ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL

GEOMETRIC FACTORS IN ROADWAY DRAINAGE

Ph.D. THESIS

Sevgi CAVDAR

Department of Civil Engineering

Hydraulics and Water Resources Engineering Programme

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İSTANBUL TEKNİK ÜNİVERSİTESİ ★ LİSANSÜSTÜ EĞİTİM ENSTİTÜSÜ

YOL DRENAJINDA GEOMETRİK FAKTÖRLER

DOKTORA TEZİ

Sevgi CAVDAR
(501132502)

İnşaat Mühendisliği Anabilim Dalı

Hidrolik ve Su Kaynakları Mühendisliği Programı

Tez Danışmanı: Dr. Ögr. Üyesi Ali UYUMAZ

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Sevgi Cavdar, a Ph.D. student of ITU Graduate School student ID 501132502 successfully defended the dissertation entitled "GEOMETRIC FACTORS IN ROADWAY DRAINAGE", which she prepared after fulfilling the requirements specified in the associated legislations, before the jury whose signatures are below.

Thesis Advisor :	**Asst. Prof. Dr. Ali UYUMAZ** İstanbul Technical University	.....
Jury Members :	**Prof. Dr. Şevket ÇOKGÖR** Istanbul Technical University	.....
	Prof. Dr. Zekâi ŞEN Istanbul Medipol University	.....
	Assoc. Prof. Dr. İsmail DABANLI Istanbul Technical University	.....
	Asst. Prof. Dr. Mustafa Nuri BALOV Istanbul Gelişim University	.....

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To my selfless savior, Dr. Ali Uyumaz Hocam.

FOREWORD

Data suggest that inclement weather increases the risk of roadway accidents. Making drainage a priority in designing roadways prevents a significant portion of these accidents, lowers the degree of nuisances to overall road users, and increases roadways' longevity. With the motivation to contribute to roadway drainage design, I compiled this work with the help of a large tribe that I must thank for, for without their support, this could not have happened.

I would first and foremost like to thank my supervisor, Dr. Ali UYUMAZ, from whom I benefitted greatly all along the way. He has always supported me during the ups and downs of this very long journey, be it academic or personal. His humor was also essential for completing this work while he encouraged me to finish. I would also like to thank my committee members. Dr. Şevket ÇOKGÖR guided me in each step and was always there for help. Dr. İsmail DABANLI pushed me to ensure I was progressing. I want to also recognize late professor Dr. Ahmet DOĞAN from Yildiz Tech, who served on my committee for a couple of semesters and provided great feedback on my work; may he be enveloped in peace. And a thank you to all ITU and Hydraulics Lab family.

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Oh, and this section cannot be complete without the mention of PHD Comics and my plants.

All praise be to Him.

June 2022

Sevgi CAVDAR
(RA)

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ABBREVIATIONS

AASHTO	**:** The American Association of State Highway and Transportation Officials
COI	**:** Curb Opening Inlet
C&S	**:** Cristina and Sansalone (2003) study
HEC-22	**:** Hydraulic Engineering Circular 2013
ICE	**:** Institute of Civil Engineers
KWE	**:** Kinematic wave equation
US & SI	**:** United States Customary Units & International System of Units
The US	**:** United States of America
TUIK	**:** Turkish Statistical Institute
TOC	**:** Time of concentration
USACE	**:** United States Corps of Engineers

SYMBOLS

a : Inlet depression (acceleration in Li (1954))[1]

a_g : Gutter depression, suggestion

a_i : Inlet depression, suggestion

a_{local} : Inlet depression that is only at the vicinity of the inlet

A : Area (empirical coefficient in Uyumaz model in chapter 4)

A_0 : Cross sectional area for an incoming flow of Q_0 in Li (1954)

α : Angle roadway centerline makes with the resultant of the cross and longitudinal slopes

b : Roadway width or the distance perpendicular to the curb from the end of gutter or shoulder to the highest point contributing to the pavement flow

B : Empirical coefficients in Uyumaz study

C : Runoff coefficient (In chapter 4: empirical coefficient in Uyumaz model; a function of several numbers in Li (1954) model)

C_F : Correction factor

D : Empirical coefficients in Uyumaz study

E : Specific energy of the approach flow

E_0 : Flow in tbe depressed part of the gutter divided by the total flow upstream inlet opening (HEC-22)

F : Ratio of inertial forces to gravity forces: $F = V/\sqrt{gy}$, i.e., Froude number (In Izzard 1977 model, it is the Froude number at distance W from the curb)

g : Gravitational acceleration

I : Rainfall intensity

k : $K_M S^{1/2} / n$

K : $3/8 K_M K_R$ (In chapter 4: A function of inlet length in Soares (1991) model; empirical coefficient in Li (1954))

K_M : Unit conversion factor in Manning's equation, 1 for SI and 1.486 for US

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[1] In this section parantheses are used to document the description for a symbol that is employed for more than one parameter.

K_R : Conversion factor in the Rational Method, $36x10^5$ for SI and 43200 for US

K_T : Coefficient equal to 6.92 or 0.933, units in SI and US

K_u : Coefficient equal to 0.817 for SI and 0.6 for US

L : Flow path length (Curb opening inlet length in Chapter 4)

L_0 : Curb opening length for a fully captured flow when $a = 0$

L_1 : $2.29W^{-1/6}S_x^{0.3}FT$ (see Izzard 1977 model)

L_2 : $3.01W^{-1/6}S_x^{1/2}FT$ (see Izzard 1977 model)

L_3 : $1.65FT$ (see Izzard 1977 model)

L_a : Curb opening length for a fully captured flow with the inlet depressed a (Izzard 1950)

L_i : Installed inlet length in Izzard 1977

L_T : Full capture lenght calculated using equation (4.14), using equivalent slope concept

L_{UT} : Upstream transition lenght for a locally depressed inlet

m : Constant (=5/3) (In chapter 4: discharge coefficient in Uyumaz model; $\left(L_{UT} / y_{OF}\right)^{0.22}$ in Muhammad 2018)

n : Manning's roughness coefficient

q : Unit discharge

Q : Discharge

Q_0 : Theoretical full capture capacity of a curb opening inlet

Q_a : Full capture capacity for a depressed inlet (or the approach flow)

Q_A : Flow amount contributed from area, A (number after A specifies which area contributes)

Q_G : Total precipitation that falls within road center line to curbline

Q_{HEC} : Flow calculated using HEC model (Muhammad 2018)

Q_i : Intercepted flow in Izzard 1977 model

Q_{OF} : Flow existing inlet may capture at full capacity

Q_R : Amount of over-lane flow (area free of spread limitation)

Q_s : Channel flow amount that is comprised within the surface of the lanes for a depressed roadway (see Figure 4.9)

