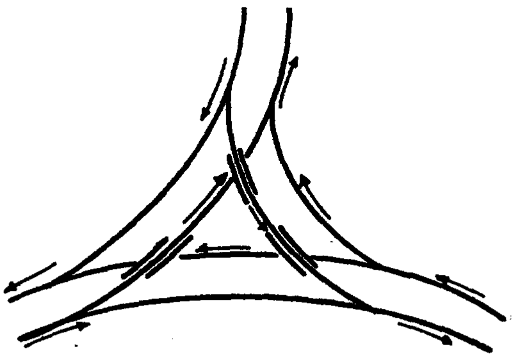
(c) "T" type three-leg interchange

Figure 3.1. Three-Leg Interchange with Single Structure (7)

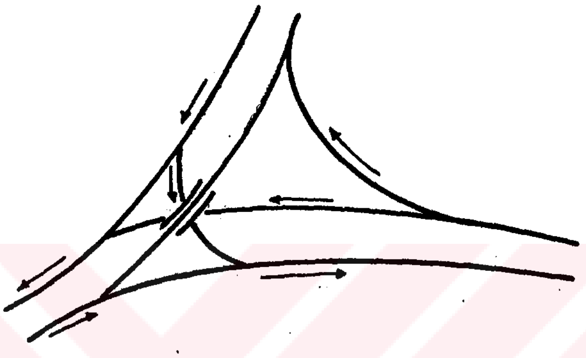
Both Figures 3.1(a) and (b) are less costly than Figure 3.1(c), consequently if the site conditions are favorable for Figures 3.1(a) and (b), then Figure 3.1(c) should not be taken into consideration. Meanwhile the biggest advantages of Figure 3.1(c) is that the collector-distributor roads eliminate the weaving on the main road.

3.4.1.1. "Y" Type Three-Leg Interchanges

"Y" type interchanges shown in Figure 3.2 are desirable at the intersection of a through freeway with the terminal of a major freeway. Both of the freeways must be carrying heavy traffic because direct connections and complicated bridges are to be constructed. Although both of the figures need less right-of-way than trumpet interchange, they are both expensive systems.



(a) "Y" type interchange with three bridge



(b) "Y" type interchange with one bridge

Figure 3.2. Three-Leg "Y" Type Interchanges (7)

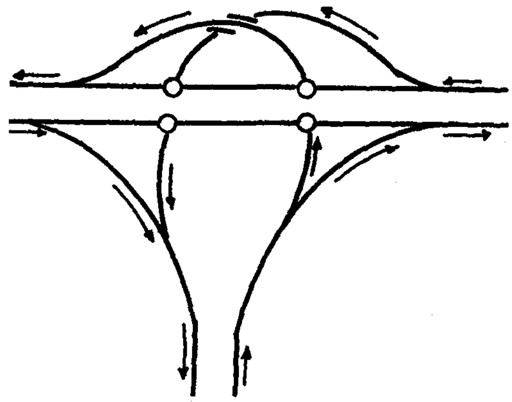
3.4.1.2. "T" Type Three-Leg Interchanges

"T" type three-leg interchanges shown in Figure 3.3 may be used where the intersecting roads are important and carry big volumes of traffic.

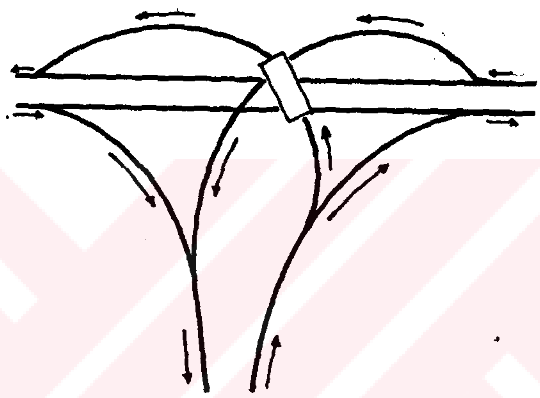
The structures used on the interchanges are costly and make the driver feel comfortable.

Figure 3.3(a) is a three-leg interchange with three structures. Figure 3.3(b) is an alternative of Figure 3.3(a) and the three structures are converted to three-level structure by the replacement of the intersection points.

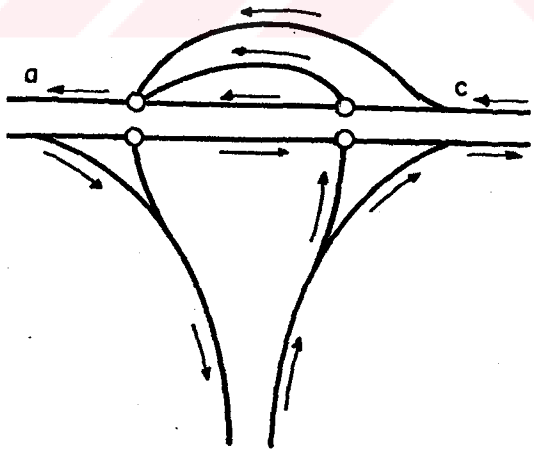
Figure 3.3(c), which is another version of Figure 3.3(a), provides an intersection with two-level structure.



(a) Three-leg interchange with three structure



(b) Three-leg interchange with three-level structure



(c) Three-leg interchange with two-level structure

Figure 3.3. Three-Leg Interchanges with Multiple Structures (2)

3.4.2. Interchanges with Four Intersecting Legs

There are five general types of four-leg interchanges. They are as follows;

i) Ramps in one quadrant

ii) Diamond interchanges

iii) Partial cloverleaves (ramps in two or three quadrants)

iv) Full cloverleaves

v) Interchanges with direct and semidirect connections.

Each type shall be explained and their fields of application shall be given here in after.

3.4.2.1. Ramps in One Quadrant

This type of interchange is available if the traffic volume is low or if there are some restrictions such as topography or culture.

Because there is only one ramp, it should be two-way ramp carrying traffic in both directions.

The best place for the application of this type is the intersection of a scenic parkway and a county highway (2). This means the turning maneuvers is usually light and consist of passenger cars mainly.

Sometimes, this type can be applied as the first step of a full cloverleaf. In Figures 3.4 and 3.5, different patterns of four-leg interchange with single ramp are shown.

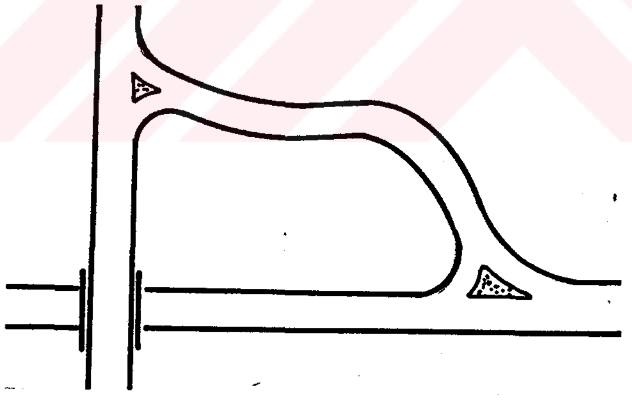


Figure 3.4. Four-Leg Interchange, Ramps in One Quadrant (2)

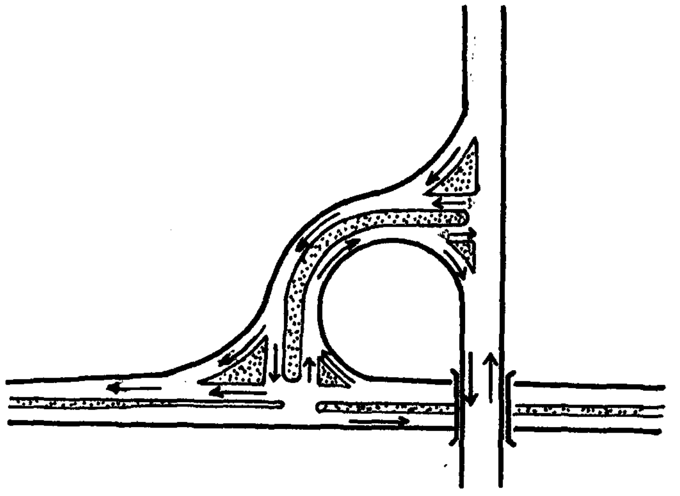


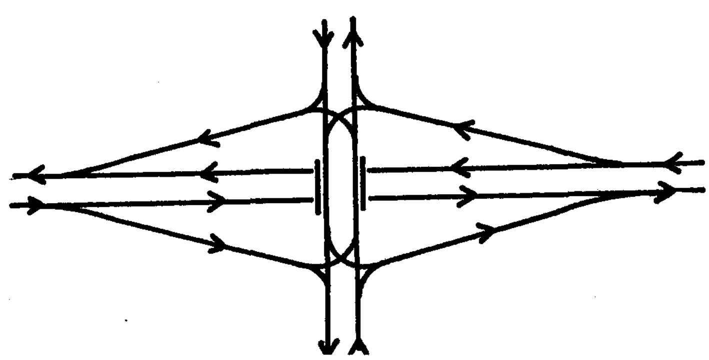
Figure 3.5. Four-Leg Interchange, Ramps in One Quadrant (the Early Step of Stage Construction) (2)

3.4.2.2. Diamond Interchanges

Diamond interchanges are desirable at the intersection of major roads, which are carrying heavy traffic, with minor roads. Such as the intersection of primary distributor roads with district distributors or an arterial road with a city street. The most important parameter, which makes the diamond interchange desirable, is their small right-of-way requirement compared to other types of four-leg interchanges. The other important advantage of diamonds is that the merging and driverging on major highway can be made with high speeds by slip roads. On the other hand, the at-grade intersection on minor roads seems to be the biggest disadvantage of these interchanges.

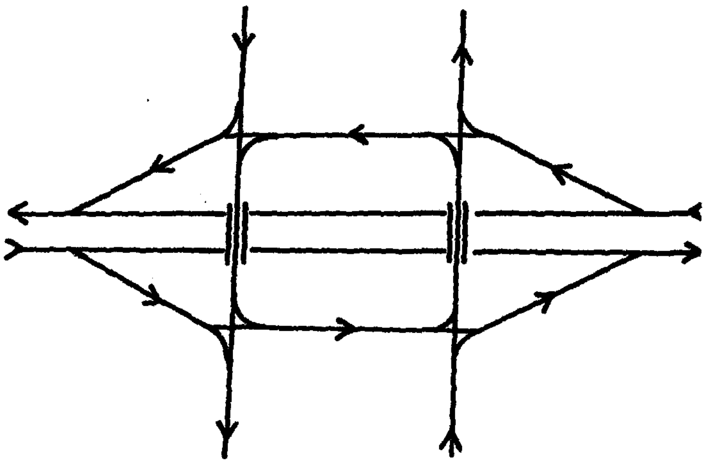
Different arrangements of diamond interchanges are shown in Figure 3.6.

In United States many conventional diamonds are being converted to single-point and compressed forms. These two forms offer advantages in relation to minimizing right-of-way but do have different construction costs (16).

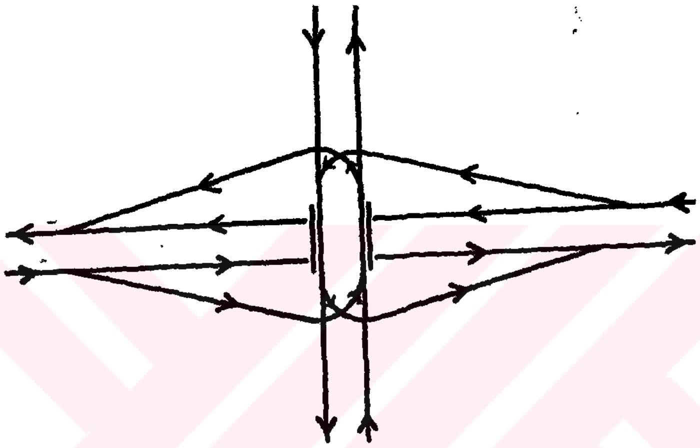


(a) Conventional diamond

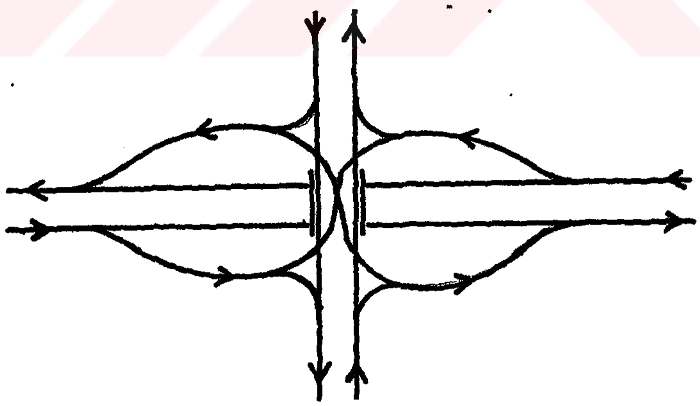
Figure 3.6. Diamond Interchange Forms (16)



(b) Split diamond



(c) Compressed diamond

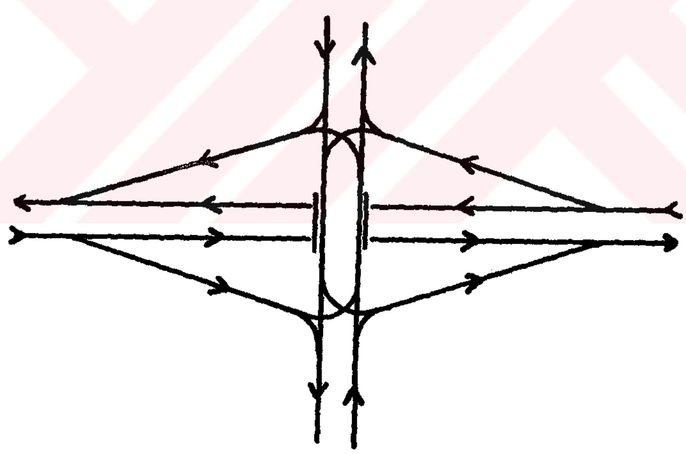


(d) Single-point diamond

Figure 3.6. Continued

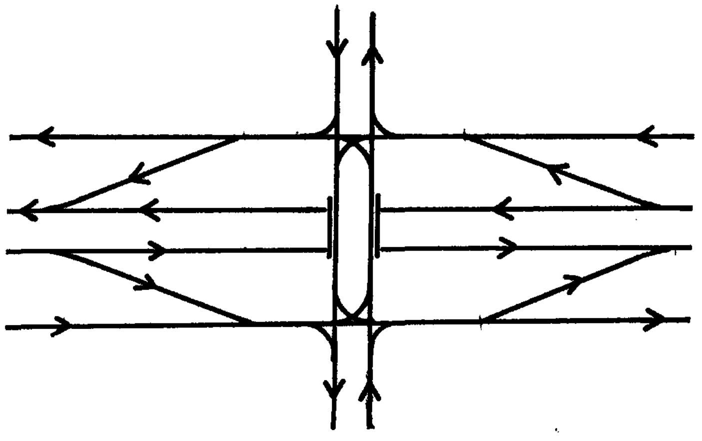
If there happens two one-way streets, which intersect the major highway, split diamond interchange is more appropriate.

Different patterns of conventional diamond systems are shown in Figure 3.7. Conventional diamond interchanges may be connected to frontage roads by ramps as shown in Figure 3.7(b), or they can be converted to cloverleaves as illustrated in Figure 3.7(c).

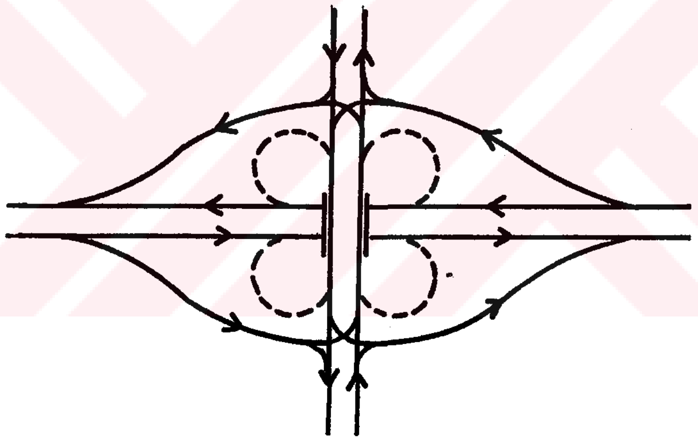


(a)

Figure 3.7. Diamond Interchanges, Conventional Arrangements (2)



(b)



(c)

Figure 3.7. Continued

Figure 3.8 is a split diamond interchange with frontage roads. To diverge from the freeway for some reason or to merge is especially easy if this interchange is used.

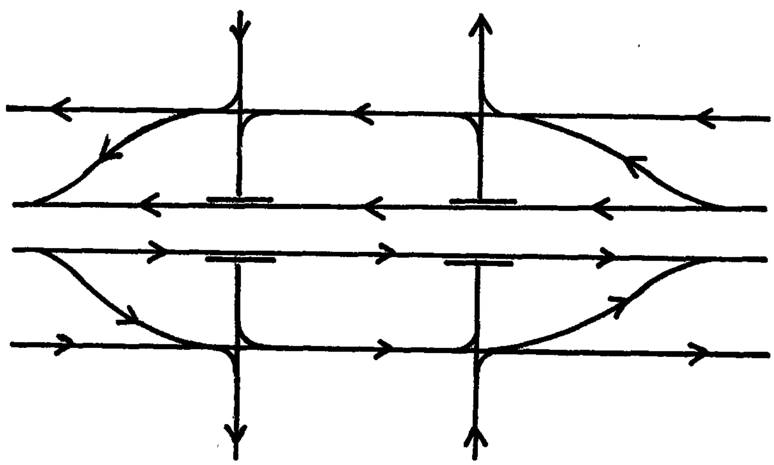
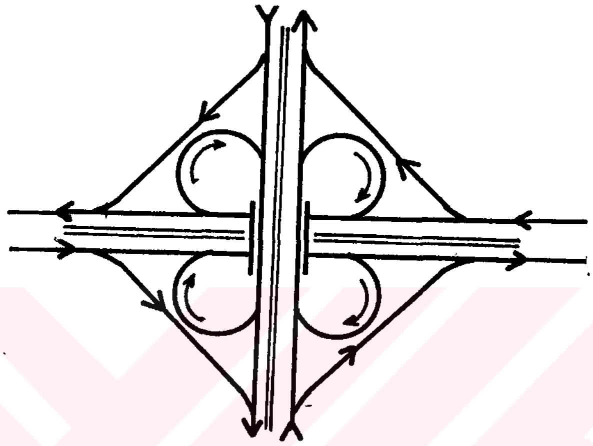


Figure 3.8. Split Diamond Interchange with Frontage Roads (2)

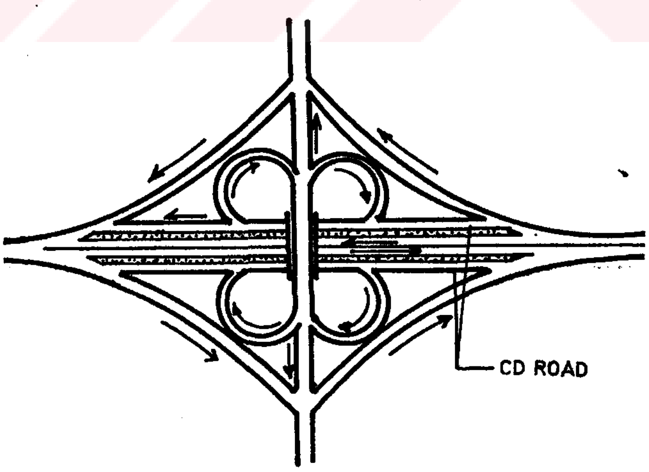
3.4.2.3. Full Cloverleaf Interchanges

Full cloverleaves are four-leg interchanges in which all left turning maneuvers are provided by loops. If there exists loop in each quadrant, it is called as full cloverleaf. Although all the crossing conflicts are eliminated by the use of this interchange, the disadvantages of this type are the excessive right-of-way requirement and the weaving problems.

Weaving can be defined as the section where the traffic entering and leaving a highway at close points. Cloverleaves are the best examples where weaving occurs. Weaving can occur between two ramps in adjacent quadrant. Weavings can cause accidents and have to be eliminated. The widely used solution for the elimination is the use of collector-distributor roads as shown in Figure 3.9 (b). But the collector-distributor roads mean extra right-of-way and investment costs.



(a) Basic cloverleaf



(b) Cloverleaf with collector-distributor roads

Figure 3.9. Full cloverleaf interchanges (18)

The geometric configurations of loops are influenced by the traffic volume and driving speed. If speeds are high, the loop radius gets bigger so that the travel distance increases. On the other hand if the left turning maveuvers are high, it may be necessary to design the loops of the cloverleaf with two lanes and add a collector-distributor road which will bring extra cost.

3.4.2.4. Partial Cloverleaf Interchanges

Depending on the number of turning maneuvers and traffic volume on the cross roads, it is not always necessary to use a full cloverleaf. On the other hand, the pattern of partial cloverleaf may change according to culture and site conditions. For example an existing railway or river effects the design pattern of partial cloverleaf as shown in Fig.3.10.

The simplest form of partial cloverleaf is the one with two-quadrants. It can be used as the first stage of full cloverleaf. If some turnings are predominant than it helps to find the place of the necessary ramps. The suitability of site conditions should not be underestimated.