Q_w : Channel flow amount that is comprised within the gutter width for a roadway with depressed cross slope S_w (see Figure 4.9)

Q_W : Amount of rainfall that falls on the roadway limited with W from the curb (over-gutter precipitation)

q_y : Unit discharge flow in the longitudinal direction

S : Roadway surface slope

S_e : Equivalent [cross] slope of a roadway

S_L : Roadway longitudinal slope

S_{UT} : Longitudinal slope for he upstream transition

S_w : Cross slope of a depressed gutter (see Figure 4.9)

S_x : Roadway cross slope

S_{x1} : Cross slope of the roadway (equivalent to S_x, see also S_{x2}

S_{x2} : Cross slope of the gutter that is continuously depressed (as opposed to locally depressed or the roadway cross slope

t : Time

T : Flow spread perpendicular to curb right upstream of an inlet

T_{max} : Maximum allowable flow spread

T_{OF} : Width of flow corresponding to Q_{OF}

T_t : Travel time

T_{ti} : Over-lane flow travel time

v : Velocity (see Figure 4.10)

V : Velocity

W : *length* of gutter or shoulder (Figure 3.1) (while they are the same, it is referred as gutter width in Chapter 4)

x : Maximum possible inlet spacing (Distance from the upstream end of the inlet opening)

x_1 : See Figure 3.2

x_{first} : Maximum allowable inlet spacing for the first one in a configuration

x_{last} : Maximum allowable inlet spacing for the last one in a configuration

y : Distance in the longitudinal direction (Flow depth at the curb in Chapter 4)

y_1 : Flow depth that *corresponds* to inlet length L (see Figure 4.10)

y_{OF} : Normal depth in the approach section for flow Q_{OF}

z : Flow depth (over-lane or channel flow)

z_{max} : Maximum allowable channel flow depth

z_r : Over-lane flow depth

$z_{R\&R}$: Flow depth calculated using Ross and Russam (1968) model

Δ : Difference

θ_0 : Angle roadway surface makes with the vertical (see Figure 4.10)

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GEOMETRIC FACTORS IN ROADWAY DRAINAGE

SUMMARY

Optimal drainage of roadways is crucial for safety, comfort, and economics. Lack of drainage results in reduced friction, lowered visibility, and ponded water creating danger and (at the least) nuisances to overall road users and the owners of nearby properties. In the long run, deterioration in roads follows as the standing water gets in in warmer climates and snowmelt expands in colder with the following frost damages; these decline driving safety and comfort for the second time in addition to the added cost of reconstruction and rehabilitation of the roadway. Thus, considerations regarding roadway drainage are an indispensable part of roadway design for both the overall safety of users and properties and the longevity of the roadway itself.

Roadway flow takes two forms: over-lane (i.e., overland) flow and gutter flow (i.e., channel flow) adjacent to the curb. Thus, roadway drainage encompasses both over-lane flow thickness optimization and controlling gutter flow. This dissertation investigates the most efficient and effective slope orientation for roadways of various widths, environmental conditions, and functionalities to establish necessary draining. That encompasses producing a physically based model for over-lane flow depth to investigate hydroplaning risks based on slope orientation; to examine the correctness of the common perception that inlet capacity is what controls inlet spacing. It also aims to frame the existing flush curb opening inlet studies within their scope and determines the missing parts to the most commonly used guidelines.

While there are studies concerned with the sideway gutter flow and its spread, the flow contributing to hydroplaning directly from the roadway surface is less studied, and the geometric limits to antihydroplaning values are not well established. This work employs a kinematic wave equation (KWE) and provides a model to calculate water depth for the over-lane flow. The flow depths obtained via the new model are compared with antihydroplaning flow depths to find optimal cross slopes for various combinations for given longitudinal slopes. Establishing hydroplaing-free roads is the first step in establishing safe roads. A large amount of literature focuses on limiting the flow spread from the curb for a range of cross slope values; this leads to various flow depths due to of altered cross section, but the standards seem to fall shy in controlling the depth of flow. HEC-22, the urban drainage design manual of the US Department of Transportation, endorses limiting the flow depths to curb height and fixes the criterion for the inlet spacing to maximum allowable flow spreads. HEC-22 also recommends inlet spacing to be between 90-150 m. The common perception is that inlet capacity controls inlet spacing, but the impact of slope orientation under given criteria (such as a particular depth limitation) does not get much attention. This work, with no regard to inlet capacity but using the antihydroplaning configurations from the earlier part, analyzes maximum allowable inlet spacings and compares them to HEC-22's inlet spacing recommendation of 90-150 m to show any orientation that needs consideration beyond inlet capacity in

designing inlet spacing. Maximum allowable inlet spacings are obtained for fixed maximums to flow depth and spread for the channel flow, and fixed maximums to over-lane water film thickness. The impact of slope orientation on inlet spacing was then analyzed. Having the results for varying slope orientations is crucial especially in changing terrains where perhaps each inlet spacing needs updating with changing slope, and spacing is critical for optimal drainage. The sizing of inlets under various slope orientations is important as the capacity varies with slope orientation. Curb opening inlets, the vertical openings at curbs, were investigated in this work also, for they noninterfere with the roadway and nonclog being a secure and effective candidate to most areas. A detailed analysis of the foundational models of curb opening inlets is presented for various geometric designs of flush orientations located on-grade. These geometries/types include simple vertical openings on curbs, openings with warped pavement in the close vicinity to increase capture capacity, and openings with not only the vicinity of the inlet depressed but also the approach gutter to increase drainage capacities further. The existing models were studied in detail and the misattributes to fundamental models were brought to the light.

This work aimed to decrease the aftermath of inclement weather for the concerned partners from the bicyclist to the maintenance person via providing analyses to improve roadway drainage with the help of correct roadway slope orientation. This work contributes to (self) drainage by providing:

1) an easy to apply model for computing the over-lane flow depths so the geometries that are safe in terms of hydroplaning could be determined by any designer.
2) a detailed analysis of geometric configurations and putting forward not only which ones are *safe* in terms of hydroplaning but also which require attention in determining inlet spacing beyond the criteria of inlet capacity and maintenance purposes.
3) the details on curb opening inlet literature and investigating some inconsistencies regarding certain designs in what is probably the most commonly used manual (i.e. HEC-22).

YOL DRENAJINDA GEOMETRİK FAKTÖRLER

ÖZET

Yağış esnasında ve hemen sonrasında karayolları birer kanal görevi görürler. Dolayısıyla, yolların yüzeysel drenajı, yolun kendisi, kullanım sağlayanlar, ve yol çevresindekiler için büyük önem taşımaktadır. Drenajı düzgün tasarlanmayan otoyollar farklı yönlerden tehlike oluştururlar. Motorlu araç ve kullanıcıları açısından su sıçramaları görüş mesafesini sıfıra kadar indirebilmekteyken, biriken sular manevra hakimiyetini ortadan kaldırabilmekte, trafiği yavaşlatmakta, ve belli hız değerlerinde ilerleyen taşıtlar için yol yüzeyindeki su filmi kalınlığının eşik değerin üzerine çıkması lastiklerin yerle kontağını keserek özellikle yol tasarımından kaynaklanan hidroplan (su yatagi) riski oluşturmaktadır. Diğer taraftan saglıksız drenajın yaya, bisikletli, ve engelliler için oluşturacagı olumsuzluklar aşikardır, ki buna kontolsüz motorlu araç trafiği de eklendiğinde büyük riskler oluşmaktadır. Özellikle bordürlerin yer aldığı yollar yüksek debiler biriktirebileceğinden, akım kontrolü sağlanamadığı takdirde çevreye de riskler oluşturur. Bunların yanı sıra, özellikle düzlemsel yüzeyini koruyamayan yollarda ve su alma açıklıklarının doğru planlanmadığı bölgelerde, biriken sular yol gövdesine nüfuz ederek, alttemele zarar verebilmekte, bu da kaplamaya zarar vereceğinden hem sürüş konfor ve güvenliğini etkilemekte, hem yol onarım masraflarını arttırmakta, ve hem de yolun ömrünü kısaltmaktadır. Donma noktasının altına düşen sıcaklıklarda, yagmur suyu veya günün sıcak saatlerinde eriyen kar suları tahliye edilmediği takdirde donup genleşeceginden, yola tahribati kronik hale getirmektedir. Bu tür yollarda ilerlemekte olan trafikte sürücüler oluşan deformasyonlardan uzaklaşmak için şerit ihlallerine yol açmakta, bu da trafikte, özünde yine drenaj problemlerinin yol açtığı, aksamalar ve tehlikeler oluşturmaktadır.

İstatistikler olumsuz hava koşullarının kaza riskini arttıdığını ortaya koymaktadır, fakat bu kazaların birçoğu kullanıcı kaynaklı olarak sınıflandırılmaktadır ve istatistiklere göre yol kaynaklı hatalar ülkemizde oldukça düşüktür. Fakat, hem bireysel tecrübelerimiz var olan yolların olumsuz hava koşullarına çok da uygun olmayabildiğini göstermekteyken, hem de bu durum son yıllarda özellikle batı ülkelerinde pekçe kabul gören güvenli sistem tetkiki (safe system approach), anlayışına terstir. Bu sistem kapsamında kullanıcı hataları ve sonrasındaki etkiler minimize edilmeye çabalanir. Birleşmiş Milletler, mesela, 2030 yılına kadar yol kazalarındaki ölüm ve yaralanmaların bu kapsamda yarıya indirilmesini hedeflemektedir. Güvenli sistem tetkiki, hataların insan kaynaklı sınıflandırılmasının mazeret kabul edilemeyecegini benimser. Bu doğrultuda, olumsuz hava şartlarının sebep olduğu kazaların minimize edilebilmesi amacıyla, yol tasarım ve drenajının yeniden gözden geçirilmesinde fayda vardır.

Yol tasarımı, genel kabül gören bazi durumlar dışında, ülkeden ülkeye ve hatta şehirler arasında farklılıklar oluşturabilir. Bu bir taraftan anlaşılabilirdir çünkü topografya, yağış rejimi, ve bölgenin gelişmişliği hem eşik değerlerini ve hem de beklentileri şekillendirmektedir. Diğer taraftan, özellikle araştırma bazlı somut

gerçekler vardır ve belirli düzeyde bir standard oluşmuş durumdadır. Drenaj açısından incelendiğinde, yolun kendini drene edebilecek kapasiteye sahip olması büyük önem tasımaktadır; bunun sağlanabilmesi için, yol yüzeyinin düzlemsele yakın olması su akışının bloke edilmesini ortadan kaldıracağından büyük avantajlar taşımaktadır. O halde düzlemsel olarak ele alabildiğimiz yol yüzeyinin açısal olarak konumlandırılışına gerekli önem verilirse, yanal eğim sayesinde yüzeye gelen yağış bordüre yönlendirileceğinden ve yolun boyuna eğimi sayesinde akışına devam edeceğinden, yağışın araç trafiğine etkisi belli ölçülerde azaltılmış olur. Bu eğimlerin değerlerinin belirlenmesinde sürücü konforu ve farklı kaynaklı kazaların minimize edilmesi göz önünde bulundurulmasının yanı sıra, tasarım yağış şiddetine bağlı olarak oluşturacakları film kalınlıklarının su yatağı oluşturmayacak şekilde düzenlenmiş olması taşıt güvenliği açısından büyük önem taşımaktadır. Diğer taraftan, eğim değeri su girişlerinin kapasitelerini de doğrudan etkilemektedir. Bu bakımdan, enine ve boyuna eğim değerlerinin etkilerinin doğru tespiti önemlidir ve bu çalışmada tekdüze enine eğim ve sıfır dışı boyuna eğim değerlerinin etkileri incelenmiştir. Bu kapsamda bu çalişmada öncelikle su yatağı yani hidroplana eğimin etkileri farklı yol genişlikleri ve yağış şiddetleri için araştırılmıştır. Bunu yaparken yüzey suyu için kinematik dalga yaklaşımından istifade edilerek bir derinlik hesaplama modeli oluşturulmuş, ve farklı eğim değerleri için bu model kullanılarak elde edilen su derinliklerinin deneysel olarak farklı araştırmacılarca tespit edilmiş olan hidroplan tehdidi oluşturan kalınlıklarla kıyaslanmıştır. Normal şartlarda, topografyanın boyuna eğime etkisi kaçınılmazdır; artan boyuna eğimler ise, eğer enine eğimlerle dengelenmezlerse, su yolu uzunluğunu arttırmakta ve film kalınlıklarının tehlikeler oluşturabilecek düzeylere yükselmesine yol açmaktadır. Bu açıdan farklı değerler için sonuçların incelenmesi önem taşımaktadır. Güvenlik kısıtlamaları göz önünde bulundurulunca enine eğimler maximum %6'ya kadar incelenmiş ve eğim değeri arttıkça eğimin su derinliğine etkisinin azaldığı gözlemlenmiştir. Örneğin enine eğim degerinin %1'den %2'ye çıkarılması ile %5'den %6'ya çıkarılmasının (aradaki fark eşit olsa da) su yüksekliğine etkisi aynı değildir; dolayısıyla bu durum yol konforu ve güvenliği de dikkate alınarak bir değerlendirme yapılmalıdır. Çalışmanın sonuçları göstermiştir ki, enine eğim optimizasyonu olumsuz hava koşullarının yol açabileceği su yatagi problemlerini önemli ölçüde bertaraf edebilmektedir. İklim şartları da göz önünde bulundurularak—donma olaylarının yaşandığı bölgelerde kış sürüş güvenliği açısından enine eğim artımında dikkatli olunmalıdır—enine eğimler şekillendirilirken, yol yüzeyi su miktarlari değerlendirmede aktif bir parametre olarak değerlendirilmelidir.