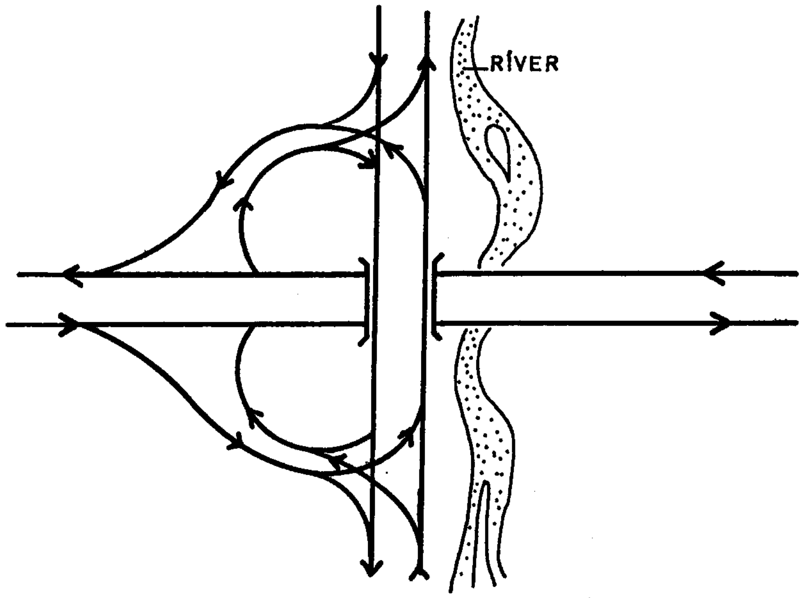
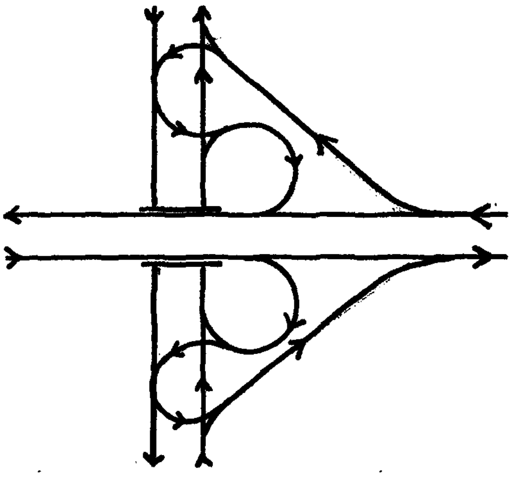


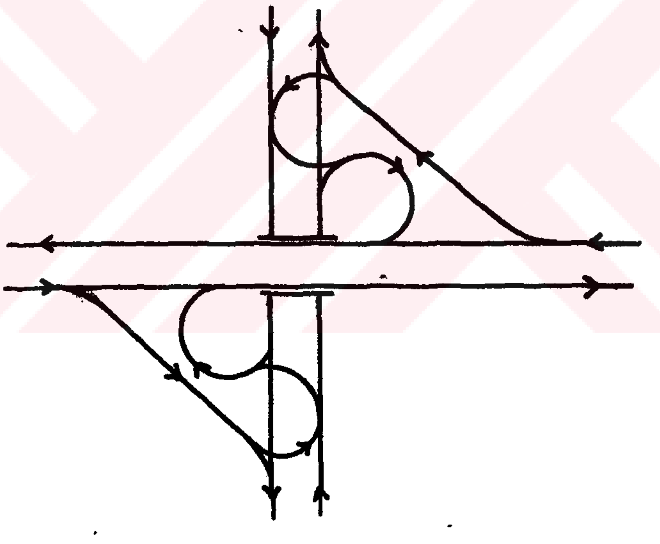
Figure 3.10. Modified Partial Cloverleaf (7)

The different patterns of partial cloverleaf with two ramps are shown in Figure 3.11.

If ramps are on the same side of minor road as in Figure 3.11(a) or diagonally opposite as in Figure 3.11(b) and (c), this case shall be desirable since all turning maneuvers to and from major road are made from right. It is better to avoid the use of pattern shown in Figure 3.11(d), because in this partial cloverleaf, crossing on major highway can not be eliminated.

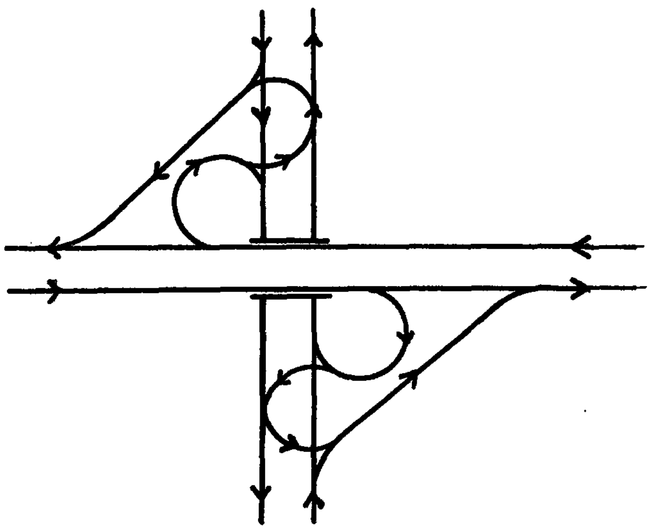


(a) Ramps are on both sides of major road

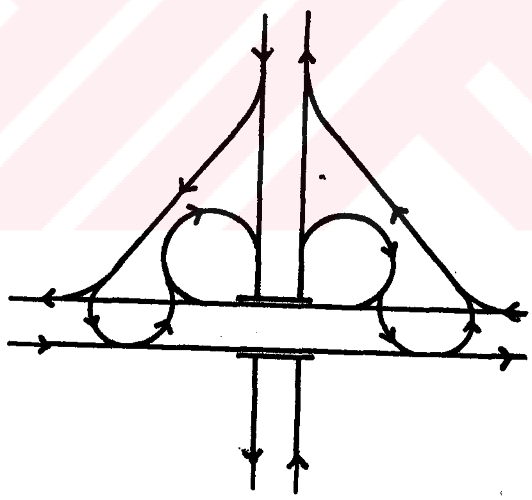


(b) Ramps are diagonally opposite each other

Figure 3.11. Partial Cloverleaf Ramp Layouts (2)



(c) Ramps are diagonally opposite each other



(d) Raps in two adjacent quadrants

Figure 3.11. Continued

Depending on the site conditions and predominant turning maneuvres, the arrangement of partial cloverleafs can be augemented.

A partial cloverleaf with three quadrants is shown in Figure 3.12 where Figure 3.13 shows a partial cloverleaf with four quadrants but two loops.

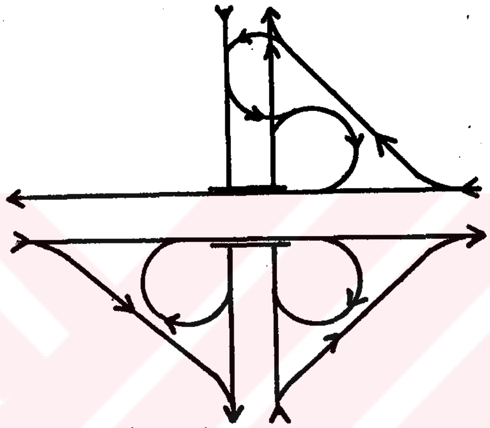


Figure 3.12. Partial Cloverleaf with Three Quadrants (2)

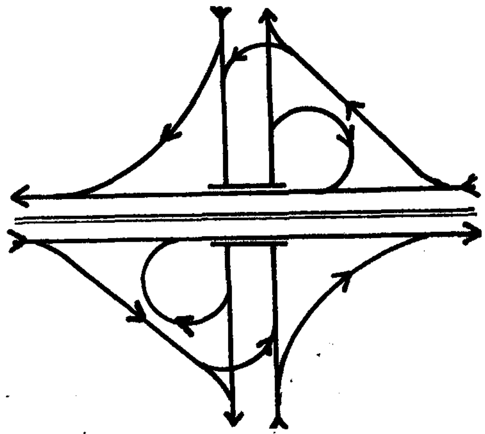


Figure 3.13. Partial Cloverleaf with Four Quadrants (2)

3.4.2.5. Directional and Semidirectional Interchanges

A directional connection can be defined as one-way roadway and interchanges that use direct connections for the major left-turn movements are called directional interchanges (2). Directional connections can only be constructed if the left turning volumes are heavy since the construction cost of these interchanges is very expensive. By the use of these interchanges; the level-of-service increases, the travel distance decreases and enable the drivers to travel almost as fast as they drive on a through highway.

On an intersection where some turning movements are light and some others are heavy, it is possible to design an interchange which consists of both loops and direct connections. An interchange pattern with the combination of loops and direct connections is given in Figure 3.14.

If all turning movements are heavy, a fully directional interchange is most suitable solution on the intersection of two freeways. On this interchange there shall be no confusing for the drivers since left turners should make a maneuver to left and right turners to right. A typical fully directional interchange is shown in Figure 3.15.

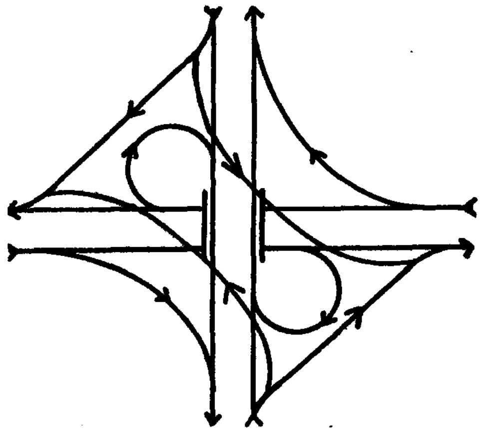


Figure 3.14. Directional Interchange with Two Loops (2)

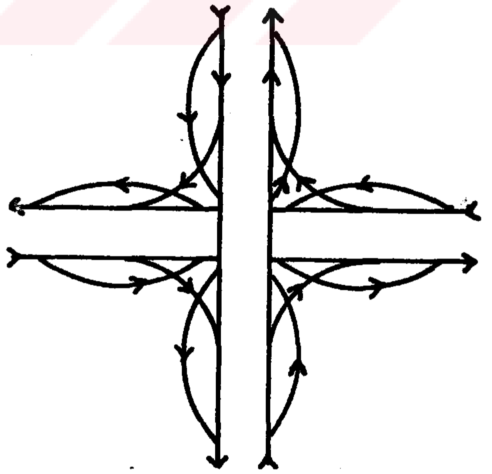
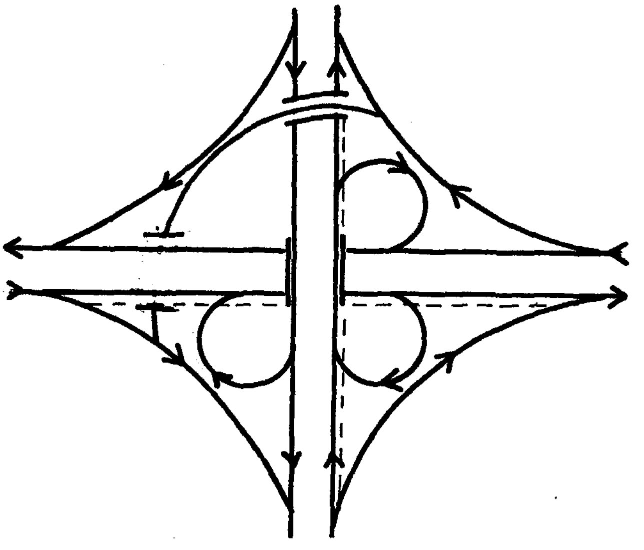
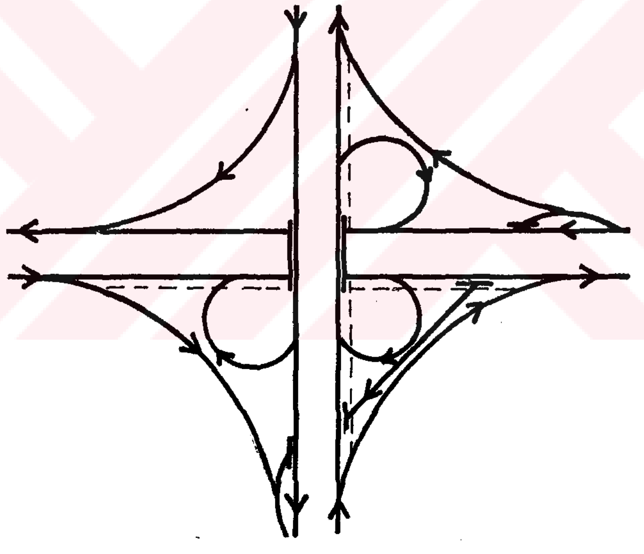


Figure 3.15. Fully Directional Interchange (7)

Semidirect connections are indirect in alignment but not as much as loops. According to predominant turning movements, it is necessary to use interchanges consisting of semidirect connections and loops. In Figure 3.16 a semidirectional interchange with loops is shown. The important factors to be taken into consideration during the design of semidirectional interchanges are; minor turning movements can use loops and the weaving can be eliminated by the provision of collector-distributor roads so that the efficiency and capacity is improved.



(a)



(b)

Figure 3.16. Semidirectional Interchanges with Weaving (2)

NOTES

[1] Freeway-to-freeway interchanges link the freeway segments together (20).

CHAPTER IV

ADAPTABILITY OF INTERCHANGES

4.1. Factors Effecting Interchange Types and Adaptability

The traffic volume, topography and right-of-way are effective criteria on deciding the type of the intersection. Although at-grade intersections and interchanges have different area of use, the limits between the two are not very strictly defined.

Interchanges shall be constructed to eliminate the inevitable congestions on AGI's in case of increasing traffic volumes exceeding AGI's capacity. Nevertheless, a B/C or similar economical analysis should be made prior to decide to construct an interchange instead of at-grade intersection.

4.1.1. Traffic and Operation

When a minor road and a major road cross at at-grade intersection the traffic on the major road is disturbed slightly. But on the other hand, delays begin to occur on the minor road. If the traffic is heavy on the minor road, this problem can only be solved by signalization.

Sometimes grade-separated intersections are turned to be undesirable. This happens if the ramps of the interchange are long and steep or if the merging or speed-change lanes are inadequate. These conditions cause the capacity decrease and disturb the through traffic.

If the through traffic on the highway is heavy, only one ramp which provides most of the turning maneuvers may be suitable.

Within the interchange, right turning maneuvers are provided with ramps which are direct connections. If the left turns are on minor crossroad where the traffic is light, diamond interchanges are more desirable, it is better to use cloverleaf interchange otherwise.

At-grade intersections are the main reasons of queuing, delays and accidents. Especially when the proportion of heavy vehicles in the traffic composition is high because of their adverse acceleration ability, the stops and delays get longer. Therefore the use of interchange with necessary control devices bring many advantages alongwith.

4.1.2. Site Conditions

Usually interchanges can be applied under rolling terrain conditions easily. Ramps can be connected to the interchange with less difficulty compared with the flat terrain conditions where the biggest problem encountered is the appearance and the grades that are not suitable for vehicular operations. The appearance can be improved by landscaping.

The right-of-way required for an interchange depends on the turning maneuvers which should be provided with ramps and the access which require additional area. The grade separation effect the profile of the highway.

4.1.3. Safety

The most significant aspect of interchanges is vast improvement in safety. Because the crossing and turning conflicts are almost eliminated, the possibility of an accident is absolutely minimum. But on the other hand, the grade separation, itself, naturally has a tendency to cause accidents. To protect from those potential accidents, the precautions such as, increase in roadway widths or the use of protective devices as bridge abutments and piers can be taken.

Consequently, left turns are provided with loops or direct connections and right turn with ramps so that turning conflicts caused by crossing traffic will be eliminated.

4.1.4. Economic Factors

Interchanges are the most expensive type of intersections. The cost of structure, ramps, grading, landscaping, markings, lighting and other control devices make the interchange an expensive structure compared to at-grade intersections. On the other hand, directional interchange is the most costly type of interchanges because it is necessary to construct more than one structure.

While making a comparison between at-grade intersections and interchanges, vehicle operating cost must be taken into consideration. Vehicle operating cost depends on turning maneuvers where rise and fall happen in vehicular speeds. In other words, it depends on the traffic and the design. Usually on through roads there happens no extreme reduction in speed and the right-turning maneuvers are the same for at-grade intersection and interchange. Hence the added vehicle operating cost is the same. But for left-turning, depending on the type of interchange, the travel distance may increase compared with at-grade intersection but the speed reduction is not very low. Usually for intermediate to heavy traffic, the total vehicle-operating cost at an interchange will be reasonably lower than that for an at-grade intersection.

4.2. The Applications and the General Principles of State Highways (KGM) on Interchanges Planning

In 1985 some general criteria were published for grade-separated intersections by KGM. These general principles are also applicable for over and under passes. They are as follows:

i) First of all the necessity of a grade-separated intersection should be justified economically and technically.

ii) The need of grade-separated intersection should be previously considered and determined during the design of the roads and necessary precautions must be taken.

iii) Grade-separated intersections should not be considered as the first and unique solution for the vehicles and pedestrains, safety. On the contrary they are the last solution after determining the insufficiency and inconvenience of other precautions, because grade-separated intersections are much more expensive than other methods such as channelization and signalization.

iv) State Highways deals only with the road construction and apportenances which are within the scope of its responsibility.

v) The grade-separated intersection should be so proposed that implementation of it should provide a solution for a wide hinterland. In other words, grade-separated intersection should not be planned in order to solve small-scale spot bottlenecks which occur only during some special days. Interchanges should not be considered as a short-term solution. During its planning all the environmental effects have to be taken into consideration.

vi) During the planning and construction of grade-separated intersections it is advisable to apply the same solutions for the same conditions so that a standard would be established based on technical principles.

The general criteria explained above are valid not only for highway-highway intersection, but also for highway-railroad and highway-pedestrian intersections.

The specific technical criteria for construction of interchanges adopted by KGM are as follows;

i) First of all, the plan of interchanges should be confirming with the general principles of KGM.

ii) If the roads connected to the arterial highway are considered as an integral part of the arterial highway than the interchange is necessary to eliminate bottlenecks and spot congestions.

iii) When the traffic demand exceeds the capacity of at-grade intersection, interchange shall be required.

iv) When road user expenses due to delays and stops are higher than the cost of construction an interchange.

v) All measures should have been tried and implemented for at-grade intersections before an intersection is proposed.

vi) The type of terrain and topography on which grade separation is to be located shall be taken into consideration as a major factor in decision (12).

4.3. Determination of the Type of Interchange

Firstly, it is known that the possibility of accident is more likely if there are turning or crossing conflicts on high-design-speed highways. Hence, at-grade crossing should not be permitted on these types of highways, if possible.

When the freeways cross each other and turning volumes are heavy, directional interchange must be used with the provisions of landspacing or with other arrangements for its appearance. The biggest disadvantage of directional interchange is its high cost because of complex structure.

When turning volumes are high for some flows and light for some others, it is possible to use an interchange which is the combination of direct, semidirect connection and loop ramps.

If at-grade intersection is prohibited and there is no right-of-way problem, cloverleaf is recommended. The biggest disadvantage of this type is the high cost and the weaving problem although the weaving problem may be eliminated with collector-distributor roads. Especially in urban areas, cloverleafs are not desirable due to high land costs.

In some cases, available land may not be adequate for one or more quadrants or one or more movements when a partial cloverleaf is recommended. Partial cloverleaves permit the further development of the interchange when land becomes available to complete the cloverleaf.

Diamond interchange is the common interchange type which is used when a major road crosses a minor road. The most important disadvantage of diamonds is that intersection for minor road is at-grade. This crossing effects the capacity of the diamond interchanges. If the traffic on both of the roads gets high, the signalization on the at-grade terminal of the interchange may become unsufficient in the future.

Interchanges with three intersecting legs are necessary where two of the three legs form a through road. "Y" type is more adaptable for heavy left-turning movements compared with "T" type three-leg interchanges.

In final decision for the type of interchange to be implemented, the following factors should be studied in detail and the alternatives must be compared accordingly:

i) Capacity

ii) Cost

iii) Weaving length

iv) Harmony with the environment

v) Potential for stage construction

vi) Availability of land

vii) Grades

viii) Curve radius

The best alternatives shall be selected after these studies, and the final decision among them shall be taken by the planners.

During the planning of Ankara ring road, KGM has developed criteria to determine the most convenience places where interchanges shall be located and for the types of interchange to be constructed.

The locations of interchanges were determined according to the following principles;

i) Trip duration (the length of the trip)

ii) Largeness of city spaces

iii) Traffic volumes

iv) Interchange construction cost

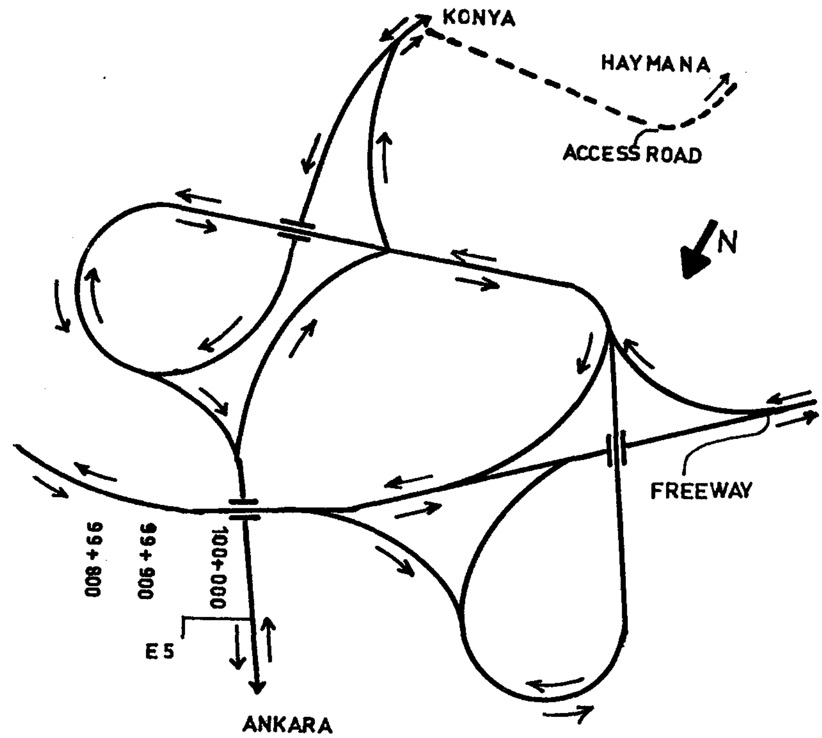
If the site conditions are favorable, the entrances to the freeway could be chosen downhill and the exist from the freeway could be uphill for safety considerations.

Another important problem is the spacing of interchanges. There is no precise formula for the spacing of interchanges. The aim of using interchanges is to provide sufficient accesses to traffic without interfering with the flow of through traffic on the freeway. The spacing of interchange is the most important factor in obtaining higher level-of-services of the freeway.