Çalışmanın ilerleyen kısmında enine ve boyuna eğimin geleneksel olarak kabul edilegörmüş yagmur suyu yayılma genişlikleri için ve aynı zamanda yayaların yağmur suyundan etkilenmelerini minimize edecek su yükseklikleri icin—ki bunlar birbirlerinden oldukça farklıdırlar—su alma açıklıklarının ara mesafelerine etkileri arastırılmıştır. Bu araştırma yapılırken ilk kısımda elde edilen model kullanılarak hidroplan riski içeren konfigürasyonlar elimine edilmiştir. Kalan modeller rasyonel method ve Manning denkleminin yol geometrisine uyarlanmiş hali ile kullanılarak (Izzard modifiye) giriş açıklıkları arasında olabilecek maksimum mesafeler elde edilmiştir. Bunlar bakım gibi nedenler göz önüne alınarak tavsiye edilen açıklık genişlikleriyle kıyaslanarak, boyuna ve enine eğimin etkileri ortaya konulmuştur. Su derinliği olarak geleneksel uygulama bordür derinliğiyle kıyas yapmaktır, ancak bu durum özellikle yayalar, bisikletliler, ve engelliler için istenmeyen senaryolar oluşturmaktadır ve bu kullanıcıların yolu kullandıkları durumlarda drenaj tasarımının

gerekli önlemler doğrultusunda yapılması önem arzetmektedir. Bu durum kotu hava şartlarında acil durumların göz önünde bulundurulduğu haller için de önem arzeder. Boyuna eğimlerin %10'a kadar ve yanal eğimlerin %6'ya kadar olduğu durumlar için analizler yapılmış olup her ne kadar belli bir bölgeye bağlı çalışılmasa da yağmur sularının toplanma zamanı 5 dakika varsayılır ve tekerrürü 10 yıl alınırsa, 250 mm hr^{-1}'a kadarki şiddetlerde hidroplan tehlikesinin 15 m'ye kadar genişliklere sahip yollarda giderilebildiği görülmüştür. Çalışma su alma acıklıklarının—model her ne kadar enine eğimle ters orantılı olarak değiştiğini gösteriyor olsa da (S_x^{-1}), su derinliğinin de enine eğime bağlı olmasından ötürü—enine eğimle doğru orantılı olarak değiştiğini ($S_x^{5/3}$ ile) ortaya koymuştur. Bu da çoğunlukla enine eğimin artmasının açıklık mesafesini arttırdığı anlamına gelmektedir. Bu durum enine eğim değerinin tolere edilebilir maksimum su derinliğinin maksimum su yayılmasına oranına eşit veya küçük olduğu durumlar için geçerlidir. Diger bir ifade ile su derinliğinin maksimum derinlik değerlerine ulaşmadığı durumlar için. Eğer oran enine eğim değerinden küçük olursa, mesafe S_x^{-1} ile değişmektedir; bunun sebebi, enine eğimden bağımsız olarak modelde maksimum su derinliğinin kullanılıyor olmasından kaynaklanmaktadır. Ortalama olarak yayılma genişliğinin 1 m'nin altında olacağını varsayar ve bir bordür yüksekliğinin de 0.1 m olduğunu varsayarsak, elde edilen oran 0.1 olacağından ve enine eğim değeri muhtemel %10'un altında değer alacagından, derinlik kontrolü gerektirmeyen böyle bir durumda tolere edilebilir mesafelerin $S_x^{5/3}$ ile değişeceği aşikardır. Fakat eğer tolere edilebilir derinlikler 0.03 m değerlerine düşerse, o vakit değişim S_x^{-1} ile gercekleşir. Boyuna eğimin artışı, diğer taraftan, genel itibariyle maksimum su alma ağzı açıklığı mesafelerini hidroplana yol açmadığı sürece arttırmaktadır. Bu doğrultuda, özellikle yol kenarı su yüksekliğinin önem taşıdığı durumlarda, su alma ağızlarının konumları planlanirken, yol geometrisinin (diger bir ifade ile maksimum su derinliğinin de) ayrıca parametreler arasında incelenmesi gerekmekte ve giriş konumları buna göre karar verilmelidir. Diğer bir ifade ile, eğer yayılma genişliği kadar su yüksekliği de önem taşıyorsa, sualma ağızlarının (veya kentdışı yollarda dolgu şevi erozyonunu engellemek veya farklı amaçlar için kullanılan bordürlerin olması durumunda su oluklarının) ara mesafelerinin tayininde, enine eğimin etkisinin seçilen değere göre mesafeyi belli bir limite kadar arttırıcı etkisi olurken, bu limitin aşımından sonra azaltıcı etkiye sahip olduğunu ortaya koymuştur.

Son olarak bu çalışmada yağış sonrası suların uzaklaştırılması amacıyla inşa edilen su giriş yapılarından bördürde bırakılan açıklıklara ait modeller incelenmiştir. Bordür taşında bırakılan düsey açıklıklar ülkemiz Türkiye'de karşılaşılan bir tür olmamakla birlikte, yol yüzeyinde yer almaması açısından büyük avantajlar taşır. Izgaralar motorlu/motorsuz araç trafiğini etkilemekte olup riskler oluştururken, bordürdeki açıklıklar için böyle bir durum söz konusu değildir. Ayrıca düşey açıklık oldukları için, bunların tıkanması da pek sözkonusu değildir, dolayısıyla çok az bakım gerektirir; gerekli bakım ve onarım çalışmalarında ise trafikte yaşanabilecek aksamalar konumları gereği minimize edilmiş olur. Bu tür giriş yerlerinin kapasitelerinin veya gerekli açıklık miktarlarının ölçümlendirilebilmesi için birçok çalışma yapılmış olmakla birlike, tamamen analitik olan çalışmalar nadirdir. Bu konuya damga vuran çalışma ise 1950 yılında yapılmıştır. Sonra gelen arastırmacılarca Carl F. Izzard tarafindan yapılan bu çalışma analitik çözümlü model sundugu ve deneysel çalışmalar kullanılarak empirik modellerın de ortaya konulduğu gerekçesiyle pekçe övülmüştür. ABD Federal Karayolu İdaresi'nin son Kentsel