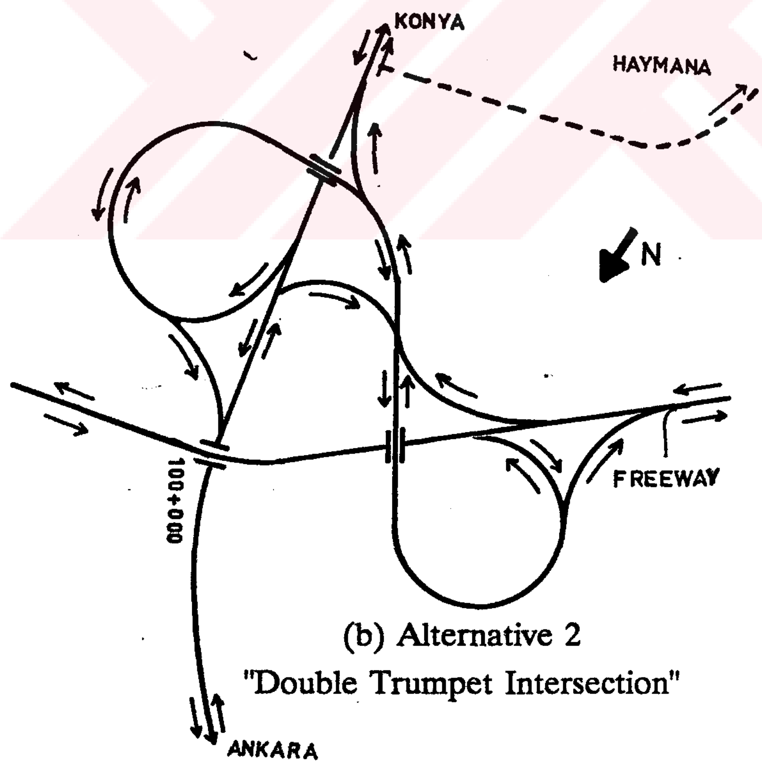
Two types of interchanges will be used. They are System Interchange and Service Interchange. System Interchanges connect freeways to freeways and Service Interchanges connect freeways to other highways.

The ramps must be planned according to the traffic volume and topography. The ramps usually serve in one-direction. But if the expected traffic volume is low, than the ramps can be operated in both directions.

In order to understand how to find the best alternative among several types of interchanges, a comparison study in terms of the principles mentioned before is helpful. The International Incorporation Parsons, which is the project services subcontractor of Gerede-Ankara Freeway and Ankara Ring Road, has prepared many interchange alternatives comparison study for the intersections at the preliminary planning studies. One of the interchange alternatives comparison study has been made for the intersection of road E-5 with the freeway. Alternatives are shown on Figure 4.1 and on Table 4.1.

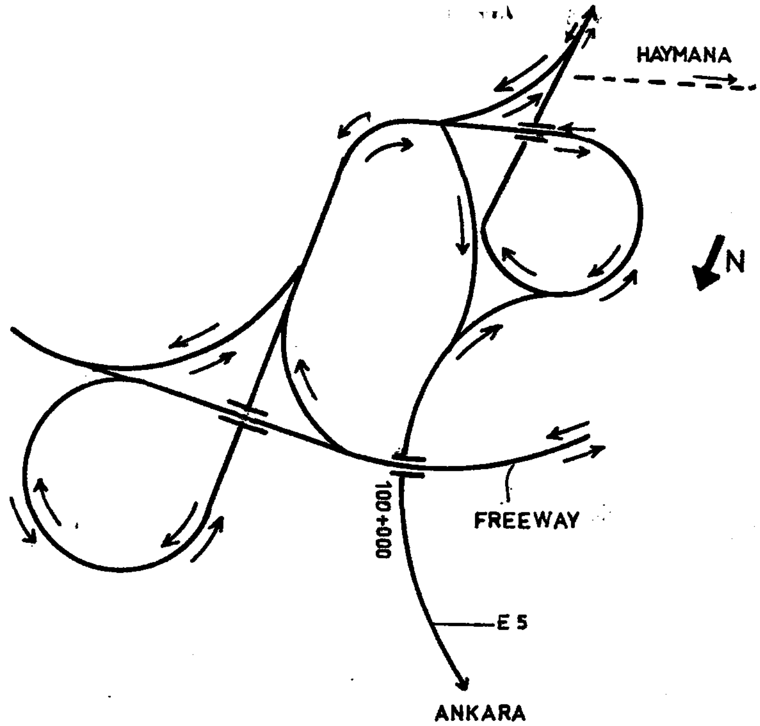


(a) Alternative 1
"Double Trumpet Intersection"

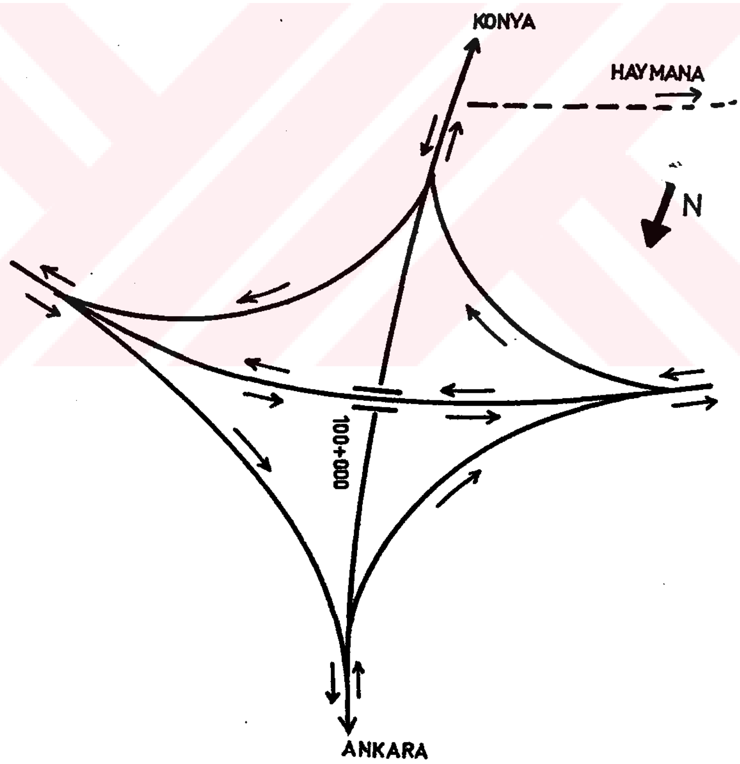


(b) Alternative 2
"Double Trumpet Intersection"

Figure 4.1. Proposed Interchange Alternatives to the Intersection of E5 with Motorway (11)

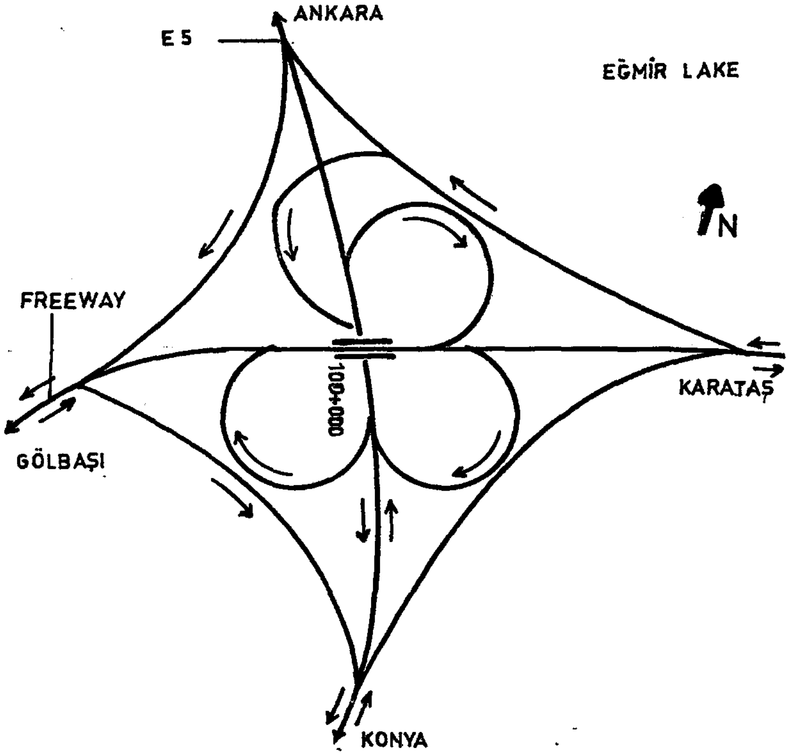


(c) Alternative 3
"Double Trumpet Intersection"

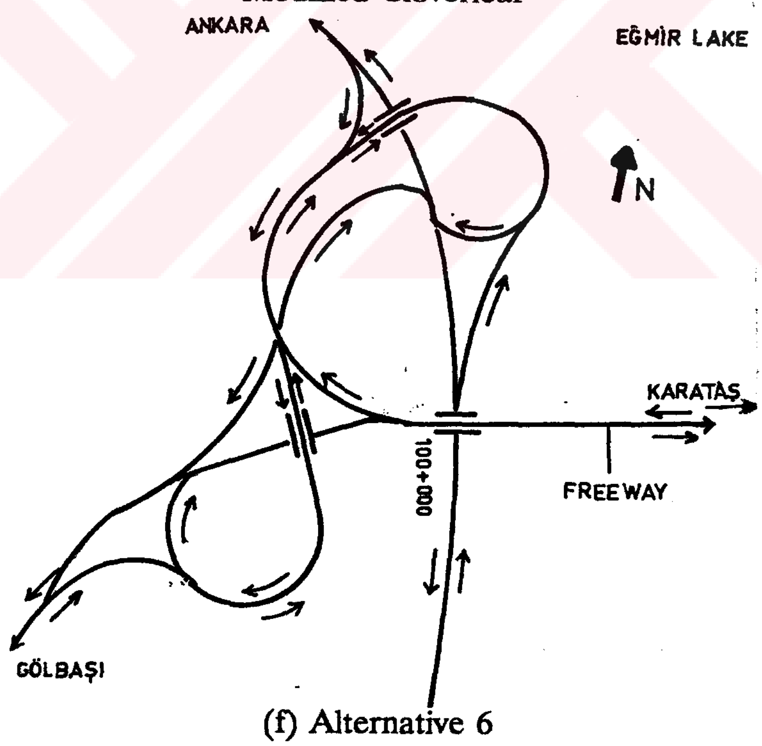


(d) Alternative 4
"Diamond Intersection"

Figure 4.1. Continued

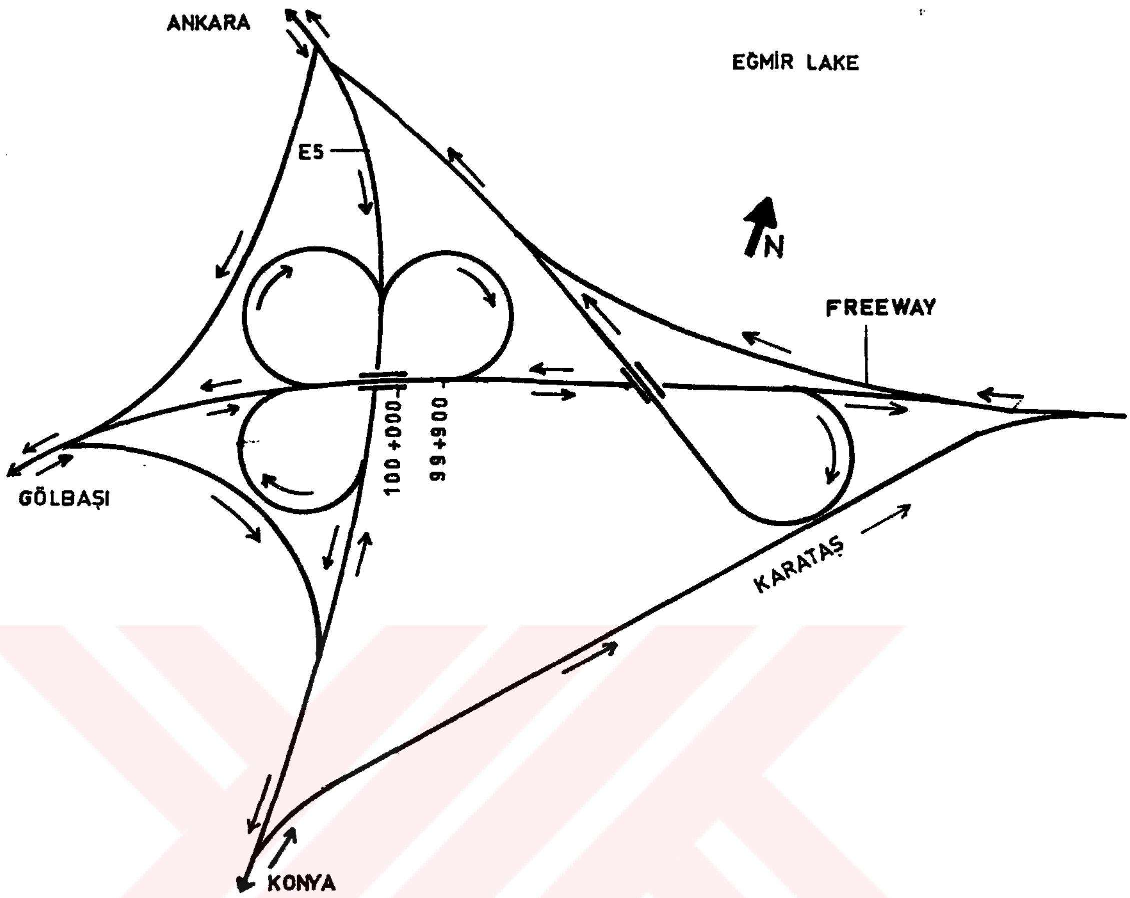


(e) Alternative 5
"Modified Cloverleaf"



(f) Alternative 6
"Double Trumpet"

Figure 4.1. Continued



(g) Alternative 7
"Modified Cloverleaf"

Figure 4.1. Continued

In Table 4.1, property expropriation and power line diversions are not included in the construction cost. Number of buildings expropriated are taken approximately.

Table 4.1. Interchange Alternatives Comparison (11)

	Structure Type	Length (m)	Weaving Length	Max. Grade %	Min.Curve Radius (m)	Volume (Mm^3)		Construction $ Cost (M)	Expropriated Property (No)	Capacity/ Movement	Level of Service
						Cut	Fill				
ALT 1	Double Trumpet	4650	300	5,2	105-130	1,6	1,25	10,35	30	230-1140*	B
ALT 2	Double Trumpet	4500	200	5,5	105-130	1,3	1,5	9,9	20	230-1140*	B
ALT 3	Double Trumpet	4900	>300	2,9	105-130	1,0	1,6	10	20	230-140*	B
Alt 4	Diamond	2000	-	4	-	0,45	0,25	3,75	-	230-1140*	D
ALT 5	Modified cloverleaf	6600	200	5,0	100	0,85	0,725	9,0	10 140Wk.shops	1200**	D
ALT 6	Double Trumpet	4900	250	6,0	75	1,35	0,9	10,7	10	230-1140*	B
ALT 7	Modified Cloverleaf	7100	200	8,5	75	0,9	0,73	9,5	10	1200**	F

* The interchange capacity depends on merging and diverging capacities

** The interchange capacity depends on weaving capacity

The cost of earthworks for alternate 3 may increase significantly due to the need for expanded earthworks for high embankment over marshy ground (lake deposits) and other grading and land improvement developments.

Capacity of interchange movement is limited by ramp merging and diverging capacities for alternatives 1,2,3,4 and 6.

Capacity of interchange movement is limited by weaving capacity for alternatives 5 and 7.

For alternative 2, the weaving legnth is 200 m. This length works well enough to give level-of-service B but the effect of gradient is not considered. The combination of steep gradient and short weaving length make this alternative sensitive to high proportions of slow vehicles.

The weaving length of alternative 6 is 250 m. This provides an acceptable facility.

The weaving length of alternative 5 and 7 is 200 m. on local and collector-distributor roads. This weaving length is not satisfactory for the higher flows, and level-of-service will fall to F.

At the link roads of trumpet interchanges the traffic speed is 60 kph. as in an urban street, cloverleaf interchange based on weaving traffic condition at 60 kph. On the other hand the proposed diamond interchange is based on at grade turning movements on E-5 and causes long delays and a level-of-service D.

In Table 4.2, the alternative ranking according to level-of-service and overall terms is given.

The expropriation costs are based on assumption on property and land values and may vary considerably from these assumed values. Increase in land values would favour alternatives 1,2,4 and 6 and decreases would favour alternatives 3,5 and 7 as shown in Table 4.2.

Power line diversion values shown in Table 4.2 are based on costs of moving one pylon and were obtained from TEK. They do not allow for compensation for grid shutdowns. Increases in these costs would favour options 4,5 and 7.

Table 4.2. Alternatives Ranking (11)

	Construction Cost $M	Approximate Expropriation Cost $M	Approximate Power Line Diversion Cost $M	Ranking of Level-of-Service	Total of Cost $M	Overall Ranking
ALT1	10.35	3.9	0.3	1	14.55	2
ALT2	9.9	3.5	0.2	4	14.6	3
ALT3	10	4.0	0.25	3	14.25	4
ALT4	3.75	1.3	0.0	7	5.05	7
ALT5	9.0	5.7	0.1	5	14.8	5
ALT6	10.7	2.8	0.2	2	13.7	1
ALT7	9.5	3.8	0.1	6	14.4	6

After the evaluation of each type, alternatives 1 and 6 seem as the best alternatives. When these two alternatives are compared, the following advantages can be mentioned;

Advantages of alternate 1 are;

a) Link length provides good weaving capacity.
b) Geotechnical and drainage problems are less than those in alternatives 2,3 and 6.
c) Provides best level-of-service.

Advantages of alternate 6 are;

a) Link length provides adequate capacity.
b) Has the least expropriation cost.
c) Good level-of-service.
d) Requires less extensive major power line diversion than alternate 1 (11).

Alternate 6 is the best and most suitable interchange under prevailing conditions. Although its pattern is a little complex, this alternate results as the best alternative.

Another case study for the interchange type determination could be İzmir-Aydın Freeway and İzmir Ring Road. This freeway is 144 km. in length and fully access control facility. The Ring Road starts from the state highway and north-west of İzmir and ends at near Balçova as shown on Figure 4.2. İzmir Ring Road is 50 km. in length and it will be two way with three lanes on each direction. The freeway between Tahtalıçay and Işıkkent shown in Figure 4.2 is considered as the part of the Ring Road.

İzmir-Aydın Freeway starts at Tahtalıçay and extends in the south-east direction. Its length is approximately 84 km. It is a two way-three lane toll road.

The locations of interchanges are determined according to the following criteria;

a) Traffic volume

b) Drivers expectation

c) The existing and planned site conditions

There are three types of interchange which are included to this project. They are cloverleaf, trumpet and diamond type intersections.

Cloverleaf interchanges will generally have at least 60 m. radius and one way loop ramps. There will be free flow at every direction and collector-distributor roads will be constructed.

Trumpet interchanges will be used where three directions are crossed. Mostly the loop ramps will be one lane and at least 60 m. in radius, but the loop ramp radius will be 120 m. and two-lane for interchange I/C4 because of its heavy traffic conditions.

Wherever the turning movements are light, diamond interchange is proposed (15).

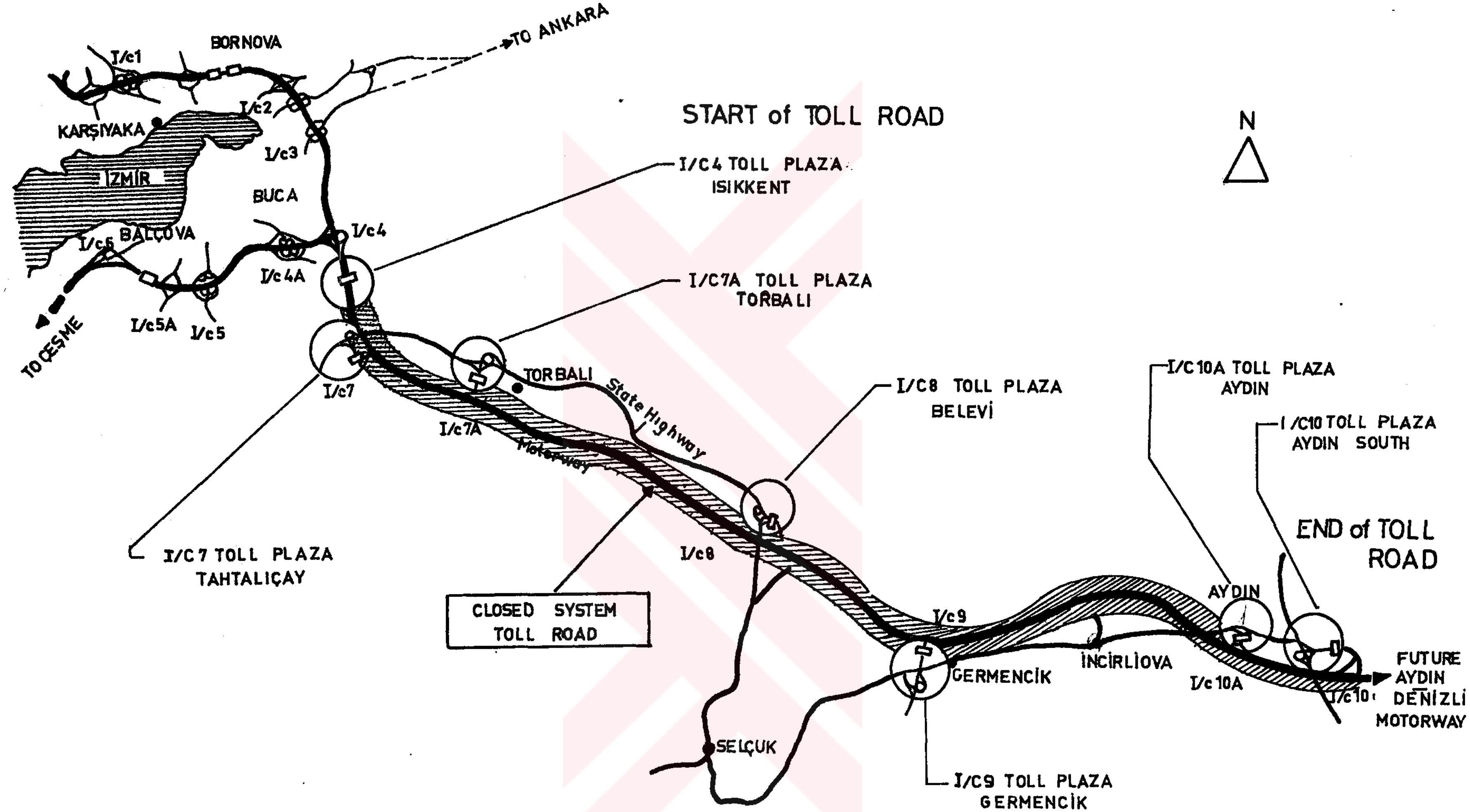


Figure 4.2. İzmir-Aydın Freeway and İzmir Ring Road (15)

CHAPTER V

GRADE-SEPARATED INTERSECTIONS FOR URBAN HIGHWAYS

5.1. Analyses to be Made for Planning Urban Interchanges

The intersections are the locations where jam densities and long delays have been experienced. The performance of traffic flow of urban roads is related with the performance of intersections. Therefore, it can be concluded that the roads having high capacities are not effective if superimposed intersections are not adequate.

The level-of-service at an intersection decreases when traffic volume approaches capacity, thus traffic congestion and delay begin to occur. This traffic congestion does not only cause the loss of time but also economic losses.

While searching a solution for the congested intersections, some basic analyses must be performed. These analyses are;

a) Capacity analyses

b) Level-of-service analyses

c) Economical analyses

Grade-separated intersections are usually undesirable in urban areas because of the expensive right-of-way, construction costs and limited site conditions. Moreover, especially in rapid but non-sound urbanizated cities the solution of a congested intersection causes the congestion of another nearby intersection.

In order to increase traffic capacity and provide safety in an urban road before a grade separation is implemented, the following precautions should be previously taken to be able to eliminate the problems:

i) The use of busways, pedestrian ways.

ii) The use of bypass and freeways to separate the transit traffic from the urban traffic.

iii) The use of one way streets and divisional islands.

iv) Arrangement of park places out of the roads and prohibition of on-the-road parking.

v) Signalization, prohibition of left turning, limited access to roads.

In situations where these solutions are inefficient the use of over and under passes both for pedestrians and vehicles or the use of grade separated intersections are to be proposed (19).

In Table 5.1 the capacities of some intersections are given. These capacities are also the ones adopted by KGM.

Figure 5.1 is a rough graph given a general idea on intersections in urban roads. This figure is prepared according to Russian traffic conditions (3).

Figure 5.1 has been expanded and applied to Turkey's traffic conditions by Assoc.Prof.Dr. Muhittin Özdirim as shown in Figure 5.2. While preparing this figure traffic safety was taken into consideration as the primary factor.

Figure 5.1. Intersection Capacities in terms of Hourly Design Traffic (1)

INTERSECTION TYPES	UNCHANNELIZED		CHANNELIZED	
	Unsignalized (pcph)	Signalized (pcph)	Unsignalized (pcph)	Signalized (pcph)
Three-leg "T" intersection	600~1000	1000~1500	1000~1900	1500~2000
Four-leg intersection	800~1000	1100~1600	1400~2000	1500~2400
Roundabout, 2 lane, four leg diameter = 197~263 ft (60~80 m)	2000~3000	2000~3200	2000~3000	2000~3200
Roundabout, 3 lane, four leg diameter = 329~460 ft (100~140 m)	3000~5000	3500~5500	3000~5000	3500~5500
Partial Cloverleaf	-	-	3000~5000	-
Trumpet	-	-	3000~5000	-
Cloverleaf	-	-	3000~6000	-

AADT = Average Anually Daily Traffic

HDT = Hourly Design Traffic

$$HDT = \frac{AADT}{10}$$

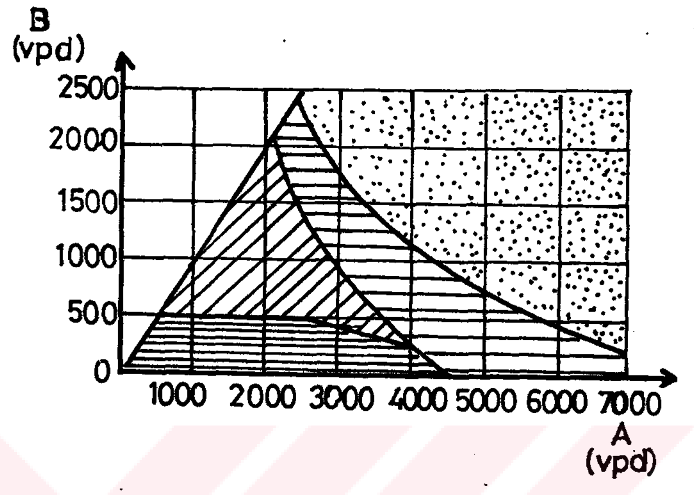
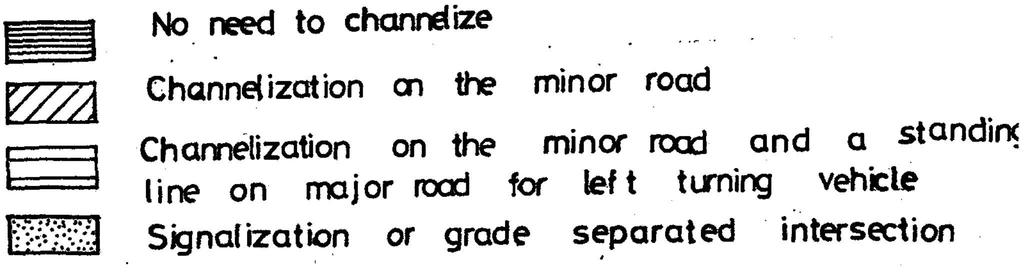


Figure .5.1. Intersections capacity (3)



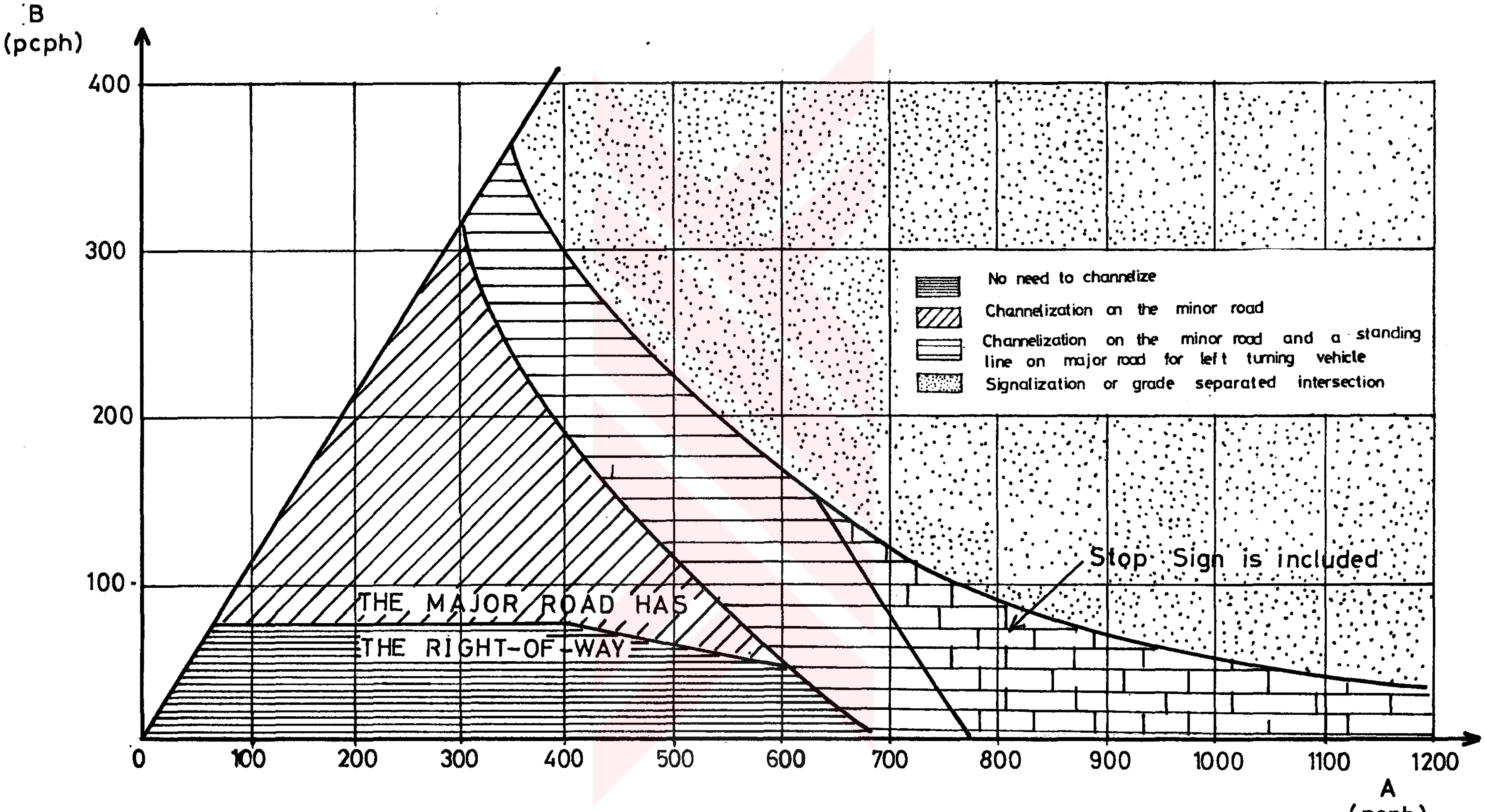


Figure .5.2. Decision Criteria for Simple Intersections in Türkiye (23)

5.1.1. Capacity Analysis

When an urban intersection is congested there are several precautions such as signalization, channelization or using islands as explained earlier in this chapter. If the congested intersection has been signalized, channelized and operating with the help of some islands or rotary, than some more detailed structures may help to solve the congestion. In urban roads, especially on arterial underground railway is a preferred solution for many congested at-grade signalized intersections. But sometimes it is not possible to construct an underground railway than grade-separated intersection could be applied.

At the beginning of searching a solution for the congested intersection, a capacity analysis is deemed to be essential to observe the existing situation.

Signalized intersection is one of the most complex traffic regulating systems. Signalized intersection analyses must consider a wide variety of factors including the amount and distribution of traffic movements, traffic composition, geometric characteristics, and the details of intersection signalization.

The capacity of a highway depends on the geometric characteristics and composition of the traffic streams. The capacity of a highway can be improved by the modification of geometric conditions.

The capacity ratio of a signalized intersection can be expressed by a ratio which is calculated by dividing demand flow rate to capacity.

The level-of-service of a signalized intersection can be expressed in terms of average stopped delay per vehicle.

There are several, elements which effect the capacity of a signalized intersection as given below;

a) Geometric characteristics

b) Composition of traffic stream

c) Allocation of green time

d) Phase plan

e) Cycle length

f) Signal timing

In order to see the operation of a signalized intersection it is necessary to compute the (v/c) ratios; for individual movements, for the overall intersection and the level-of-service of the intersection.

In Figure 5.3, the methodology of capacity and level-of-service analyses of HCM is shown as a flow chart.

The capacity and level-of-service of a signalized intersection are determined in five modules as shown in this figure.

5.1.1.1. Input Module

The input module is essentially a summary of the geometric, traffic and signalization characteristics needed to find other parameters. Where an existing case is under study, most of these data will be obtained from field studies.

a) Traffic Volumes (hourly)

b) Lane Configuration

1. Number of lanes
2. Lane widths

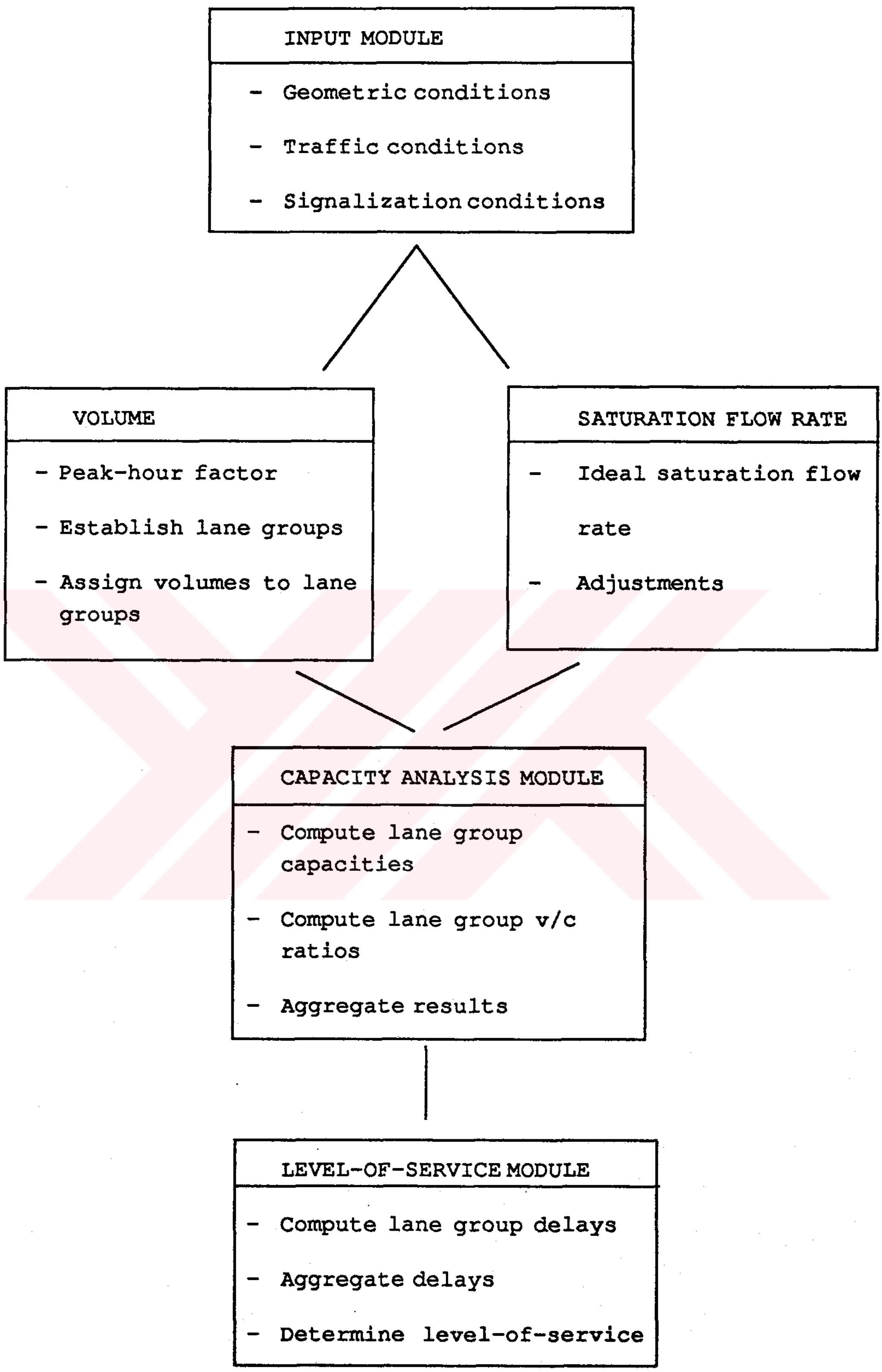


Figure 5.3. Methodology of Capacity and Level-of-Service Analysis for Signalized Intersections (22)

3. Traffic movements using each lane

4. Location of curb parking lanes

5. Length of storage bays

6. Islands

7. Location of bus stops

c) Geometric and Traffic Conditions

1. Percent grade

2. Percent heavy vehicles

3. Parking characteristics for the approach

4. Number of local buses stopping per hour to discharge or pick up passengers

5. The peak-hour factor

6. The number of pedestrians per hour using pedestrian crossings

7. Pedestrian controls at the intersection

8. Indication of the arrival type. Shown in Table 5.2.

Table 5.2. Relationship Between Arrival Type and Platoon Ratio (22)

Arrival Type	Range of Platoon Ratio
1	0.00 to 0.50
2	0.51 to 0.85
3	0.86 to 1.15
4	1.16 to 1.50
5	> 1.51

$$R_p = \frac{PVG}{PTG} \tag{5.1}$$

R_P = Platoon ratio

PVG = Percentage of all vehicles in the movement arriving during the green phase

PTG = Percentage of the cycle length that is green for the movement (g/C x 100)

5.1.1.2. Volume Adjustment Module

a) Hourly volumes

b) Conversion of hourly volumes to peak flow rates by PHF method

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

v_p = Converted hourly volume

V = Hourly volumes (vph)

$$PHF = \frac{Peak\ hour\ volume}{4.(15\ \text{min}\ peak\ volume)} \tag{5.2}$$

c) Lane group

d) Lane group flow rate (v_g)

e) Number of lanes in the lane group

f) Lane utilization factor (U). It can be obtained from Table 5.3.

g) The adjusted lane group flow rate

$$v = v_g \cdot U \tag{5.3}$$

v_g = Lane group flow rate (vph)

U = Lane utilization factor

v = Adjusted flow rate for the lane group (vph)

Table 5.3. Lane Utilization Factors (22)

No of through lanes in group	Lane utilization factor (U)
1	1.00
2	1.05
≥ 3	1.10

h) The proportion of left or right turns in lane group

$$P_{LT} = \frac{v_{LT}}{v_g} \tag{5.4}$$

$$P_{RT} = \frac{v_{RT}}{v_g} \tag{5.5}$$

5.1.1.3. Saturation Flow Rate Module

In this module, the total saturation flow rate that can be accommodated by the lane group under prevailing conditions is computed.

Steps that should be followed during the calculation of saturation flow rate are as follows;

a) Description of lane groups.

b) The ideal saturation flow rate: For most computations this value will be taken as 1800 pcphgpl.

c) Adjustment factors: The ideal saturation flow rate is multiplied by the number of lanes in the lane group and by eight separate adjustment factors. These factors are given in Table 5.4 to 5.11.

Table 5.4. Adjustment Factor for Lane width (22)

Lane Width, ft	8	9	10	11	12	13	14	15	≥ 16
Lane Width Factor, f_w	0.87	0.90	0.93	0.97	1.00	1.03	1.07	1.10	Use 2 Lanes

Table 5.5. Adjustment Factor for Heavy Vehicles (22)

Percent Heavy Vehicles, %HV	0	2	4	6	8	10	15	20	25	30
Heavy Vehicle Factor, f_{HV}	1.00	0.99	0.98	0.97	0.96	0.95	0.93	0.91	0.89	0.87

Table 5.6. Adjustment Factor for Grade (22)

	DOWNHILL			LEVEL	UPHILL		
Grade, %	−6	−4	−2	0	+2	+4	+6
Grade Factor, f_g	1.03	1.02	1.01	1.00	0.99	0.98	0.97

Table 5.7. Adjustment Factor for Parking (22)

NO. OF LANES IN LANE GROUP	NO PKG	NUMBER OF PARKING MANEUVERS PER HOUR, N_m				
		0	10	20	30	40
1	1.00	0.90	0.85	0.80	0.75	0.70
2	1.00	0.95	0.92	0.89	0.87	0.85
3	1.00	0.97	0.95	0.93	0.91	0.89

Table 5.8. Adjustment Factor for Bus Blockage (22)

NO. OF LANES IN LANE GROUP	NUMBER OF BUSES STOPPING PER HOUR, N_B				
	0	10	20	30	40
1	1.00	0.96	0.92	0.88	0.83
2	1.00	0.98	0.96	0.94	0.92
3	1.00	0.99	0.97	0.96	0.94

Table 5.9. Adjustment Factor for Area Type (22)

TYPE OF AREA	FACTOR f_a
CBD	0.90
All other areas	1.00

Table 5.10. Adjustment Factor for Right Turns (22)

CASE	TYPE OF LANE GROUP	RIGHT-TURN FACTORS, f_{RT}
1	EXCLUSIVE RT LANE; PROTECTED RT PHASING	0.85
2	EXCLUSIVE RT LANE; PERMITTED RT PHASING	$f_{RT} = 0.85 - (\text{peds}/2,100)$ peds $\leq$ 1,700 $f_{RT} = 0.05$ peds > 1,700

Case 2 (continued):

No. of Conf. Pedestrians* (peds)	0	50 (Low)	100	200 (Mod)	300	400 (High)	500
Factor	0.85	0.83	0.80	0.75	0.71	0.66	0.61
No. of Conf. Pedestrians (peds)	**600**	**800**	**1,000**	**1,200**	**1,400**	**1,600**	**≥ 1,700**
Factor	0.56	0.47	0.37	0.28	0.18	0.05	0.05

Case 3 — EXCLUSIVE RT LANE; PROTECTED PLUS PERMITTED PHASING

$f_{RT} = 0.85 - (1 - P_{RTA})(\text{peds}/2,100)$

$f_{RT} = 0.05$ (minimum)

No. of Conf. Pedestrians (peds)	Prop. of RT Using Prot. Phase, P_{RTA}					
	0.00	0.20	0.40	0.60	0.80	1.00
0	0.85	0.85	0.85	0.85	0.85	0.85
50 (Low)	0.83	0.83	0.84	0.84	0.85	0.85
100	0.80	0.81	0.82	0.83	0.84	0.85
200 (Mod)	0.75	0.77	0.79	0.81	0.83	0.85
300	0.71	0.74	0.76	0.79	0.82	0.85
400 (High)	0.66	0.70	0.74	0.77	0.81	0.85
600	0.56	0.62	0.68	0.74	0.79	0.85
800	0.47	0.55	0.62	0.70	0.77	0.85
1,000	0.37	0.47	0.56	0.66	0.75	0.85
1,400	0.18	0.32	0.45	0.58	0.72	0.85
≥ 1,700	0.05	0.20	0.36	0.53	0.69	0.85

Case 4 — SHARED RT LANE; PROTECTED PHASING

$f_{RT} = 1.0 - 0.15 P_{RT}$

Prop. of RT in Lane, P_{RT}	0.00	0.20	0.40	0.60	0.80	1.00
Factor	1.00	0.97	0.94	0.91	0.88	0.85

Case 5 — SHARED RT LANE; PERMITTED PHASING

$f_{RT} = 1.0 - P_{RT}[0.15 + (\text{peds}/2,100)]$

$f_{RT} = 0.05$ (minimum)

No. of Conf. Pedestrians (peds)	Prop. of RT in Lane Group, P_{RT}					
	0.00	0.20	0.40	0.60	0.80	1.00
0	1.00	0.97	0.94	0.91	0.88	0.85
50 (Low)	1.00	0.97	0.93	0.90	0.86	0.83
100	1.00	0.96	0.92	0.88	0.84	0.80
200 (Mod)	1.00	0.95	0.90	0.85	0.80	0.75
400 (High)	1.00	0.93	0.86	0.80	0.73	0.66
600	1.00	0.91	0.83	0.74	0.65	0.56
800	1.00	0.89	0.79	0.68	0.58	0.47
1,000	1.00	0.87	0.75	0.62	0.50	0.37
1,400	1.00	0.84	0.67	0.51	0.35	0.18
≥ 1,700	1.00	0.81	0.62	0.42	0.23	0.05