Drenaj Dizayn Manueli olan HEC-22'nin modellerinin belli ölçüde bu çalışmadan türetildiği 2018 yılında detaylarıyla günışığına çıkarılmış, öncelikle uyumlu olarak başlayan türetilmelerin sonrasında yön değişip tüm modelleri depresyonsuz duruma indirgeme amacıyla eşdeğer eğim ilişkilerinin ortaya konularak Izzard modellerinden sapıldığı etraflıca detaylandırılmıştır. Sıkça kullanılan bir manuele önderlik etmiş olması, bu çalışmanın önemini vurgulamak açısından önemlidir. Okumakta olduğunuz çalışma göstermiştir ki, esasında Izzard modelleri bazi açılardan yanıltıcıdır ve eşdeğer eğim kavramı Izzard tarafından aleni olarak belirtilmemiş olsa da uygulanmıştır; bu bağlamda HEC-22'nin Izzard'dan sapmasından ziyade, ayak izlerini takip ettiğini söylemek mümkündür. Izzard'in modeller arasında geometrik benzerlik hipotezi ileri sürüp, tutarlı sonuçlar elde ettikten sonra, hiç bir açıklama yapmaksızın bu modelleri ileri sürdüğü bu çalışmada ortaya konulmuş ve sonuçlar bagımsız deneysel data ile test edilmiştir. Ne var ki, bu eşdeğer eğimler açıklık önünde lokal eğim olan veya bordür boyunca belli bir genişlikte sürekli bir yükseltilmiş eğim olan durumlar için tespit edilmiş olsa da, ikisinin birlikte bulundugu durumlar için tespit edilmemiştir. Bu geometrilerin uygulamada sıkça kullanılmasına rağmen manuelde yer almamiş olması, ve belirsiz bir şekilde de olsa, tek tipmiş gibi muamele edilebilecekmiş gibi lanse edilmesi şaşırtıcıdır. Oysa ki, bu konuda literatürde yapılmış çalışmalar –ki üniversitemiz İstanbul Teknik Üniversitesinde bu tez çalışmasını yöneten Dr. Ali Uyumaz tarafından yapılmış çalışmalar da–mevcuttur ve bunlar pekçe kullanılan bu manuele uyarlanmalıdır. HEC-22, bordürde bırakılan açıklıkların ülkemizde uygulanması durumunda kullanılabilecek bir rehber olarak görülebilir ve özellikle bu bakımdan da eksikliklerinin tespit edilmesi, önem taşımaktadır. Bordürde bırakılan açıklıkların uygulanması halinde, önemli varsayılan bir kısım diğer çalışmalardan da bahsedilmiştir.

Sonuç itibariyle, sağlıklı yol dreni için, farklı yönleriyle geometrik unsurların etkileri araştırılmiş, su alma ağzı açısından bordürde bırakılan açıklıklar üzerine yapılan çalışmalar derinlemesine incelenmiştir. Özellikle son yıllarda artan yağışlar ile, daha tutarlı yol dizaynı öngörülebildiği gibi kaçınılmazdır ve tüm yönleriyle ele alınmalıdır, ve optimum eğim değerleri bu noktada aktif rol oynamaktadır.

1. INTRODUCTION

Optimal drainage of roadways is crucial for safety, comfort, as well as economics. Lack of drainage results in reduced friction, lowered visibility, and ponded water which create danger and (at the least) nuisances to overall road users and to the owners of nearby properties. In the long run, deterioration in roads follows as the standing water gets in in warmer climates and snowmelt expands in colder with the following frost damages; these decline driving safety and comfort for a second time in addition to the added cost of reconstruction and rehabilitation of the roadway. Thus, considerations regarding roadway drainage are an indispensable part of roadway design for both the overall safety of users and properties and the longevity of the roadway itself.

1.1 Motivation and the Purpose of Dissertation

With 60% of world's vehicles, 93% of fatalities are from low- and middle-income countries. Turkish Statistical Institute (TUIK) reports 187,963 accidents with fatalities and/or injuries in Turkey in 2021, and of these, TUIK reports, only 0.4% is caused by the roadway related factors and 97.7% is user related (TUIK, 2022). While this seemingly depicts existence of safe roadways, the safe system approach, embraced by most transportation communities, aims to address the risks and accommodate human mistakes for the safety of all users from various angles. With the goal to design safe roads, United Nations target halving fatalities and injuries by 2030 (WHO, 2022; NRSS, 2022). This means, attributing the mistakes to human error is unacceptable, and the entire system should be designed to work in such a way that the aftermath of user mistakes are minimized.

The US Federal Highway Administration mentions that 21% of the accidents are inclement weather related and 70% of those are due to wet pavement (FHWA, 2020). Despite that, the measures taken are limited.

Figure 1.1 shows a region from a city center (2022, Turkey), and

Figure **1.2** shows the situation right adjacent to a bridge after the storm ceases (2022, Turkey); these photographs depict lack of proper engineering of the road infrastructures. In a holistic approach, these cannot be tolerated, and adequate drainage must be established, for which government organizations are responsible; additionally, especially highly populated areas must always have fast access for emergency situations regardless of weather. Additionally, vicinity of critical spots such as hospitals must have different roadway design criteria, which necessitates investigating different scenarios. Roadway drainage is a function of slope orientation, roughness, inlet spacing, and inlet capacity (Brown et al. 2009—also referred to as HEC-22). Beyond the gradients, roads are designed to have a transverse slope, so the flow could be directed to the side of the pavement and controlled. Uncontrolled roads have the flow run on the road for long distances increasing nuisances and hazards. Existence of cross slope not only shortens flow path and narrows flow spreads from the side, but also increases the capture capacity of inlets as a consequence of it in removing flow. There are three important factors that need interfering for proper drainage: 1) limiting over-lane flow depths to protect needed skid resistance as the lack of it leads to hydroplaning (and accidents); 2) diverting the flow to the side of the road properly; and 3) keeping it at a reasonable offset and depth to prevent any encroachments and unwanted accumulation. Of the three, last requires proper spacing and sizing of inlets to direct the flow into underground facilities. Second is related to sloping the roadway properly to prevent any ponding water and collect the flow at the edge of the pavement, which also means minimizing deformation of roadway overtime. First one is mostly about flow path length to keep the water film thickness under control.

Establishing these factors (without sacrificing riding quality much) requires optimization within various factors. Road designer may control a few parameters such as the speed limit, texture depth, or the geometric configuration to improve roadway drainage. Of these, speed limit should never be the starting point and only be considered after all alternatives are exhausted; texture decay over time is a chronic issue; and thus, the optimal configuration of roadway slopes is critical. Local and federal governments provide guidelines to set standards, and the priorities differ with differing environmental factors, cultural practices, and evolving research. Spain, for example, considers 25-year return period in designing roads, USA uses 10-year, and

Dudman (2014) takes 1-year return period in Australia holding the opinion that drivers lower their speed as a consequence of reduced visibility—which could never be acceptable for the autobahn in Germany. Thus, it is only logical to escalate the research and consider practices beyond the traditional to cope with climate change, increased urbanization, and the continuously recurring existing problems.

Figure 1.1 : Roadway in city center during a rainstorm, Sivas, Turkey (2022) CNNTurk (2022).

Figure 1.2 : Situation adjacent to a bridge after a stormwater ceases (Sivas, Turkey, 2022). Haberturk (2022).

The purpose of this dissertation is to investigate the most efficient and effective slope orientation for roadways of various widths, environmental conditions, and functionalities to establish necessary draining. That encompasses producing a physically based model for over-lane flow depth to investigate hydroplaning risks based on slope orientation; to investigate the correctness of the common perception that inlet capacity is what controls inlet spacing. It also aims to frame the existing flush curb opening inlet studies within their scope. Thus, the objectives are to provide:

1) an easy to apply model for computing the over-lane flow depths so the geometries that are safe in terms of hydroplaning could be determined by any designer easily
2) a detailed analysis of geometric configurations that are safe in terms of hydroplaning
3) the geometric configurations that requires consideration in determinin inlet spacing for accurate spacing analysis (in contrast to common practice of considering only spread limits)
4) a detailed investigation of curb opening inlet literature and alonog with inconsistencies regarding certain designs in what is probably the most commonly used manual (i.e. HEC-22).

1.2 Theoretical Framework and Dissertation Outline

Roadway drainage encompasses both over-lane flow thickness optimization and controlling channel flow at the road side adjacent to the curb. Flood routing could be done using shallow water equations and some work has been done to cover over-lane waterfilm thicknesses numerically (Wolff, 2013; Ressel et al., 2019) which are phenomenal for the existing roads with warped surfaces; however, these require a wide range of topographic data as well as programming and are not practical for use. Simpler models using kinematic wave equation proves to be effective for overland flow analysis and provide sufficient accuracy. Ross and Russam (1968) and Gallaway et al. (1979) investigated water film thicknesses experimentally with different roadway geometries; however, their solutions are limited to the tested conditions. Cristina and Sansalone (2003) developed a rainfall-runoff kinematic

wave model for highways, but the solution remains in differential form, not practical for use, and they were only concerned with the concentration time of the flow. While there are studies concerned with the sideway gutter flow and its spread, the flow contributing to hydroplaning directly from the roadway surface is less studied, and the geometric limits to antihydroplaning values are not well established. The second chapter addresses that: it employs a kinematic wave equation (KWE) and provides a model to calculate water depth for the over-lane flow. The flow depths obtained via the new model are compared with antihydroplaning flow depths to find optimal cross slopes for various combinations.

Antihydroplaning designs are essential for road safety; however, it is only the first step in establishing safe roads. There is a large literature that focuses on limiting the flow spread from the curb (AASHTO, 2018; HEC-22) for a range of cross slope values; this leads to various flow depths as a result of altered cross section, but the standards seem to fall shy in controlling the depth of flow. HEC-22, the urban drainage design manual of the US Department of Transportation, endorses limiting the flow depths to curb height and fixes the criterion for the inlet spacing to maximum allowable flow spreads. Wong and Moh (1997) do spacing analysis (dismissing hydroplaning) also using fixed spread for the inlets used in Singapore. Gomez et al. (2002), however, limit the maximum flow depth at the curb to 3 cm, stating that it is the "sole thickness of a normal shoe." Wong (1994) proposes a kinematic wave solution to calculating time of concentration for the gutter flow and using that he obtains the rainfall intensity to find the amount of flow due to rainfall and from there determines inlet spacing for a commonly used Singapore inlet. Wong and Moh (1997), using the results of Wong (1994), analyze the effect of allowable flow spreads on the inlet spacing; they find an exponential relationship in that an increase in the spread results in an exponential increase in the inlet spacing. The concentration time models were then tested against experimental data in Wong (2001) for the channel flow. HEC-22 promotes using Rational Method for roads. HEC-22 also recommends inlet spacing to be between 90-150 m. The common perception is that inlet capacity controls inlet spacing (Wong 1994, HEC-22), but the impact of slope orientation under given criteria (such as a certain depth limitation) does not get much attention. chapter 3, with no regard to inlet capacity but using the antihydroplaning configurations from the earlier part, analyzes maximum allowable

inlet spacings and compares them to HEC-22's inlet spacing recommendation of 90-150 m to show any orientation that needs consideration beyond inlet capacity in designing inlet spacing. Maximum allowable inlet spacings are obtained for fixed maximums to flow depth and spread for the channel flow, and fixed maximums to over-lane water film thickness. The impact of slope orientation on inlet spacing was then analyzed. Having the results for varying slope orientations is crucial especially in changing terrains where perhaps each inlet spacing needs updating with changing slope, and spacing is critical for optimal drainage

The sizing of inlets under various slope orientations is important as the capacity varies with slope orientation. Curb opening inlets, the vertical openings at curbs, were investigated in the 4th chapter for they non-interfere with the roadway and non-clog being an effective candidate to most areas. A detailed analysis of the foundational models of curb opening inlets are presented for various designs of flush orientations located on-grade. These include simple vertical openings on curbs (Izzard 1950; Li 1954; Uyumaz 2002; HEC-22; Muhammad 2018), openings with warped pavement in the close vicinity to increase capture capacity (Izzard 1950; Li 1954; Izzard 1977; HEC-22; Muhammad 2018), and openings with not only the vicinity of the inlet depressed but also the approach gutter to increase drainage capacities further (Soares 1991; Uyumaz 1992&2002). The existing models were studied in detail and the misattributes to fundamental models were brought to the light.

1.3 Hypothesis

Roadway drainage is crucial for the safety of users, properties, and the roadway itself. Geometric design of roads impacts the water film thickness for the over-lane flow with the variations in flow path length. Water film thickness could be calculated integrating a kinematic wave method to over-lane flow. Geometric design of roadways rules the inlet spacing beside various other factors and must be considered in designing inlet spacing. Curb opening inlet literature have some mixed understanding of the important works and Hydraulic Engineering Circular 22 (HEC-22) has some mixed concepts regarding the different depression geometries and the models do not capture them systematically that the design equations should be updated.