Table 5.10. Continued (22)

RIGHT-TURN FACTORS, f_{RT}

Case 6 — Shared RT Lane; Protected Plus Permitted Phasing

$$f_{RT} = 1.0 - P_{RT}\,[0.15 + (\text{peds}/2{,}100)\,(1 - P_{RTA})]$$

$$f_{RT} = 0.05 \text{ (minimum)}$$

Prop. RT's Using Prot. Phase P_{RTA}	No. of Conf. Peds. (peds)	Prop. of RT's in Lane Group P_{RT} 0.00	0.20	0.40	0.60	0.80	1.00
0.00	All	Same as Case 5					
0.20	0	1.00	0.97	0.94	0.91	0.88	0.85
	50	1.00	0.97	0.93	0.90	0.86	0.83
	200	1.00	0.95	0.91	0.86	0.82	0.77
	400	1.00	0.94	0.88	0.82	0.76	0.70
	600	1.00	0.92	0.85	0.77	0.70	0.62
	1,000	1.00	0.89	0.79	0.68	0.58	0.47
	1,400	1.00	0.86	0.73	0.59	0.45	0.32
	≥ 1,700	1.00	0.81	0.62	0.42	0.23	0.20
0.40	0	1.00	0.97	0.94	0.91	0.88	0.85
	50	1.00	0.97	0.94	0.91	0.87	0.84
	200	1.00	0.96	0.92	0.88	0.83	0.79
	400	1.00	0.95	0.89	0.84	0.79	0.74
	600	1.00	0.94	0.87	0.81	0.74	0.68
	1,000	1.00	0.91	0.83	0.74	0.65	0.56
	1,400	1.00	0.89	0.78	0.67	0.56	0.45
	≥ 1,700	1.00	0.87	0.75	0.62	0.49	0.36
0.60	0	1.00	0.97	0.94	0.91	0.88	0.85
	50	1.00	0.97	0.94	0.90	0.87	0.84
	200	1.00	0.96	0.92	0.89	0.85	0.81
	400	1.00	0.95	0.91	0.86	0.82	0.77
	600	1.00	0.94	0.89	0.84	0.79	0.74
	1,000	1.00	0.93	0.86	0.80	0.73	0.66
	1,400	1.00	0.92	0.83	0.75	0.67	0.58
	≥ 1,700	1.00	0.91	0.81	0.72	0.62	0.53
0.80	0	1.00	0.97	0.94	0.91	0.88	0.85
	50	1.00	0.97	0.94	0.91	0.88	0.85
	200	1.00	0.97	0.93	0.90	0.86	0.83
	400	1.00	0.96	0.92	0.89	0.85	0.81
	600	1.00	0.96	0.92	0.88	0.83	0.79
	1,000	1.00	0.95	0.90	0.85	0.80	0.75
	1,400	1.00	0.94	0.89	0.83	0.77	0.72
	≥ 1,700	1.00	0.94	0.88	0.81	0.75	0.69
1.00	All	Same as Case 4					

Case 7 — Single Lane Approach

$$f_{RT} = 0.90 - P_{RT}\,[0.135 + (\text{peds}/2{,}100)]$$

$$f_{RT} = 0.05 \text{ (minimum)}$$

No. of Conf. Peds. (peds)	Prop. of RT's in Single Lane P_{RT} 0.00	0.20	0.40	0.60	0.80	1.00
0	1.00	0.87	0.85	0.82	0.79	0.77
50 (Low)	1.00	0.87	0.84	0.81	0.77	0.74
100	1.00	0.86	0.83	0.79	0.76	0.72
200 (Mod)	1.00	0.86	0.81	0.77	0.72	0.68
300	1.00	0.85	0.79	0.74	0.69	0.64
400 (High)	1.00	0.84	0.78	0.72	0.65	0.59
600	1.00	0.82	0.74	0.66	0.59	0.51
800	1.00	0.80	0.71	0.61	0.52	0.42
1,000	1.00	0.79	0.67	0.56	0.45	0.34
1,200	1.00	0.77	0.64	0.51	0.38	0.25
1,400	1.00	0.75	0.61	0.46	0.31	0.16
≥ 1,700	1.00	0.73	0.55	0.38	0.21	0.05

Case 8 — Double Exclusive RT Lane; Protected Phasing

0.75

Table 5.11. Adjustment Factor for Left Turns (22)

Case	Type of Lane Group	Left-Turn Factor, f_{LT}						
1	Exclusive LT Lane; Protected Phasing	0.95						
2	Exclusive LT Lane; Permitted Phasing	Special Procedure						
3	Exclusive LT Lane; Protected Plus Permitted Phasing	0.95[a]						
4	Shared LT Lane; Protected Phasing	$f_{LT} = 1.0/(1.0 + 0.05\ P_{LT})$						
		Prop. of LT's in Lane, P_{LT}	0.00	0.20	0.40	0.60	0.80	1.00
		Factor	1.00	0.99	0.98	0.97	0.96	0.95
5	Shared LT Lane; Permitted Phasing	Special Procedure						
6	Shared LT Lane; Protected Plus Permitted Phasing	$f_{LT} = (1{,}400 - V_o)/[(1{,}400 - V_o) + (235 + 0.435\ V_o)P_{LT}]$ $V_o \le 1{,}220$ vph; $f_{LT} = 1/[1 + 4.525\ P_{LT}]$ $V_o \geq 1{,}220$ vph						
		Opposing Volume V_o	Prop. of Left Turns, P_{LT}					
			0.00	0.20	0.40	0.60	0.80	1.00
		0	1.00	0.97	0.94	0.91	0.88	0.86
		200	1.00	0.95	0.90	0.86	0.82	0.78
		400	1.00	0.92	0.85	0.80	0.75	0.70
		600	1.00	0.88	0.79	0.72	0.66	0.61
		800	1.00	0.83	0.71	0.62	0.55	0.49
		1,000	1.00	0.74	0.58	0.48	0.41	0.36
		1,200	1.00	0.55	0.38	0.29	0.24	0.20
		≥ 1,220	1.00	0.52	0.36	0.27	0.22	0.18
7	Single Lane Approach	Special Procedures						
8	Double Exclusive LT Lane; Protected Phasing	0.92						

d) Computation of adjusted saturation flow rates can be calculated by multiplying the ideal saturation flow rate by the number of lanes in the lane group and by each of the eight adjustment factors using Equation 5.6.

$$S = S_o \cdot N \cdot f_w \cdot f_{HV} \cdot f_g \cdot f_p \cdot f_{bb} \cdot f_a \cdot f_{RT} \cdot f_{LT} \qquad (5.6)$$

S = Saturation flow rate for the subject lane group (vph)

S_o = Ideal saturation flow rate per lane, usually 1800 pcphgpl.

N = Number of lanes in the lane group

f_w = Adjustment factor for the width; given in Table 5.4.

f_{HV} = Adjustment factor for heavy vehicles in the traffic stream; given in Table 5.5.

f_g = Adjustment factor for approach grade; given in Table 5.6.

f_p = Adjustment factor for the existence of a parking lane adjacent to the lane group; given in Table 5.7

f_{bb} = Adjustment factor for the blocking effect of local buses stopping within the intersection area; given in Table 5.8.

f_a = Adjustment factor for area type given in Table 5.9.

f_{RT} = Adjustment factor for right turns in the lane group; given in Table 5.10.

f_{LT} = Adjustment factor for left turns in the lane group; given in Table 5.11.

5.1.1.4. Capacity Analysis Module

In these module, the aim is to find (v/c) ratio for approaches and for the intersection.

Steps that should be followed during capacity analyses are as follows;

a) Description of lane group.

b) The adjusted flow rate for each lane group.

c) The adjusted saturation flow rate for each lane group.

d) Computation of the flow ratio for each lane group.

e) Identification of critical lane groups. A critical lane group is the lane group with the highest flow in each phase or set of phase. The flow ratios for critical lane groups must be summed.

f) The green ratio for each lane group. The g/C ratio for each lane group. The g/C ratio is the effective green time divided by the cycle length. Where change intervals are in the range of 3 to 5 sec, the effective green time can be assumed to be equal to the actual green time.

g) Computation the capacity of each lane group. The capacity of each lane group is computed as the saturation flow rate times the green ratio by the use of Equation 5.7.

$$c_i = s_i \cdot (g/C) \tag{5.7}$$

h) Computation of v/c ratios for each lane group. The v/c ratio for the lane group is the ratio of adjusted flow to capacity.

$$x_i = \frac{v_i}{c_i} \tag{5.8}$$

i) Computation of the critical v/c ratio. The critical v/c ratio (x_c), is computed according to the Equation 5.9.

$$x_c = \frac{\sum_i (v/s)_{ci} \cdot C}{C-L} \tag{5.9}$$

$(v/s)_{ci}$ = The flow ratio for critical lane group.

C = Cycle length (sec)

L = Lost time (sec)

At the end of capacity analyses, the obtained critical x is the indicator of signalized intersection operation.

If the ratio x_c exceeds 1.00; it means the number of elements which effect capacity is not sufficient. In other words, geometric characteristics, phase plan, signal timing and cycle length are inadequate. But on the other hand, if x_c is less than 1.00; it means there is more available capacity for the traffic increase and all the elements which effect capacity are sufficient.

5.1.2. Level-of-Service Analyses

Level-of-service can only be determined according to average stopped delay. Delay is the lost of travel time and lost travel time because of driver discomfort and frustration.

As the capacity ratio, delay is also effected by many factors as given below;

a) Cycle length

b) Allocation of green time

c) Capacity ratio (v/c) of lane groups andapproach

Level-of-service should be determined by the help of volume adjustment, saturation flow rate and capacity analysis module results.

a) Lane group description.

b) Computation of first term delay (d_1) which is the uniform delay, i.e., the delay that occurs if arrival demand in the subject lane group is uniformly distributed over time. d_1 can be calculated by the use of equation 5.10.

$$d_1 = 0.38.C.\frac{[1-(g/C)]^2}{[1-(g/C).x]} \tag{5.10}$$

d_1, depends on (v/c) ratio, the green ratio (g/C), and the cycle length.

c) Computation of second term delay (d_2) which is the incremental delay of random arrivals over uniform arrivals, and for the additional delay due to cycle failure. d_2 can be calculated by the use of equation 5.11.

$$d_2 = 173.x^2.[(x-1)+\sqrt{(x-1)^2+(16.x/c)}] \tag{5.11}$$

d_2 is dependent on (v/c) ratio, the capacity for the lane group.

d) Computation of delay and level-of-service for each lane group. The delay for each lane group is the sum of the first and second term, multipled by the progression factor. Calculated by the use of equation 5.12.

$$d = (d_1 + d_2).PF \tag{5.12}$$

d_1 = First term delay (sec./veh.)

d_2 = Second term delay (sec./veh.)

PF = Progression factor can be obtained from Table 5.12

d = Average stopped delay per vehicle for the lane group (sec./veh.)

From the delay found the level-of-service is obtained according to Table 5.13.

e) Computation of the delay and level-of-service for each approach by the use of equation 5.13.

$$d_A = \frac{\sum_i d_i v_i}{\sum_i v_i} \tag{5.13}$$

d_A = Delay for approach A(sec/veh)

d_i = Delay for lane group i on approach A (sec./veh.)

v_i = Adjusted flow for lane group i (vph)

Table 5.12. Progression Adjustment Factor, PF (22)

TYPE OF SIGNAL	LANE GROUP TYPES	v/c RATIO, X	ARRIVAL TYPE				
			1	2	3	4	5
Pretimed	TH, RT	≤ 0.6	1.85	1.35	1.00	0.72	0.53
		0.8	1.50	1.22	1.00	0.82	0.67
		1.0	1.40	1.18	1.00	0.90	0.82
Actuated	TH, RT	≤ 0.6	1.54	1.08	0.85	0.62	0.40
		0.8	1.25	0.98	0.85	0.71	0.50
		1.0	1.16	0.94	0.85	0.78	0.61
Semiactuated	Main St. TH, RT	≤ 0.6	1.85	1.35	1.00	0.72	0.42
		0.8	1.50	1.22	1.00	0.82	0.53
		1.0	1.40	1.18	1.00	0.90	0.65
Semiactuated	Side St. TH, RT	≤ 0.6	1.48	1.18	1.00	0.86	0.70
		0.8	1.20	1.07	1.00	0.98	0.89
		1.0	1.12	1.04	1.00	1.00	1.00
	All LT*	all	1.00	1.00	1.00	1.00	1.00

* This category refers to exclusive LT lane groups with protected phasing only. When LT's are included in a lane group encompassing an entire approach use factor for the overall lane group type

Table 5.13. Level-of-Service Criteria for Signalized Intersections (22)

Level of Service	Stopped delay per vehicle (sec.)
A	≤ 5.0
B	5.1 to 15
C	15.1 to 25
D	25.1 to 40.0
E	40.1 to 60.0
F	> 60.0

f) Computation of the delay and level-of-service for the intersection. The average delay per vehicle for the intersection as a whole is found by adding the product of approach flow rate and approach delay for all approaches and dividing the sum by the total intersection flow rate. It can be calculated by the use of equation 5.14.

$$d_I = \frac{\sum_A d_A \cdot v_A}{\sum v_A} \tag{5.14}$$

d_I = Average delay per vehicle for the intersection (sec./veh.)

v_A = Adjusted flow for approach A (vph)

To find the level-of-service of the intersection, Table 5.13 should be used with the help of d_I.

5.1.3. Interpretation of Capacity and Level-of-Service Analyses Result

At the end of capacity and level-of-service analysis, it is obtained (v/c) ratios and delays for each lane group and for the intersection.

There are three outcomes from the analyses results;

a) v/c > 1.00 for the intersection

b) v/c < 1.00 for the intersection

c) v/c < 1.00 for the intersection but v/c > 1.00 for some lane groups.

The first outcome indicates that signal timing, cycle length, geometric characteristics or phase plan are not sufficient.

The second outcome means that the existing situation is adequate and there is available capacity for traffic flow increases.

For the last possibility, where critical (v/c) ratio is less than 1.00 but exceeds 1.00 for lane groups, it means that the green time was misappointed for the phases.

Before searching improvement types to increase capacity where it is not adequate, it's better to check the delay levels and combine the results of capacity and level-of-service module to understand the existing situation of the intersection.

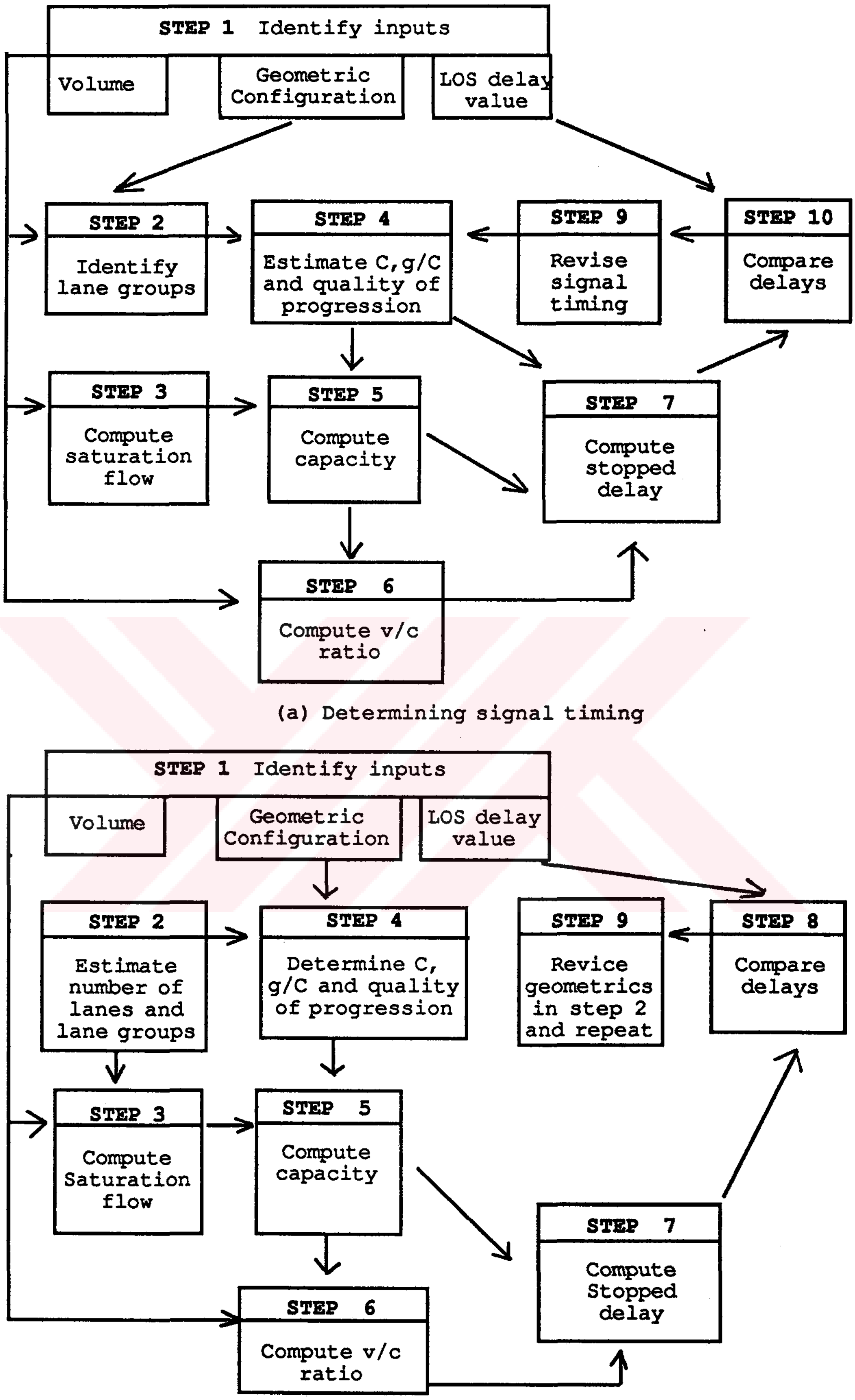
Sometimes, delay of the intersection is too long although the capacity ratio is less than 1.00. However, low delay is not the indicator of a capacity ratio less than 1.00.

Because delays are affected easily from progression factor, adequate capacity of the intersection can not reduce delays.

If the contributor of long delay is not the bad progression, than improvements such as lengthening the signal cycle, changing the signal phase plan or making changes in intersection geometry will be effective on reducing delays. But if the contributor of long delay is progression, the above improvements will have very small betterment.

When it's necessary to make changes in signalization or geometric conditions, the computation schemes shown in Figure 5.4 can be helpful.

Figure 5.4(a) and 5.4(b) should be followed for determining signal timing parameters and geometric parameters respectively.



(a) Determining signal timing

(b) Determining number of lanes

Figure 5.4. The Computation Scheme of Signalization and Geometrics (22)

CHAPTER VI

CASE STUDY

The objective of this case study is to apply the planning criteria of urban grade-separated intersection to an existing congested at-grade intersection and to evaluate the advantages provided by implementation of an urban GSI. In this case study, the use of a procedure for calculating the user benefits of highway improvement for sketch-planning analysis was studied.

One of the cities suffering severe traffic congestions is Ankara. Although Ankara was established in the same time with Turkish Republic and has most favorable terrain conditions, the traffic congestions have been increasing like İstanbul due to inadequate city planning measures.

In Ankara, one of the most congested intersections is İsmet İnönü Intersection. This intersection is located where Atatürk Boulevard and İsmet İnönü Boulevard cross each other. The existing condition of the intersection is shown in Figure 6.1.

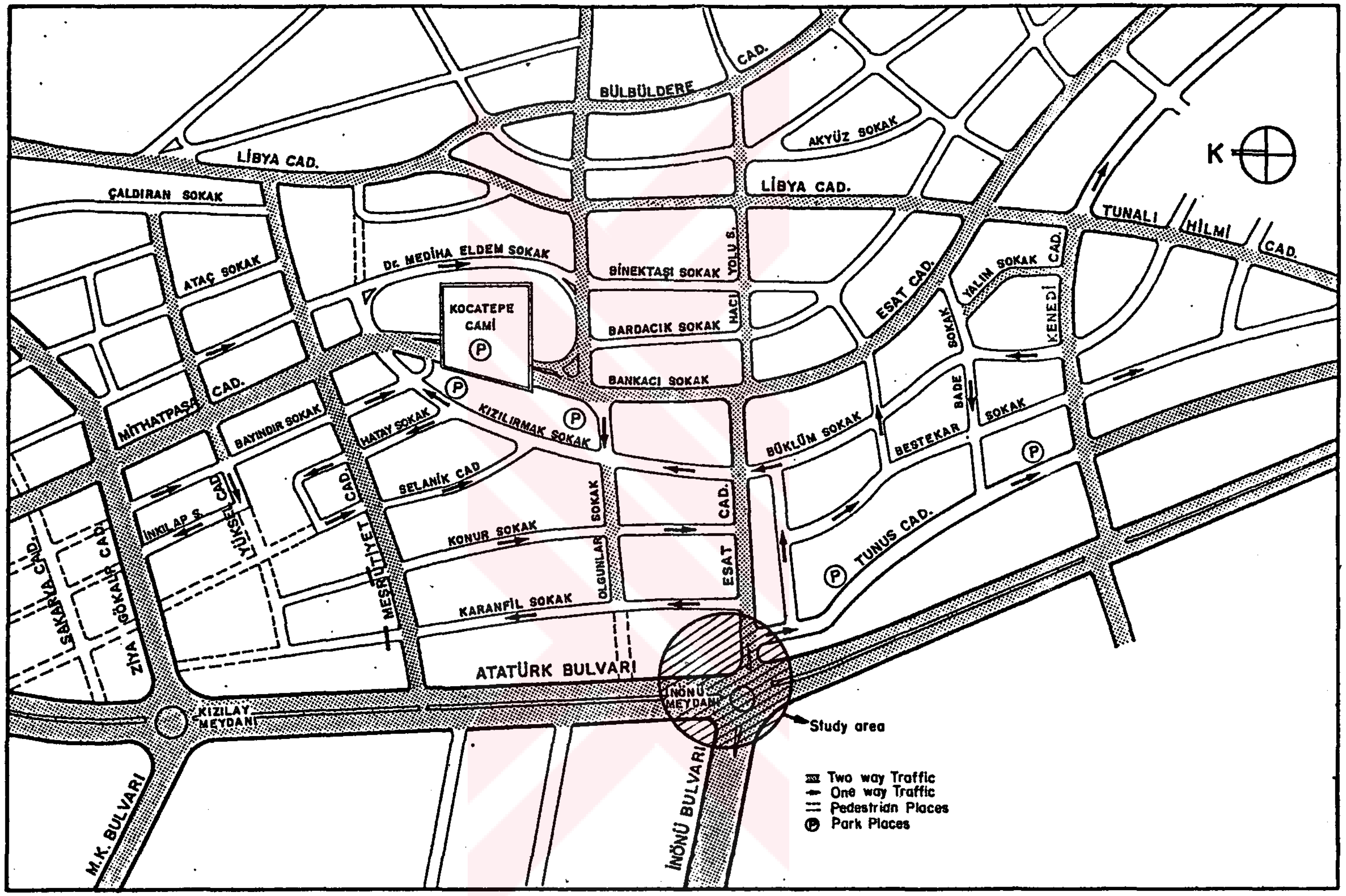


Figure 6.1. General Geometric Layout of İsmet İnönü Intersection

The problems of the intersection are; the big amount of left-turning movements which decrease the capacity of the intersection; in the vicinity of the intersection, there exists a large number of public buildings such as Töbank, Ministry of Agriculture and Forestry, General Directorate of Security, Turkish Parliament. Therefore, right-of-way problem is still valid. Due to subway construction and Kavaklıdere discharge channel laying under the intersection, an underpass alternative cannot be considered for a new design (8).