2. HYDROLOGICAL CONSIDERATIONS IN DESIGNING ROADWAYS: AVOIDING HYDROPLANING

High water depths on lanes risk safety. If the increased water depths create pressures equal to or more than the weight of the vehicle, the vehicle hydroplanes—starts riding on the water with a lack of directional control and braking ability. There are two sources of water that create dangerous depths: (1) intrusion of gutter flow into the lanes, and (2) precipitation on the road surface. Latter is reduced through altered pavement cross slopes. Ross and Russam (1968) and Gallaway et al. (1979) investigated water depths experimentally with different roadway geometries; however, their solutions are limited to the tested conditions. Especially for existing roads, Wolff (2013) obtained a numerical solution to shallow water equations in two-dimensional form. Cristina and Sansalone (2013) developed a rainfall-runoff kinematic wave model for highways, but the solution remains in differential form, not practical for use, and they were only concerned with the concentration time of the flow. Studies on drainage facilities are concerned with the sideway gutter flow and its spread, but the flow contributing to hydroplaning directly from the roadway surface is less studied, and the geometric limits to antihydroplaning values are not well established.

To begin investigating flow widths from the curb, water on the roadway surface should be recognized; thus, the conditions leading to dangerous depths in the absence of gutter flow will be investigated under various design rainfall intensities. This section employs a kinematic wave equation (KWE) and provides a water depth solution for roadways, which is then tested against several studies from the literature for verification. A wide range of rainfall intensity and cross slope values are entered into this solution to examine the significance of cross slope in combination with changing road width and grade. These flow depths could be compared with antihydroplaning flow depths obtained from the studies linking water depths to speed to get optimal cross slopes and peak flow in determining the size of the collection facilities.

2.1 Background

Shortening the travel distance of a raindrop that lands on the road surface results in a water level decrease, which is possible by adjusting the cross slopes. The distance is determined by the angle alpha (α) (Figure 2.1A), which is the angle roadway centerline makes with the resultant of the cross and longitudinal slopes S_x and S_L, respectively (Figure 2.1B). If $S_L = 0$, then $\alpha = \pi / 2$ and that reduces the travel path to roadway width, flow lines perpendicular to the curb—under the assumption that no ruts, local indentations, or bumps exist. However, in case of $S_L \neq 0$, the perpendicular assumption fails, increasing the length of flow. As the cross slope is a geometric factor that can best be controlled by the roadway designer, selecting an optimal value is essential for safer road design.

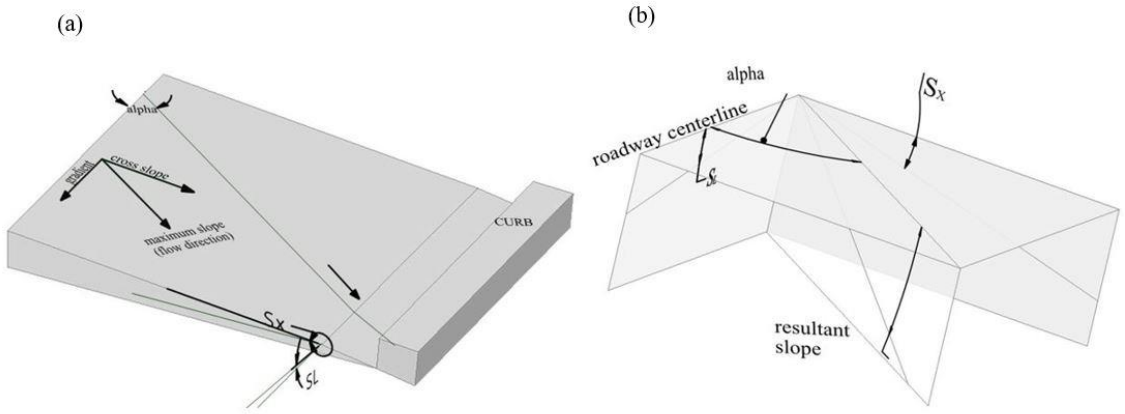

Figure 2.1 : Longitudinal and cross slopes forming the resultant, which helps obtain alpha and flow direction: (a) on a roadway section and (b) slopes at a magnified view.

2.2 Sheet Flow Solution to Traffic Lanes for Estimating Water Depths

Water on a roadway may move as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. Time of concentration—the time required for rainfall landing on the farthest point of the roadway to reach the inlet-opening lip—can be calculated as the sum of the travel times, T_t's, within the various consecutive flow segments and some of these components are also essential for estimating the flow depth at the pavement-gutter intersection. Charbeneau et al. (2009) conducted experiments measuring water depth-runoff on various roadway geometries under different rainfall intensities for sheet flow and concluded that

hydraulic effects of rainfall were negligible. McCuen and Spiess (1995) investigated the limiting criteria for sheet flow. They concluded that for the composite parameter $nL/\sqrt{S}$ [L], values below 30 for the International System of Units (SI) (100 for the US customary units, US) gives acceptable errors—where n is the Manning roughness coefficient, L is the flow length m (ft.), and S is the surface slope. As long as sheet flow exists, a kinematic wave equation may be employed to estimate the time of concentration, but if it does not, friction slope does not equate to the bed slope (i.e. gravity and friction forces are not balanced), and thus kinematic wave assumption fails. For the most part, it is safe to assume that the criteria holds for the flow from the roadway centerline up to the pavement-gutter line, although it may not be the case for the gutter flow. In this study, the concern is the part up to the pavement-gutter intersection, so the sheet flow solution suffices with a single travel time, T_{ti}.

With the assumption Manning's equation works for the friction losses for sheet flow:

$$V = \frac{K_M}{n} z^{2/3} S^{1/2} \tag{2.1}$$

where K_M is the unit conversion factor 1 for SI and 1.486 for US units; n is Manning's roughness coefficient (Table 2.1); z is the flow depth, used in place of hydraulic radius for shallow flow and S is the energy grade line m m^{-1} (ft. $ft.^{-1}$), which is equivalent to the pavement slope, defined as $S = \left(S_x^{\,2} + S_L^{\,2}\right)^{1/2}$ (Figure 2.1B). If $S_L = 0$, S equals the cross slope of the pavement, S_x. Equation (2.1) for unit discharge:

$$q = Vz = \frac{K_M}{n} z^{5/3} S^{1/2} \tag{2.2}$$

From this expression, one can see sheet flow has the form (Henderson and Wooding 1964):

$$q = kz^m \tag{2.3}$$

where $k = K_M S^{1/2}/n$ and $m = 5/3$. As Δy and $\Delta t \to 0$ inflow and outflow can be written as follows:

$$\text{inflow: } (q_y + I\Delta y)\Delta t$$
$$\text{outflow: } (q_y + \frac{\partial q}{\partial y}\Delta y)\Delta t \tag{2.4}$$

where q_y is the unit discharge flow in the longitudinal direction in $m^2\ s^{-1}$ ($ft.^2\ s^{-1}$) and I is the source term (rainfall intensity) in mm hr^{-1} (in. hr^{-1}) (Figure 2.2). Difference between the inflow and outflow in equation (2.4) equals the change in storage:

$$(q_y + I\Delta y)\Delta t - (q_y + \frac{\partial q}{\partial y}\Delta y)\Delta t = (z + \frac{\partial z}{\partial t}\Delta t)\Delta y - z\Delta y \tag{2.5}$$

Simplifying equation (2.5), one obtains the continuity equation for the sheet flow:

$$\frac{\partial q}{\partial y} + \frac{\partial z}{\partial t} = I \tag{2.6}$$

Table 2.1 : Manning's roughness coefficient (n) for overland sheet flow (Equation (2.9)), after HEC-22.