A capacity and level-of-service analysis for the existing condition of İnönü Intersection was conducted, followed by a comparison made between an urban GSI investment and an AGI in terms of delay, depreciation and fuel costs.

The necessary vehicle counting for capacity analysis which are made by General Directorate of Electricity, Gas and Bus Service of Ankara in August 17th 1990 is shown in Table 6.1, 6.2, 6.3 and 6.4. The counting interval is 0.8:15-09:15 (AM.). This interval is known to be the peak hour of the day for this intersection and the total flow in the peak hour is given as 10747 pcph.

Table 6.1. Esat Street Vehicle Counting Results

COUNTING DIRECTION: FROM ESAT STREET TO KIZILAY					
	VEHICLE TYPES				
Interval	Private Car	Taxi	Bus*	Minibus	Total(pc)
08:15-08:30	24	24	7	12	81
08:30-08:45	24	22	8	4	74
08:45-09:00	14	23	7	3	61
09:00-09:15	26	33	4	2	73

COUNTING DIRECTION: FROM ESAT STREET TO İNÖNÜ BOULEVARD					
	VEHICLE TYPES				
Interval	Private Car	Taxi	Bus*	Minibus	Total(pc)
08:15-08:30	93	30	53	86	368
08:30-08:45	100	36	42	52	314
08:45-09:00	132	55	27	26	294
09:00-09:15	432	39	32	29	271

COUNTING DIRECTION: FROM ESAT STREET TO ÇANKAYA					
	VEHICLE TYPES				
Interval	Private Car	Taxi	Bus*	Minibus	Total(pc)
08:15-08:30	12	5	0	0	17
08:30-08:45	11	7	0	0	18
08:45-09:00	22	18	1	0	43
09:00-09:15	13	8	3	0	30

* 1 Bus ≡ 3 passenger cars.

Table 6.2. Atatürk Boulevard Vehicle Counting Results

COUNTING DIRECTION: FROM ÇANKAYA TO ESAT STREET					
	VEHICLE TYPES				
Interval	Private Car	Taxi	Bus*	Minibus	Total(pc)
08:15-08:30	13	17	1	1	34
08:30-08:45	24	17	0	0	41
08:45-09:00	19	14	1	0	36
09:00-09:15	23	19	4	1	55

COUNTING DIRECTION: FROM ÇANKAYA TO KIZILAY					
	VEHICLE TYPES				
Interval	Private Car	Taxi	Bus*	Minibus	Total(pc)
08:15-08:30	280	116	34	15	513
08:30-08:45	256	138	38	10	518
08:45-09:00	248	150	28	9	491
09:00-09:15	248	145	27	12	486

COUNTING DIRECTION: FROM ÇANKAYA TO İNÖNÜ BOULEVARD					
	VEHICLE TYPES				
Interval	Private Car	Taxi	Bus*	Minibus	Total(pc)
08:15-08:30	153	49	6	22	242
08:30-08:45	203	47	7	33	304
08:45-09:00	227	74	3	39	349
09:00-09:15	157	75	9	50	309

1 Bus ≡ 3 passenger car

Table 6.3. Atatürk Boulevard Vehicle Counting Results

COUNTING DIRECTION: FROM KIZILAY TO ESAT STREET					
	VEHICLE TYPES				
Interval	Private Car	Taxi	Bus*	Minibus	Total(pc)
08:15-08:30	77	42	10	11	160
08:30-08:45	154	68	16	14	284
08:45-09:00	74	75	11	16	198
09:00-09:15	67	76	12	5	184

COUNTING DIRECTION: FROM KIZILAY TO ÇANKAYA					
	VEHICLE TYPES				
Interval	Private Car	Taxi	Bus*	Minibus	Total(pc)
08:15-08:30	126	115	31	15	349
08:30-08:45	130	134	28	20	368
08:45-09:00	106	146	24	19	343
09:00-09:15	174	227	27	33	515

COUNTING DIRECTION: FROM KIZILAY TO İNÖNÜ BOULEVARD					
	VEHICLE TYPES				
Interval	Private Car	Taxi	Bus*	Minibus	Total(pc)
08:15-08:30	110	76	32	7	289
08:30-08:45	97	63	18	3	217
08:45-09:00	83	60	10	-	173
09:00-09:15	52	41	10	1	124

1 Bus ≡ 3 passenger car

Table 6.4 İsmet İnönü Boulevard Vehicle Counting Results

COUNTING DIRECTION: FROM İNÖNÜ BOULEVARD TO KIZILAY					
	VEHICLE TYPES				
Interval	Private Car	Taxi	Bus*	Minibus	Total(pc)
08:15-08:30	142	59	22	3	270
08:30-08:45	96	56	16	6	206
08:45-09:00	99	36	20	4	199
09:00-09:15	100	52	19	7	216

COUNTING DIRECTION: FROM İNÖNÜ BOULEVARD TO ÇANKAYA					
	VEHICLE TYPES				
Interval	Private Car	Taxi	Bus *	Minibus	Total(pc)
08:15-08:30	76	38	0	3	117
08:30-08:45	85	31	4	7	135
08:45-09:00	79	46	6	3	146
09:00-09:15	75	41	1	4	123

COUNTING DIRECTION: FROM İNÖNÜ BOULEVARD TO ESAT STREET					
	VEHICLE TYPES				
Interval	Private Car	Taxi	Bus*	Minibus	Total(pc)
08:15-08:30	118	42	41	2	285
08:30-08:45	110	45	37	21	287
08:45-09:00	132	60	26	15	285
09:00-09:15	148	43	13	22	252

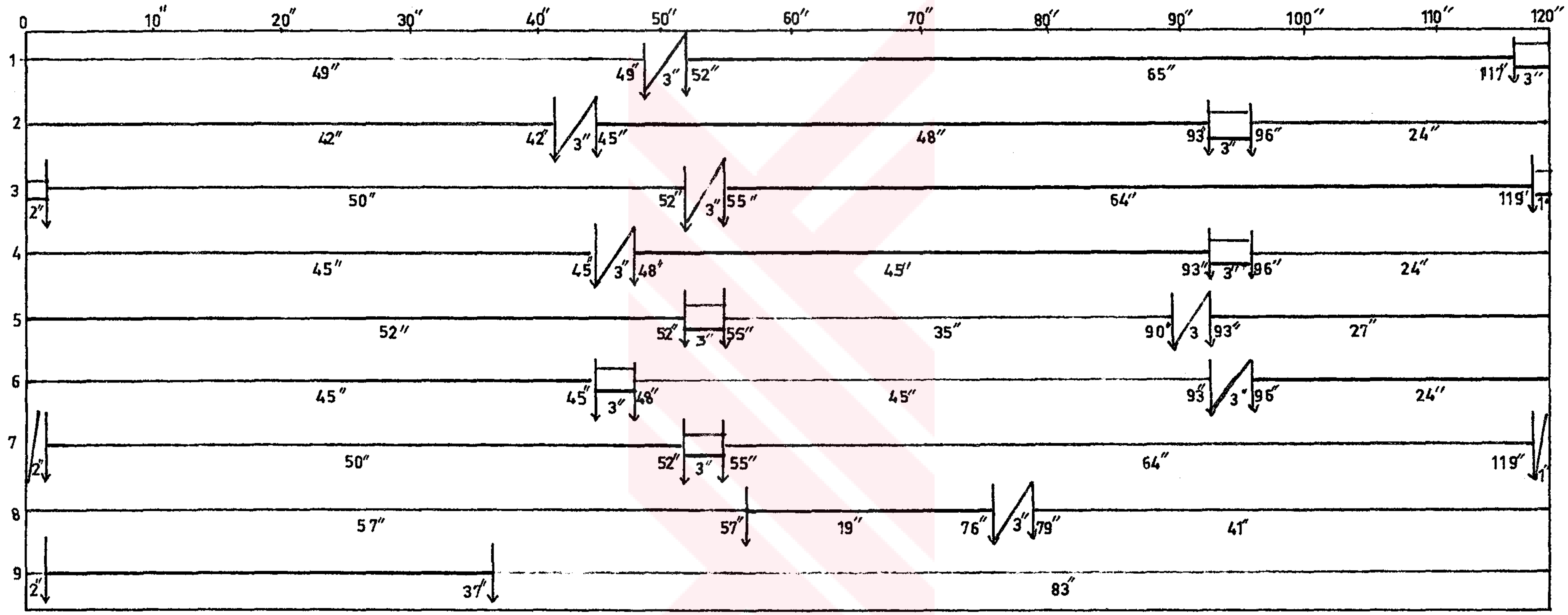
1 Bus ≡ 3 passenger car

The signalization sheme of İnönü Intersection is shown in Figure 6.2. This signalization scheme is for 120 sec. cycle length which is used during the peak morning hours. This scheme is prepared by Branch Directorate of Traffic in Ankara.

6.1. Capacity Analysis

The intersection of Atatürk Boulevard and İsmet İnönü Boulevard is illustrated in Table 6.5, which is the Input Module Worksheet for this calculation. It is a four-leg intersection with a two-phase, pretimed signalization based on 120 sec. cycle. Atatürk Boulevard has four lanes in each direction, while Esat Street has two and İnönü Boulevard has four lanes in each direction

The capacity analyses is made on a module-by-module basis following the steps given in Chapter 5.



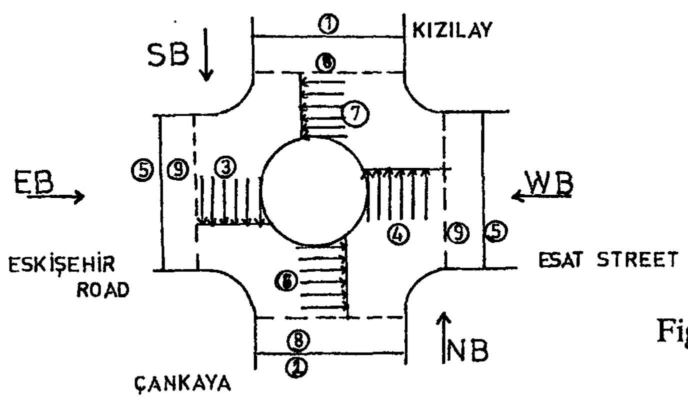


Figure 6.2. Signalization Scheme of İsmet İnönü Intersection

6.1.1. Input Module

The Input Module Worksheet for this calculation is shown in Table 6.5. All relevant volumes and geometric conditions are illustrated in the diagram on the upper half of the worksheet.

a) Traffic Volumes: Traffic volumes are shown in Tables 6.1, 6.2, 6.3 and 6.4.

b) Percent Grade: Percent grade is calculated on the intersection area; "+" indicates upgrades, while "-" indicates downgrades.

c) Percent Heavy Vehicles: A "heavy vehicles" is any vehicle with more than four tires touching the pavement. Each bus is assumed to be equal to three passenger cars. Percent heavy vehicle shown in the third column of Table 6.5 is calculated by interpolation.

d) Parking characteristics for the Approach: There exists no any adjacent parking lane at the intersection "N" is entered to the fourth column.

Table 6.5. Input Module Worksheet for İsmet İnönü Intersection

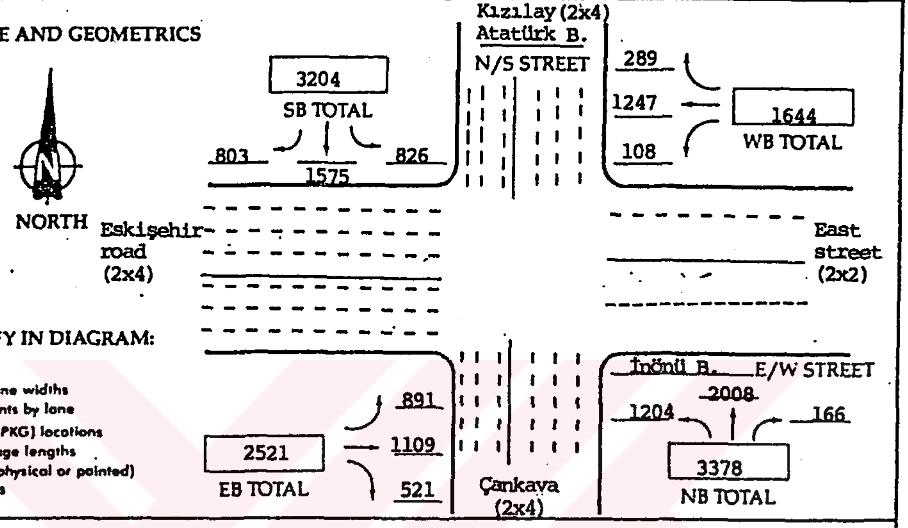
INPUT WORKSHEET

Intersection: Atatürk Boulvard and Ismet İnönü Bulvard Date: ____

Analyst: ____ Time Period Analyzed: 08:15–09:15 Area Type: ☒ CBD ☐ Other

Project No.: ____ City/State: Central City

VOLUME AND GEOMETRICS



IDENTIFY IN DIAGRAM:

1. Volumes
2. Lanes, lane widths
3. Movements by lane
4. Parking (PKG) locations
5. Bay storage lengths
6. Islands (physical or painted)
7. Bus stops

TRAFFIC AND ROADWAY CONDITIONS

Approach	Grade (%)	% HV	Adj. Pkg. Lane Y or N	Adj. Pkg. Lane N_m	Buses (N_B)	PHF	Conf. Peds. (peds./hr)	Pedestrian Button Y or N	Pedestrian Button Min. Timing	Arr. Type
EB	0	24.40	N	–	–	0.94	812	N	20.34	3
WB	+10.28	33.57	N	–	–	0.88	1064	N	15.76	1
NB	−3.98	14.03	N	–	–	0.96	1716	N	14.96	3
SB	+3.98	21.44	N	–	–	0.92	512	N	14.36	3

Grade: + up, − down
HV: veh. with more than 4 wheels
N_m: pkg. maneuvers/hr
N_B: buses stopping/hr
PHF: peak-hour factor
Conf. Peds: Conflicting peds./hr
Min. Timing: min. green for pedestrian crossing
Arr. Type: Type 1-5

PHASING

DIAGRAM								
Timing	G = Y + R =	G = Y + R =	G = Y + R =	G = Y + R =	G = Y + R =	G = Y + R =	G = Y + R =	G = Y + R =
Pretimed or Actuated								

Protected turns | Permitted turns | ------- Pedestrian | Cycle Length ____ Sec

e) Local Busses: Any bus-stop within 250 ft.(76 m.) of the intersection. The bus-stops in each approach are not violating the limit, so it should not be considered.

f) Peak Hour Factor: The peak hour factor is calculated by the use of Equation 5.2.

g) Number of Pedestrians: The number of pedestrians per hour using the pedestrian crossing is counted at the intersection area.

h) Pedestrian Controls: Pedestrian control at the intersection is performed by signalization, not by push-botton so that "N" is entered to the eighth column.

The ninth column of Table 6.5 shows the minimum green time required for a pedestrian to cross the street.

It is calculated by Equation 6.1;

$$G_p = 7.0 + W/4.0 - Y \tag{6.1}$$

G_p = Minimum green time, in sec.

W = Distance from the curb to the nearest pedestrian refuge island, in ft.

Y = Change interval (yellow+all red time), in sec. It's taken 3 sec.

i) Arrival Type: All the approaches has essentially random arrivals except West Boundary approach. WB approach has a bad quality of progression. This is the worst platoon condition. Consequently, using Table 5.2, arrival type 3 is chosen for EB, NB and SB and arrival type 1 for WB approach.

j) Phasing: The intersection has 2 phases. It is drawn by the help of Figure 6.2. The SB approach has early cut off.

6.1.2. Volume Adjustment Module

The Volume Adjustment Module Worksheet for this calculation is shown in Table 6.6.

Table 6.6. Volume Adjustment Module Worksheet for İnönü Intersection

VOLUME ADJUSTMENT WORKSHEET										
① Appr.	② Mvt.	③ Mvt. Volume (vph)	④ Peak Hour Factor PHF	⑤ Flow Rate v_p (vph) ③ ÷ ④	⑥ Lane Group	⑦ Flow rate in Lane Group v_g (vph)	⑧ Number of Lanes N	⑨ Lane Utilization Factor U	⑩ Adj. Flow v (vph) ⑦×⑨	⑪ Prop. of LT or RT P_{LT} or P_{RT}
EB	LT	891	0.94	948	↗	948	1	1	948	1
	TH	1109	0.94	1180	→	1180	2	1.05	1239	1
	RT	521	0.94	554	↘	554	1	1	554	1
WB	LT	108	0.88	123						
	TH	1247	0.88	1417	↗ → ↘	1868	2	1.05	1961	0.07 LT 0.18 RT
	RT	289	0.88	328						
NB	LT	1204	0.96	1254	↗	1254	1	1	1254	1
	TH	2008	0.96	2092	→	2092	2	1.05	2197	1
	RT	166	0.96	173	↘	173	1	1	173	1
SB	LT	826	0.92	898	↗	898	1	1	898	1
	TH	1575	0.92	1712	→	1712	2	1.05	1798	1
	RT	803	0.92	873	↘	873	1	1	873	1

Movement volumes are entered in column 3 from the Input Worksheet. Each is divided by the PHF to produce the movement flow rate indicated in column 5.

Because left turning volumes are high and number of lanes are sufficient for EB, NB and SB approaches having exclusive left turns. On the other hand, right turning volumes have special signalization so that EB, NB and SB approaches have exclusive right turn lanes. WB approach has two lanes in each direction, it is not adequate for the traffic volume, on the other hand WB has not heavy left and right turnings therefore it has shared lane operation.

For WB approach which does not have single lane, the total flow rate in lane group can be taken as the total as shown below;

$$V_g = V_{LT}+V_{TH}+V_{RT} = 123+1417+328 = 1868 \ vph$$

Number of lanes for each lane group is entered in column eight of Table 6.6 and the lane utilization factor is selected from Table 5.3. For one-lane approach, the value is 1.0 while, for the two lane approach, the value is 1.05. These are entered in column 9 of the worksheet, and are multiplied by the flow rates of column 7 to produce the adjusted flow rates of columns 10. It should be calculated by the use of Equation 5.3.

The proportion of left and right turns in the lane group is computed by taking the turning flow rates of column 5 and dividing by the total unadjusted flow in the lane group from column 7.

For WB approach, by the use of Equations 5.4 and 5.5;

P_{LT} for approach WB is 123/1868 = 0.07 LT

P_{RT} for approach WB is 328/1868 = 0.18 RT

6.1.3. Saturation Flow Rate Module

The worksheet for the saturation flow rate module is shown in Table 6.7.

Lane group descriptions are repeated in column 2 of the worksheet. The ideal saturation flow rate is assumed to be 1800 pcphgpl. The following columns contain the number of lanes in the lane group and

Table 6.7. Saturation Flow Rate Module Worksheet for İnönü Intersection

SATURATION FLOW ADJUSTMENT WORKSHEET												
LANE GROUPS				ADJUSTMENT FACTORS								
① Appr.	② Lane Group Movements	③ Ideal Sat. Flow (pcphgpl)	④ No. of Lanes N	⑤ Lane Width f_w	⑥ Heavy Veh f_{HV}	⑦ Grade f_g	⑧ Pkg. f_p	⑨ Bus Blockage f_{bb}	⑩ Area Type f_a	⑪ Right Turn f_{RT}	⑫ Left Turn f_{LT}	⑬ Adj. Sat. Flow Rate s (vphg)
EB	↗	1800	1	0.97	0.89	1	1	1	0.90	1	0.95	1329
	→	1800	2	0.97	0.89	1	1	1	0.90	1	1	2797
	↘	1800	1	0.97	0.89	1	1	1	0.90	0.85	1	1189
WB												
	⇉	1800	2	0.97	0.86	1.05	1	1	0.90	0.98	1	2781
NB	↗	1800	1	0.97	0.93	1.02	1	1	0.90	1	0.95	1416
	→	1800	2	0.97	0.93	1.02	1	1	0.90	1	1	2981
	↘	1800	1	0.97	0.93	1.02	1	1	0.90	0.85	1	1267
SB	↗	1800	1	0.97	0.90	0.98	1	1	0.90	1	0.95	1317
	→	1800	2	0.97	0.90	0.98	1	1	0.90	1	1	2772
	↘	1800	1	0.97	0.90	0.98	1	1	0.90	0.85	1	1178

all adjustment factors necessary for calculation of ideal saturation flow rates as summarized below:

a) Number of Lanes: WB lane group has two lanes. EB, NB and SB lane groups have two through, one left and one right turning lanes.

b) Lane Width Factor: This factor is selected from Table 5.4. For the 11 ft (3.34 m) lane, the value is 0.97.

c) Heavy Vehicle Factor: This factor is determined in Table 5.5. For 24.40 percent heavy vehicle on EB approach, the value is 0.89; for 33.57 percent on WB approach the value is 0.86, for 14.03 percent on NB approach the value is 0.93 and for 21.44 percent heavy vehicles on SB approach it is 0.90.

d) Grade Factor: This factor is selected from Table 5.6. For EB approach the value is 1.00 because the grade is level; for WB approach the grade is -10.28 and the factor is 1.05; for NB approach, the grade is -3.98 and the factor is 1.02; for SB approach the grade is +3.98 and the value is 0.98.

e) Parking Factor; Determined in Table 5.7, all values are 1.00, because no parking lane exists on any approach.

f) Bus Blockage Factor: This factor is taken from Table 5.8; because there are no bus stops within 250 ft (76 m), all values are equal to 1.00.

g) Area Type Factor: This factor is taken from Table 5.9, and is 0.90, because the intersection is located in a CBD.

h) Right-Turn Factor: This factor is taken from Table 5.10. Right turns signal phasing are protected on EB, NB and SB approach and provided by exclusive lanes (Case 1 in Table 5.10), while that from WB approach is protected and provided by shared lane (Case 4 in Table 5.10).

i) Left-Turn Factor: Left-turns signal phasing are protected EB, NB and SB approach and provided by exclusive lanes (Case 1 in Table 5.11).

The ideal saturation flow rate is multiplied by all adjustments, with the resulting saturation flow rates for prevailing conditions shown in the last column of the Saturation Flow Rate Module Worksheet.

6.1.4. Capacity Analysis Module

The Capacity Analysis Module Worksheet is shown in Table 6.8.

Lane group descriptions are again repeated in column 2 of the worksheet. In the subsequent two columns, the adjusted flow for each lane group is entered from the Volume Adjustment Worksheet (Table 6.6) and the saturation flow rate is entered from the Saturation Flow Rate Worksheet (Table 6.7). From these values, flow ratios are computed as v/s and entered in column 5 of the worksheet.

At this point, the critical lane groups are determined. There is one critical lane group for each signal phase and the lane group with the highest flow ratio, v/s, of those lane groups moving in each given signal phase is critical. NBLT has 0.88 flow ratio and this is critical for phase 1 and EBLT has 0.71 flow ratio which is critical value for phase 2. Thus the sum of critical lane flow ratios is 0.88+0.71 = 1.59.

Green ratios are entered in Column 6 of the worksheet, and are found by dividing the effective green time (obtained from Fig.6.2) for the lane group by the cycle length (120 sec.).

Table 6.8. Capacity Analysis Module Worksheet for İnönü Intersection

CAPACITY ANALYSIS WORKSHEET								
LANE GROUP		③ Adj. Flow Rate v (vph)	④ Adj. Sat. Flow Rate s (vphg)	⑤ Flow Ratio v/s ③ ÷ ④	⑥ Green Ratio g/C	⑦ Lane Group Capacity c (vph) ④ × ⑥	⑧ v/c Ratio X ③ ÷ ⑦	⑨ Critical ? Lane Group
① Appr.	② Lane Group Movements							
EB	↗	948	1329	0.71	0.38	505	1.87	✓
	→	1239	2797	0.44	0.38	1063	1.16	
	↘	554	1189	0.47	0.82	975	0.57	
WB								
	⇉	1961	2781	0.70	0.53	1474	1.33	
NB	↗	1254	1416	0.88	0.58	821	1.53	✓
	→	2197	2981	0.73	0.58	1728	1.27	
	↘	173	1267	0.14	0,71	890	0.20	
SB	↗	898	1317	0.68	0,42	553	1.62	
	→	1798	2772	0.65	0,42	1164	1.54	
	↘	873	1178	0.74	0,71	836	1.04	

Cycle Length, C 120 sec

Lost Time Per Cycle, L 6 sec

$\Sigma (v/s)_{ci} =$ 1.59

$X_c = \frac{\Sigma (v/s)_{ci} \times C}{C - L} =$ 1.67

Intersection values are computed at the bottom of TABLE 6.8. The change intervals are 3 sec. per phase. The cycle length is 120 sec, with 6 sec. of lost time per cycle.

The critical v/c ratio, X_c is computed by equation 5.9 shown on the worksheet, as

$$X_c = 1.59.(120)/(120\text{-}6) = 1.67$$

Table 6.9. Green Ratios for İnönü Intersection (C=120 sec.)

Lane Group	Effective Green Time for the Lane Group (sec.)	Green Ratios (g/C)
EBLT, EBTH	45	0.38
EBRT	98	0.82
WB	64	0.53
NBLT, NBTH	69	0.58
NBRT	85	0.71
SBLT, SBTH	50	0.42
SBRT	85	0.71

Because X_c exceeds 1.00 it means one or more of the critical lane groups is oversaturated. It is an indication that the intersection design, cycle length, phase plan or signal timing is inadequate.

6.2. Level-of-Service Analyses

The Level-of-Service Module Worksheet is shown in Table 6.13. Lane group descriptions are entered in Column 2.

Values of X, g/C, C and c are entered into the appropriate columns as these will be needed to compute delay. They are obtained from the Capacity Analysis Worksheet.

The first-term delay is computed using Equation 5.10 and given in Table 6.10.

Table 6.10. First-Term Delays for İnönü Intersection (C=120 sec.)

Lane Group	Green Ratio (g/C)	v/c Ratio (x)	First-Term Delay (sec./veh.) $\left(d_1 = \frac{0,38.C.[1-(g/C)]^2}{[1-(g/C)(x)]}\right)$
EBLT	0.38	1.87	60.57
EBTH	0.38	1.16	31.34
EBRT	0.82	0.57	2.77
WB	0.53	1.33	34.13
NBLT	0.58	1.53	71.44
NBTH	0.58	1.27	30.54
NBRT	0.71	0.20	4.46
SBLT	0.42	1.62	47.99
SBTH	0.42	1.54	43.43
SBRT	0.71	1.04	14.66

The second term delay is computed from the equation 5.11 and given in Table 6.11.

Table 6.11. Second-Term Delays for İnönü Intersection (C=120 sec.)

Lane Group	v/c Ratio (x)	Capacity (Vehicle) (c)	Second-Term Delay (sec./veh.) ($d_2=173\ x^2[(x-1)+\sqrt{(x-1)^2+(16x/c)}]$)
EBLT	1.87	505	1072.84
EBTH	1.16	1063	85.55
EBRT	0.57	975	0.60
WB	1.33	1474	208.46
NBLT	1.53	821	440.38
NBTH	1.27	1728	156.52
NBRT	0.20	890	0.016
SBLT	1.62	553	579.65
SBTH	1.54	1164	451.01
SBRT	1.04	836	34.92

These values are entered in the appropriate columns of the Level-of-Service Worksheet. Progression factors are selected from Table 5.12. For the EB, NB and SB approaches, 1.00 (arrival type 3). For the WB approach the factor is 1.40 (arrival type 1).

The delay in each lane group is computed by the use of equation 5.12 and given in Table 6.12.

Table 6.12. Delay in Each Lane Group for İnönü Intersection

Lane Group	First-Term Delay (sec./veh)(d_1)	Second-Term Delay (sec./veh.) (d_2)	PF	Delay (sec./veh.) [$D=(d_1+d_2).PF$]
EBLT	60.57	1072.84	1	1113.41
EBTH	31.34	85.55	1	116.89
EBRT	2.77	0.60	1	3.37
WB	34.13	208.46	1.40	339.63
NBLT	71.44	440.38	1	511.82
NBTH	30.54	156.52	1	187.06
NBRT	4.46	0.016	1	4.48
SBLT	47.99	579.65	1	627.64
SBTH	43.43	451.01	1	494.44
SBRT	14.66	34.92	1	49.58

Table 6.13. Level-of-Service Module Worksheet for İnönü Intersection

LEVEL-OF-SERVICE WORKSHEET

Lane Group		First Term Delay				Second Term Delay				Total Delay & LOS		
① Appr.	② Lane Group Movements	③ v/c Ratio X	④ Green Ratio g/C	⑤ Cycle Length C (sec)	⑥ Delay d_1 (sec/veh)	⑦ Lane Group Capacity c (vph)	⑧ Delay d_2 (sec/veh)	⑨ Progression Factor PF	⑩ Lane Group Delay (sec/veh) (⑥+⑧) × ⑨	⑪ Lane Group LOS	⑫ Approach Delay (sec/veh)	⑬ Appr. LOS
EB	↗	1.87	0.38	120	60.57	505	1072.84	1	1133.41	F	445.51	F
	→	1.16	0.38	120	31.34	1063	85.55	1	116.89	F		
	↘	0.57	0.82	120	2.77	975	0.60	1	3.37	A		
WB											339.63	F
	⇉	1.33	0.53	120	34.13	1474	208.46	1,40	339.63	F		
NB	↗	1.53	0.58	120	71.44	821	440.38	1	511.82	F	290.72	F
	→	1.27	0.58	120	30.54	1728	156.52	1	187.06	F		
	↘	0.20	0.71	120	4.46	890	0.016	1	4.48	A		
SB	↗	1.62	0.42	120	47.99	553	579.65	1	627.64	F	419.13	F
	→	1.54	0.42	120	43.43	1164	451.01	1	494.44	F		
	↘	1.04	0.71	120	14.66	836	34.92	1	49.58	E		

Intersection Delay 373 sec/veh   Intersection LOS F

The average stopped delay per vehicle for each approach is calculated by the use of Equation 5.13.

The average stopped delay per vehicle for the intersection as a whole is computed as a weighted average of the values for each approach by the use of Equation 5.14.

Intersection Delay= [(2741x445.51)+(1961x339.63)+

(3624x290.72)+(3569x419.13)]/(2741+

1961+3624+3569)=372.98≅373 sec./veh.

Level-of-service may be assigned by comparing the computed delay values with the criteria of Table 5.13.

These results reveal that the intersection as a whole operates at LOS.F. The EB approach having the highest v/c ratio also has the highest delay.

At İnönü Intersection both delay levels and v/c ratios are unacceptable, the situation is most critical. Delay is already high and demand is over capacity. In such situations, the delay may be rapidly effected by small fluctuations in demand.

Usually in these cases geometric and signal design improvements should be reviewed.

Because delay levels are unacceptable for all lane groups, it is necessary to reconsider the phase plan or allocation of green time. On the other hand the cycle length can not be longer than 120 sec. and the right-of-way problem makes the geometric improvements non applicable.

6.3. Application of Grade-Separated Intersection

The improvement type considered in this analyses is the replacement of İsmet İnönü AGI with an urban GSI. The urban GSI is shown in Figure 6.3. It consists of the same number of lanes per lane group per approach as the İsmet İnönü AGI. All lanes are 11 ft (3.34 m) in width.

By the use of overpass shown in Figure 6.3, the through traffic on Atatürk Boulevard can flow without any stop and delay at İsmet İnönü Intersection. On the other hand, left turns from Çankaya to Eskişehir road and from Kızılay to Esat Street are provided with ramps. EB and WB

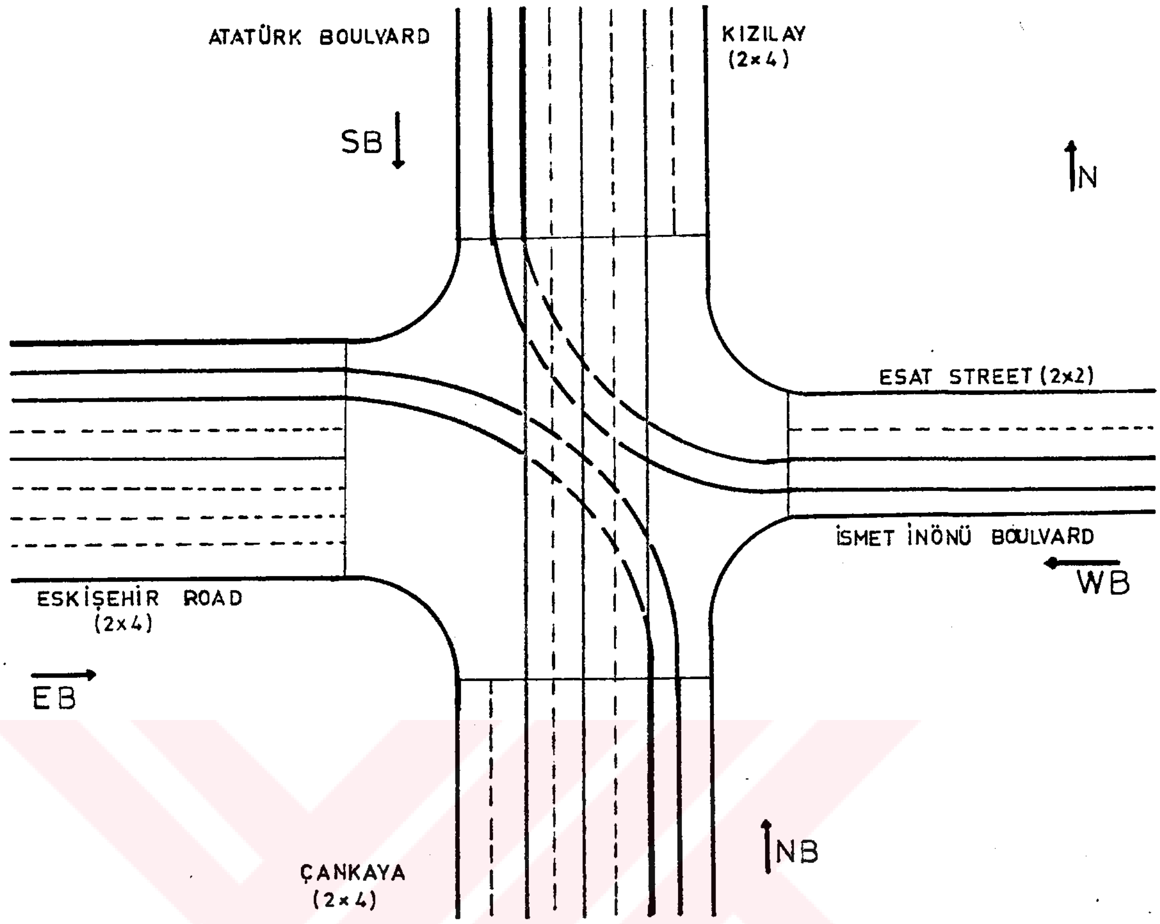


Figure 6.3. General Geometric Layout of Proposed Urban GSI.

approaching flows are again provided with a signalized intersection. The capacity and level-of-service analyses of this at-grade intersection are also to be determined.

6.3.1. Capacity Analyses

Traffic flows for the intersection of EB and WB approaches are illustrated in Table 6.14 which gives the Input Module Worksheet for this calculations. It is a two leg intersection. İsmet İnönü Boulevard has four lanes in each direction while Esat Street has only two in each direction.

6.3.1.1. Input Module

Input Module variables are all the same with the inputs on Table 6.5 except the arrival type of the WB approach. It is simply because of the longer green ratio which will cause a better quality of progression. Arrival type 3 is chosen for WB approach from Table 5.2.

6.3.1.2. Volume Adjustment Module

The Volume Adjustment Module Worksheet for this calculation is shown in Table 6.15. The values are the same with that of the EB and WB approaches as given in Table 6.6.

Table 6.14. Input Module Worksheet for the At-Grade Intersection Part of the Proposed Urban Grade-Separated Intersection on İsmet İnönü Intersection

INPUT WORKSHEET

Intersection: İsmet İnönü Intersection Date: ____

Analyst: ____ Time Period Analyzed: 08:15-09:15 Area Type: ☒ CBD ☐ Other

Project No.: ____ City/State: Central City

VOLUME AND GEOMETRICS

NORTH

SB TOTAL

Atatürk B. N/S STREET

289
1247
108
1644 WB TOTAL

Eskişehir road (1x4)

East Street (1x2)

IDENTIFY IN DIAGRAM:

1. Volumes
2. Lanes, lane widths
3. Movements by lane
4. Parking (PKG) locations
5. Bay storage lengths
6. Islands (physical or painted)
7. Bus stops

891
1109
521
2521 EB TOTAL

İnönü B. E/W STREET

NB TOTAL

TRAFFIC AND ROADWAY CONDITIONS

Approach	Grade (%)	% HV	Adj Pkg. Lane Y or N	Adj Pkg. Lane N_m	Buses (N_B)	PHF	Conf. Peds. (peds./hr)	Pedestrian Button Y or N	Pedestrian Button Min. Timing	Arr. Type
EB	0	24.40	N	-	-	0.94	812	N	20.04	3
WB	-10.28	33.57	N	-	-	0.88	1064	N	15.76	3
NB										
SB										

Grade: + up, − down
HV: veh. with more than 4 wheels
N_m: pkg. maneuvers/hr
N_B: buses stopping/hr
PHF: peak-hour factor
Conf. Peds: Conflicting peds./hr
Min. Timing: min. green for pedestrian crossing
Arr. Type: Type 1-5

PHASING

DIAGRAM								
Timing	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =
Pretimed or Actuated								

Protected turns | Permitted turns | Pedestrian | Cycle Length ____ Sec

Table 6.15. Volume Adjustment Module Worksheet for the At-Grade Intersection Part of the Proposed Urban Grade-Separated Intersection on İsmet İnönü Intersection

VOLUME ADJUSTMENT WORKSHEET										
① Appr.	② Mvt.	③ Mvt. Volume (vph)	④ Peak Hour Factor PHF	⑤ Flow Rate v_p (vph) ③ ÷ ④	⑥ Lane Group	⑦ Flow rate in Lane Group v_g (vph)	⑧ Number of Lanes N	⑨ Lane Utilization Factor U	⑩ Adj. Flow v (vph) ⑦×⑨	⑪ Prop. of LT or RT P_{LT} or P_{RT}
EB	LT	891	0.94	948	↗	948	1	1	948	1
	TH	1109	0.94	1180	→	1180	2	1.05	1239	1
	RT	521	0.94	554	↘	554	1	1	554	1
WB	LT	108	0.88	123						
	TH	1247	0.88	1417	⇉	1868	2	1.05	1961	0.07 LT 0.18 RT
	RT	289	0.88	328						
NB	LT									
	TH									
	RT									
SB	LT									
	TH									
	RT									

6.3.1.3. Saturation Flow Rate Module

The worksheet for the saturation flow rate module is shown in Table 6.16. The values are again the same with that of the EB and WB approaches as given in Table 6.7.

6.3.1.4. Capacity Analyses Module

The Capacity Analyses Module Worksheet is shown in Table 6.17. The adjusted flow rate (v), the adjusted saturation flow rate (s) and flow ratio (v/s) are the same with that of the EB and WB approaches as given in Table 6.8.

The cycle length is assumed to be 120 sec. and the green time is calculated using Equation 6.2:

$$g = (v/s).(C/x_i) \qquad (6.2)$$

g_{EB} = 0.71 (120/1.69) = 50 sec.

g_{WB} = 0.70 (120/1.33) = 64 sec.

Table 6.16. Saturation Flow Rate Module Worksheet for the At-Grade Intersection Part of the Proposed Urban Grade-Separated Intersection on İsmet İnönü Intersection

SATURATION FLOW ADJUSTMENT WORKSHEET												
LANE GROUPS				ADJUSTMENT FACTORS								
① Appr.	② Lane Group Movements	③ Ideal Sat. Flow (pcphgpl)	④ No. of Lanes N	⑤ Lane Width f_w	⑥ Heavy Veh f_{HV}	⑦ Grade f_g	⑧ Pkg. f_p	⑨ Bus Blockage f_{bb}	⑩ Area Type f_a	⑪ Right Turn f_{RT}	⑫ Left Turn f_{LT}	⑬ Adj. Sat. Flow Rate s (vphg)
EB		1800	1	0.97	0.89	1	1	1	0.90	1	0.95	1329
		1800	2	0.97	0.89	1	1	1	0.90	1	1	2797
		1800	1	0.97	0.89	1	1	1	0.90	0.85	1	1189
WB												
		1800	2	0.97	0.86	1.05	1	1	0.90	0.98	1	2781
NB												
SB												

Table 6.17. Capacity Analyses Module Worksheet for the At-Grade Intersection Part of the Proposed Urban Grade Separated Intersection on İsmet İnönü Intersection

CAPACITY ANALYSIS WORKSHEET

LANE GROUP ① Appr.	LANE GROUP ② Lane Group Movements	③ Adj. Flow Rate v (vph)	④ Adj. Sat. Flow Rate s (vphg)	⑤ Flow Ratio v/s ③ ÷ ④	⑥ Green Ratio g/C	⑦ Lane Group Capacity c (vph) ④×⑥	⑧ v/c Ratio X ③ ÷ ⑦	⑨ Critical ? Lane Group
EB	↗	948	1329	0.71	0.42	558	1.69	✓
	→	1239	2797	0.44	0.42	1175	1.05	
	↘	554	1189	0.47	0.42	499	1.11	
WB								
	⇉	1961	2781	0.70	0.53	1474	1.33	✓
NB								
SB								

Cycle Length, C 120 sec

Lost Time Per Cycle, L 6 sec

$\sum_i (v/s)_{ci} = 1.41$

$X_c = \frac{\sum (v/s)_{ci} \times C}{C - L} = 1.48$

The green ratios are calculated as follows and entered into Column 6;

$g_{EB}/C = 50/120 = 0.42$ sec.

$g_{WB}/C = 64/120 = 0.53$ sec.

Lane group capacities are computed by multiplying the green ratio, g/C, by the saturation flow rate for the lane group, s. Finally the v/c ratio, x, for each lane group is computed by dividing the adjusted lane group flow rate, v, by the capacity of the lane group, c.

Intersection values are computed at the bottom of Table 6.17. The lost time is 3 sec. per phase. The total lost is 6 sec. per cycle.

The critical v/c ratio, X_c, is computed using Equation 5.9. This equation is also given in the worksheet.

$X_c = 1.41(120)/(120-6) = 1.484$

6.3.2. Level-of-Service Analyses

The level-of-Service Module Worksheet is shown in Table 6.20. Lane group descriptions are inserted in Column 2.

Values of X, g/C, C and c are entered into the appropriate columns. They are obtained from the capacity analyses worksheet.

The first-term delay is computed from the Equation 5.10 and given in Table 6.18.

Table 6.18. First-Term Delays for the Proposed Urban GSI (C=120 sec)

Lane Group	Green Ratio (g/C)	v/c Ratio (x)	First-Term Delay (sec./veh.) $\left(d_1 = \frac{0,38.C.[1-(g/C)]^2}{[1-(g/C).x]}\right)$
EBLT	0.42	1.69	52.86
EBTH	0.42	1.05	27.44
EBRT	0.42	1.11	28.73
WB	0.53	1.33	34.13

The second term is computed from the Equation 5.11 and given in Table 6.19.

Table 6.19. Second-Term Delays for the Proposed Urban GSI (C=120 sec)

Lane Group	v/c (x)	Capacity (vehicle) (c)	Second-Term Delay (sec./veh.) ($d_2=173.x^2[(x-1)+\sqrt{(x-1)^2+(16.x/c)}]$
EBLT	1.69	558	698.79
EBTH	1.05	1175	34.26
EBRT	1.11	499	69.99
WB	1.33	1474	208.46

These values are entered in the appropriate columns of the level-of-service Worksheet.

Progression factors are selected from Table 5.12. For both EB and WB the factor is 1.00 (arrival type 3).

Table 6.20. Level-of-Service Module Worksheet for the At-Grade Intersection Part of the Proposed Urban Grade-Separated Intersection on İsmet İnönü Intersection

LEVEL-OF-SERVICE WORKSHEET												
Lane Group		First Term Delay				Second Term Delay				Total Delay & LOS		
① Appr.	② Lane Group Movements	③ v/c Ratio X	④ Green Ratio g/C	⑤ Cycle Length C (sec)	⑥ Delay d_1 (sec/veh)	⑦ Lane Group Capacity c (vph)	⑧ Delay d_2 (sec/veh)	⑨ Progression Factor PF	⑩ Lane Group Delay (sec/veh) (⑥+⑧) × ⑨	⑪ Lane Group LOS	⑫ Approach Delay (sec/veh)	⑬ Appr. LOS
EB	↗	1.69	0.42	120	52.86	558	698.79	1	751.65	F	307.80	F
	→	1.05	0.42	120	27.44	1175	34.26	1	61.70	F		
	↘	1.11	0.42	120	28.73	499	69.99	1	98.72	F		
WB											242.58	F
	↗→ →↘	1.33	0.53	120	34.13	1474	208.46	1	242.58	F		
NB												
SB												

Intersection Delay 280.60 sec/veh Intersection LOS F

The delay in each lane group is computed using Equation 5.12 and given in Table 6.21.

Table 6.21. Delay in Each Lane Group for the Proposed Urban GSI

Lane Group	First-Term Delay (sec./veh.) (d_1)	Second-Term Delay (sec./veh.) (d_2)	PF	Delay (sec./veh.) [$D=(d_1+d_2).PF$]
EBLT	52.86	698.79	1	751.65
EBTH	27.44	34.26	1	61.70
EBRT	28,73	69,99	1	98.72
WB	34.13	208.46	1	242.58

The average stopped delay per vehicle for each approach is calculated by the use of Equation 5.13.

$$\text{Delay (EB)} = [(948.751.65)+(1239.61.7)+(554.98.72)]/[948+1239+554]$$

$$\text{Delay (EB)} = 307.8 \text{ sec./veh.}$$

$$\text{Delay (WB)} = (1961.242.58)/1961=242.58 \text{ sec./veh.}$$

The average stopped delay per vehicle for the intersection as a whole is computed as a weighted average of the values for each approach by the use of Equation 5.14.

$$\text{Delay (Intersection)} = [(2741 \cdot 307.8) + (1961 \cdot 242.58)]/(2741+1961)$$

$$\text{Delay (Intersection)} = 280.60 \text{ sec./veh.}$$

Levels-of-service may be assigned by comparing the computed delay values with the criteria summarized in Table 5.13.

These calculations and results show that the intersection, as a whole, operates at LOS.F. But the intersection delay is decreased. This delay could not be reduced by the application of shorter cycle length and/or longer green time.

An economical analyses is necessary to recognize the user benefits from the implementation of the urban GSI.

6.4. Economical Analyses

The urban GSI evaluated in this study provides substantial economic benefit as a replacement for the İsmet İnönü AGI. The benefits examined in this case study are those obtained from delay, fuel consumption and depreciation.

6.4.1. Delay Loss

a) Before the implementation of urban GSI, the delay loss could be calculated as follows;

Traffic composition and passenger amounts in İsmet İnönü Intersection is given in Table 6.22.

Total passenger car passing through the İsmet İnönü Intersection in the peak hour traffic is 10747 pcph. Number of passengers is given in Table 6.23.

Table 6.22. Traffic Composition and Passenger Amounts at İsmet İnönü Intersection in the Peak Hour Traffic

Traffic Composition and Passenger Amount.			
Vehicle Type		Percent in the Flow (%)	Passenger Amounth
Bus		22	30.00
Minibus		6	10.00
Passenger Cars	Private	46	2.00
	Taxi	26	2.00

Table 6.23. Number of Passengers Through İsmet İnönü Intersection

VEHICLE TYPE	PASSENGER AMOUNT
Bus	10747.0.22.30=70930
Minibus	10747.0.06.10=6448
Car	10747.0.72.2=15476
Total	92854 passanger

The number of passenger crossing the intersection in one hour is 92854. If we assume that the intersection has the same amount of passenger traffic during 10 hour (1 day), 928540 passenger will use the intersection in one day.

According to the Table 6.13, each passanger loose 327.98 sec. while passing from the intersection. If we assume that 1/3 of the total passengers have been working and their salary is about 414 000 TL, the lost per second is;

414 000/30/24/60/60 = 0.16 TL./ sec.

The worth of 372.98 second is;

372,98 x 0.16 = 59.68 ≅ 60 TL.

Each passenger looses time equivalent to 60 TL. while passing from the intersection.

In one day the total lost of all passengers is;

928540 x 60 = 55 415 267 TL.

If we assume one year is equal to 300 working days, the total loss in one year is;

$$55\ 415\ 267 \times 300 = 16.62.10^9 \text{ TL.}$$

Each passenger is loosing 327.98 second while passing from the İsmet İnönü Intersection. This is a lost for national economy.

b) After the urban GSI, the delay lost can be calculated as follows;

Table 6.24. Traffic Composition and Passenger Amounts for the At-Grade Intersection Part of the Urban Grade-Separated Intersection on İsmet İnönü Intersection

Traffic Composition and Passenger Amount			
Vehicle Type		Percent in the Flow (%)	Passenger Amount
Bus		28	30.00
Minibus		8	10.00
Passenger Cars	Private	44	2.00
	Taxi	20	2.00

Total passenger car that pass through the intersection in the peak hour traffic is 4165 vph. The amount of passenger is shown in Table 6.25.

Table 6.25. Number of Passengers using the At-Grade Intersection Part of the Urban Grade-Separated Intersection on İsmet İnönü Intersection

VEHICLE TYPE	PASSENGER AMOUNT
Bus	4165x0.28.30=34986
Minibus	4165x0.08·10=3332
Car	4165x0·64·2=5331
Total	43649 passenger

The number of passengers using the intersection in one hour is 43649. If we assume that the intersection has the same amount of traffic during 10 hour (1 day), 436490 passenger will use the intersection in one day.

According to the Table 6.20, each passenger loose 280·60 sec. while passing the intersection. If we assume that 1/3 of the passenger work and their salary is 414 000 TL. The salary for one sec. is 0.16 TL.

The worth of 280.60 second is

$$280.60 \times 0.16 = 44.90 \text{ TL} \subsetneq 45 \text{ TL.}$$

Each passenger looses time equivalent to 45 TL. approximately while crossing the intersection.

In one day the total loss of all passengers is;

$$436\ 490 \times 45 = 19\ 598\ 401 \text{ TL.}$$

If we assume 1 year is equal to 300 day, the total loss in one year is,

$$19\ 598\ 401 \,.\, 300 = 5\ 879\ 520\ 300 \text{ TL.}$$

6.4.2. Depreciation Lost

a) Before the urban GSI, the depreciation loss can be calculated as follows;

In one hour, 10747 x 0.22 = 2364 buses

10747 x 0.06 = 645 minibuses

10747 x 0.46 = 4945 private cars

10747 x 0.26 = 2794 taxis

pass through the intersection.

Depreciation loss of vehicles is given in Table 6.26.

Table 6.26. Depreciation Loss of Vehicles (14)

VEHICLE TYPE		HOURLY DEPRECIATION (TL)
Bus		37.90
Minibus		20.00
Car	Private	17.54
	Taxi	17.54
Total		92.98

Depreciation loss in 372.98 sec. at İsmet İnönü Intersection is given in Table 6.27.

Table 6.27. Depreciation Loss in 372.98 sec. At İsmet İnönü Intersection

VEHICLE TYPE		DEPRECIATION LOST (TL)
Bus		(372.98.37.90/3600).2364=9282.60
Minibus		(372,98.20/3600).645=1336.51
Car	Private	(372.98.17.54/3600).4945=8986.26
	Taxi	(372.98.17.54/3600).2794=5077.37
Total		24 682

Daily amortization loss is

24 682.74x10=246 827.40 TL/day

Yearly amortization loss is

246 827.40x300=74 048 220 TL/year

b) After the urban GSI, the depreciation lost can be calculated as follows;

In one hour, 4165x0.28 = 1166 bus

4165x0.08 = 333 minibus

4165x0.44 = 1833 private car

4165x0.20 = 833 taxi

pass through the signalized intersection.

Depreciation lost in 280.60 sec. at the at-grade intersection part of the proposed urban grade separated intersection on İsmet İnönü Intersection is given in Table 6.28.

Table 6.28. Depreciation Loss in 280.60 sec. at the At-Grade Intersection Part of the Proposed Urban GSI. on İsmet İnönü Intersection

VEHICLE TYPE		DEPRECIATION LOSS (TL)
Bus		(280.60x37.90/3600)x1166=3444.47
Minibus		(280.60x20.00/3600)x333=519.11
Car	Private	(280.60x17.54/3600)x1833=2505.97
	Taxi	(280.60x17.54/3600)x833=1138.83
Total		7608.38

6.4.3. Fuel Loss

a) Before the urban GSI., the fuel loss can be calculated as follows;

Fuel cost was taken as equal to 2581 TL/lt for the calculations. Hourly fuel consumption of vehicles are obtained from Petrol Ofisi.

Hourly fuel consumption of vehicles is given in Table 6.29.

Table 6.29. Hourly Fuel Consumption of Vehicles

VEHICLE TYPE	HOURLY FUEL CONSUMPTION (Liter)
Bus	4
Minibus	2,5
Car	2

Yearly extra annual fuel consumption at İsmet İnönü Intersection is given in Table 6.30.

Table 6.30. Yearly Extra Fuel Consumption at İsmet İnönü Intersection

VEHICLE TYPE	FUEL CONSUMPTION IN 372.98 sec. (TL)	EXTRA ANNUAL FUEL CONSUMPTION (TL/year)
Bus	(372.98x4/3600). 2581=1070	2364x1070x300= 759 553 300
Minibus	(372.98x2.5/3600). 2581=669	645x669x300= 129 451 500
Car	(372.98x2/3600). 2581=535	7739x535x300= 1 242 109 500
Total	2274	2 131 114 300

b) After the urban GSI., the fuel loss can be calculated as follows;

Yearly extra annual fuel consumption for the at-grade intersection part of the proposed urban grade-separated intersection on İsmet İnönü Intersection is given in Table 6.31.

Table 6.31. Yearly Extra Fuel Consumption for the At-Grade Intersection Part of the Proposed Urban Grade-Separated Intersection on İsmet İnönü Intersection

VEHICLE TYPE	FUEL CONSUMPTION IN 280.60 sec (TL)	YEARLY EXTRA ANNUAL FUEL CONSUMPTION (TL/year)
Bus	(280.60x4/3600). 2581=804.70	1166x804.70x300= 281 484 060
Minibus	(280.60x2.5/3600). 2581=502.94	333x502.94x300= 50 243 706
Car	(280.60x2/3600). 2581=402.35	2666x402.35x300= 321 799 530
Total	1709.99	653 527 296

The benefits obtained from the construction of an urban grade-separated intersection in terms of delay, depreciation and fuel consumptions are summarized in Table 6.32.

The total benefit obtained from the construction of an urban GSI. to İsmet İnönü Intersection is;

$$18.82.10^9 - 6.55.10^9 = 12.27.10^9 \text{ TL/year}$$

The total delay, depreciation and fuel consumption loss which is $18.82.10^9$ TL. will decrease to $6.55.10^9$ TL. per year and $12.77.10^9$ TL. will be saved annually.

Table 6.32. The Loss Before the Proposed Urban GSI. and After the Urban GSI.

TYPE OF LOSS	Before Urban GSI. (TL/year)	After Urban GSI (TL/year)
Delay Loss	16 620 000 000	5 879 520 300
Depreciation Loss	74 048 220	22 825 140
Fuel Loss	2 131 114 300	653 527 296
Total	$18.82.10^9$	$6.55.10^9$

For a complete analyses a comparison must be made between the construction cost of the urban GSI. and the benefits obtained. Therefore a cost estimation of the urban GSI. are given here in after.

6.4.4. Cost Estimation

The planned urban GSI. is shown on Figure 6.3. The overpass has two lanes in each direction and has a 1454 ft. (442 m.) length, 46 ft. (14 m.) width; the ramp which connects Atatürk Boulevard with Eskişehir road has 578 ft. (175.6 m.) length and 11 ft. (3.34 m.) width; the last ramp, connecting Atatürk Boulevard to Esat street is 595 ft. (180,8 m.) long and 11 ft.(3.34 m.) wide.

The geometric dimensions of the overpass are assumed as follows;

Beams	= 150 cm x 200 cm
Columns	= 150 cm x 150 cm
Foundation	= 400 cm x 400 cm x 1.5 m
Slab	= 60 cm

The dimensions of the ramps are as follows;

Beams	= 100 cm x 150 cm
Columns	= 100 cm x 100 cm
Foundation	= 250 cm x 250 cm x 1.5 m
Slab	= 60 cm

The cost estimation of the urban GSI. is shown on Table 6.33.

Table 6.33. The Cost Estimation of the Proposed Urban GSI

COST ESTIMATION						
NO	Item No	TYPE OF WORK	Unit of measurement	Unit price	Amounth of material	Price (TL)
1	14.100	Exacavation near bridge area	m^3	10218	800	8 174 400
2	14.101	Excavation near bridge area	m^3	2521	5000	12 605 000
3	14.111	Excavation near foundations at any depth	m^3	15089	60000	905340000
4	14.018/K	Hand compaction of any Kind of excavated materials of filling areas	m^3	2894	1773	5131062
5	14.113	Excavation of foundations for bridges	m^3	39390	4474.5	176 250 555
6	15.047	Watering	ton	5320	60	319 200
7	15.058	Compaction	hour	38003	100	3800300
8	16.071/K	In-situ pile	m	263589	385	101481765
9	16.101/K	Unreinforced concrete for bridge foundations	m^3	72281	2701.5	195267121.5
10	16.121/K	Unreinforced concrete for the bridges except their foundations	m^3	106010	1000	106010000
11	16.132/K	Reinforced concrete for bridges	m^3	238240	809.16	192774278.4
12	16.131/K	Reinforced concrete for any kind of construction	m3	121316	1046.5	126957194
13	16.135/K	Concrete road and side walk	m^3	66351	150	9952650
14	18.191	Removal of kerbs	m	723	1600	115680

Table 6.33. Continued

15	3657	Insulation	m^2	10625	7378.38	78395245
16	3000	Cement	ton	225000	1743,92	392391212•5
17	21.053	Scaffoldings for bridges	m^3	20986	30524,34	640583925.2
18	23.001/K	Ø6~Ø~12 mm reinforcing bar workmans hip	ton	414219	606	251016714
19	23.002/K	Ø14 mm or larger size reinforcing bar workmanship	ton	283694	910	258161540
20	23.255	Grills	kg	2484	800	1987200
21	23.176/K	Balustrades	ton	2491856	1596.8	3978995661
22	3365	Concrete median kerb	m	14751	1596.8	23554396•8
23	25.016	Painting of the iran balustrades	m^2	14547	1437.12	20905784•64
24	27.501/K	2 cm thick concrete surface plastering	m^2	4701	7378•37	34685745•58
25	6030	Construction of subbase	m^3	7967	360•2	2869713,4
26	4377	Sweepping	min	14693	20	293860
27	4391	Spraying asphalt	min	29726	20	594520
28		Security equipment				50.10^6
29		Transportation				500.10^6
		TOTAL				8.079.645.844

Item numbers used for the cost estimation of the proposed urban GSI. are obtained from the 1991 unit price list for roads and bridges which is published by KGM.

6.4.5. Comparison of Investment Costs and Benefits

According to the calculation, the urban GSI. construction cost is 8 079 645 844 TL. and the yearly benefit obtained is $12.27.10^9$ TL. For a comparison, the annual maintenance and operation cost of the urban GSI. should be also taken into consideration. If we assume that the annual maintenance cost is %15 of total construction cost then;

$$8\ 079\ 645\ 884 \times 0.15 = 1\ 211\ 946\ 877 \text{ TL}$$

is assumed to be the annual maintenance and operation cost.

The annual benefit can be reduced to;

$$12.27.10^9 - 1\ 211\ 946\ 877 = 11.06\ .10^9 \text{ TL.}$$

a) Economical Analyses by the use of Net Present Value Method:

The difference between the total investment expenditures and the total benefits present value are calculated according to a determined reduction rate (i).

In economical analyses, legal interest rate (i) of %30 is taken according to law number 3095 published in the Official Gazette dated December 19 th 1984. The useful life (n) is assumed to be 20 year.

i = %30

n = 20 year

The total present value of investment expenditures is 8 079 645 844 TL.

The present value of benefits can be calculated as follows;

$$PV = 11.06x10^{9}[\frac{(1+0.30)^{20}-1}{0.30(1+0.30)^{20}}] = 36.67x10^{9}\ TL.$$

The NPV is;

$$NPV = 36.67.10^{9}\text{-}8\ 079\ 645\ 844 = 28.59.10^{9}\ TL.$$

Since the result is positive, then this project is said to be economically feasible.

b) Economical Analyses by the use of Internal Rate of Return Method

The interest rate which equalize the total investment expenditures present value to the coming years total benefits present value can be calculated as follows;

$$11.06.10^9 = 8\ 079\ 645\ 844 x p$$

$$p = \frac{11.06x10^9}{8\ 079\ 645\ 844} = 1.368$$

p = % 136.8

c) Economical Analyses by the use of Benefit Cost Ratio Method

The Benefit Cost Ratio (B/C) can be calculated as follows;

$B/C = 11{\cdot}06.10^{9}/8\ 079\ 645\ 844$

$B/C = 1.36$

Because $1{\cdot}36 > 1{\cdot}00$ this project is economically feasible.

CHAPTER VII

SUMMARY AND CONCLUSIONS

The following conclusions may be mentioned as the result of the study:

i) For all congested intersections, the capacity and level-of-service analyses should be done before applying modifications on geometry or signal cycle.

ii) If an AGI had little improvement through all geometric modifications and signalization, the possible benefits derived from the implementation of an interchange should be calculated in terms of fuel consumption, depreciation, delays and the benefits must be compared with the estimated cost of implementation.

iii) In the economical analyses for the implementation of a GSI, the interest rate should be the one which is generally used by the related gouvermental agencies in economic evaluation of comparable project.

iv) In the case study, İsmet İnönü Intersection the critical x value is determined to be 1.67 while for all other lane groups it is more than 1.00. Since the delay is 373 sec/veh. the intersection operates at level-of-service F. At the same time it is observed that intersection is oversaturated because all approaches experience relatively longer delays.

Upon the implementation of a GSI the savings; time, depreciation, and fuel are as shown in the below Table;

Type of Cost	Before Urban GSI (TL/year)	After Urban GSI (TL/year)	Savings (TL/year)
Delay Cost	16 620 000 000	5 879 520 300	10 740 479 700
Depreciation Cost	74 048 220	22 825 140	51 223 080
Fuel Cost	2 131 114 300	653 527 296	1 477 587 004
Total	$18.82.10^9$	$6.55.10^9$	$1.23.10^9$

v) Since interchanges are essential components of freeways and they are necessary at all crossroads where full access control is necessary. At the intersection of freeways with ring roads and where primary distributor roads cross each other an interchange of some type must be used.

vi) In urban areas due to high cost of land at-grade intersections are recommended. In case of replacing an inefficient AGI with GSI all measures and modifications for improving the efficiency of AGI should be considered due to high cost of improvement.

vii) The policy of KGM on "application of interchanges" was evaluated. A case study covering alternative analyses was made for the intersection of E-5 road with Ankara Ring Road which included the application of the theoretical considerations for interchanges outlined in this study. Case study consisted of three different interchange types having different design characteristics. Beside the design characteristics as length of the structure, weaving length, maximum grade, minimum curve radius, capacity, level-of-service and construction cost; the expropriation cost has the effect of changing the overall ranking of alternatives. Expropriation cost should be approximately calculated for each alternative. On the other hand it must be known by the planners that the total cost only is never adequate to find the best alternative. As shown in Table 4.2 Diamond Interchange is eliminated although it has the lowest total cost.

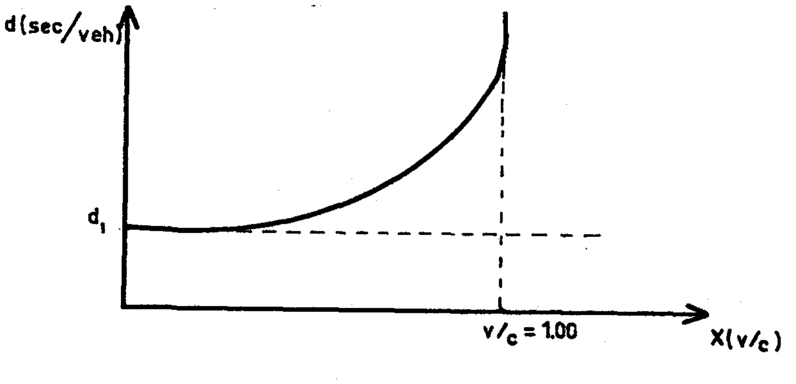
CHAPTER VIII

RECOMMENDATIONS FOR FURTHER STUDY

Suggestions for further research may be summarized as follows:

i) In the economic analyses of urban GSI a %30 interest rate was used, this is the present interest rate used in comparable studies by gouvermental agencies. It is recommended that the effect of different interest rates should be studied for a better understanding for the choice of projects.

ii) As shown in Figure below it is a good indication that when X is greater than 1.00 an urban GSI may be economical however for X values less than 1.00 other measures must be considered before deciding upon as urban GSI.



iii) Cost of accidents and air pollution may included in the evaluation of alternatives.

iv) During the planning of an interchange, the proximity of other interchanges is very important. Especially in urban areas, it is better to prepare a comprehensive plan for the evaluation of interrelated intersections and interchanges.

v) The effect of double-lane ramps on interchange cost and efficiency may be evaluated.

vi) For capacity and level-of-service analyses of a congested at-grade intersection peak hours and related traffic volumes must be considered.

vii) Driver behaviour may be included for determining factors for turning movements.

viii) The effect of traffic signs and marking on capacity and efficiency of interchanges may be evaluated.

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