Surface Description	Manning Number (n)
Smooth asphalt	0.011
Smooth concrete	0.012
Ordinary concrete lining	0.013

Using equation (2.6) in the form $\left(\frac{dq}{dz}\right)\frac{\partial z}{\partial y} + \frac{\partial z}{\partial t} = I$, and applying the method of characteristics, one obtains: $\frac{dy}{dq/dz} = \frac{dt}{1} = \frac{dz}{I}$. Hence

$$\frac{dz}{dt} = I \rightarrow z = IT_{ti} \tag{2.7}$$

where T_{ti} is the travel time for the first segment of flow, i.e. the sheet flow travel time, which is the only flow investigated in the current study. Since neither z nor T_{ti} is known, other components should also be considered. Using equation (2.3) along with $\frac{dy}{dt} = \frac{dq}{dz}$ (obtained from the method of characteristics) and inserting z in equation (2.7):

$$\frac{dq}{dz} = mkz^{m-1} \rightarrow \frac{dy}{dt} = 5/3k(IT_{ti})^{2/3} \tag{2.8}$$

Solution of this final equation after inserting k as defined in equation (2.3) yields:

$$T_{ti} = \frac{K_T}{I^{0.4}}\left(\frac{nL}{\sqrt{S}}\right)^{0.6} \tag{2.9}$$

where, T_{ti} is the sheet flow travel time, in mins; n is roughness coefficient (Table 2.1); L flow length, in m or ft.; I rainfall intensity, in mm hr^{-1} or in. hr^{-1}; S surface slope; and K_T a coefficient equal to 6.92 or 0.933, units in SI and US, respectively. This is also the equation US Federal Highway Administration suggests using.

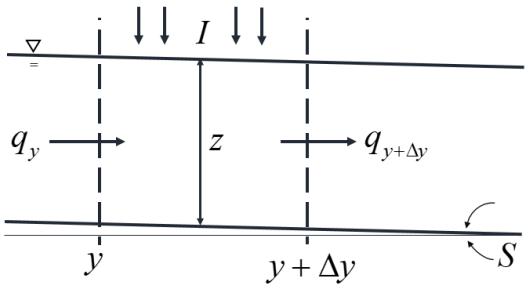

Figure 2.2 : Definition sketch for the continuity equation in the longitudinal section for sheet flow.

The angle resultant slope makes with the roadway centerline in Figure 2.1, α, is the crucial part in determining the length of flow L in equation (2.9), and defined:

$$\alpha = \sin^{-1}\left(S_x / S\right) \tag{2.10}$$

The flow length, $L = b/\sin\alpha$ in m or ft. in the above expression; S, as used in equations (2.9) and (2.10), is a function of longitudinal and cross slopes and introduced earlier in the description of equation (2.1) as the resultant slope. Using that for S, α becomes $\alpha = \sin^{-1}\left(S_x / \left(S_x^{\,2} + S_L^{\,2}\right)^{1/2}\right)$ in equation (2.10). As a result, $L/\sqrt{S}$ in equation (2.9) becomes:

$$\frac{L}{\sqrt{S}} = \frac{b/\sin\alpha}{\sqrt{S}} = \frac{b/(S_x/S)}{\sqrt{S}} = b\frac{\left(S_x^{\,2} + S_L^{\,2}\right)^{0.25}}{S_x} \tag{2.11}$$

Equation (2.11) inserted in equation (2.9) gives:

$$T_{ti} = \frac{K_T}{I^{0.4}}\left(n\frac{b/\sin\alpha}{\sqrt{S}}\right)^{0.6} = \frac{K_T}{I^{0.4}}\left(nb\frac{\left(S_x^{\,2} + S_L^{\,2}\right)^{0.25}}{S_x}\right)^{0.6} = \frac{K_T}{I^{0.4}}(nb)^{0.6}\left(\frac{S_x^{\,2} + S_L^{\,2}}{S_x^{\,4}}\right)^{0.15} \tag{2.12}$$

Using equations (2.7) and (2.12) one obtains:

$$z = K_T (Inb)^{0.6} \left(\frac{S_x^{\,2} + S_L^{\,2}}{S_x^{\,4}} \right)^{0.15} \tag{2.13}$$

Equation (2.9) ignores the transverse path a droplet takes from the gutter-pavement line to the curb line as L is the flow length on the pavement excluding the gutter flow. This second leg of flow goes unmentioned because, despite curb-opening line mandates that all water travel across the gutter, it is unimportant in terms of the time spent and may be ignored. Additionally, not only does the accumulating water slowly form channel flow (therefore cannot be treated as sheet flow), but also (because of incoming flow in the gutter) not all over-lane flow is simultaneously conveyed to the curb line.

The flow on the pavement through lanes is considered dominated by sheet flow within the limits defined by McCuen and Spiess (1995), and the relationship is obtained between the travel time and roadway geometry. Travel time is dominated by the flow length, L, for a pavement surface that ranges from the roadway width to the roadway length bounded by a sump, for $S_L = 0$ and $S_x = 0$, respectively. Using travel time, the depth of flow on the road is obtained from $z = T_{ti} I$ for a design rainfall; any spills over into the road lanes from the continuous gutter section should be checked. Depth of water is important for moving traffic, and knowing which geometries lead to hazardous hydroplaning helps set optimal cross slopes for a given rainfall intensity to avoid unwanted incidents.

2.3 Verification of the Model

The amount of water depth, which is much affected by the cross slope of the road, is crucial in causing hydroplaning, and the correctness of water depths obtained using the kinematic wave equation depends on how well the travel time is estimated. Thus, this section checks whether the solution for travel time compares well with the literature based on experimental data.

Various studies are conducted to determine the depth of rainwater experimentally. One of the earliest studies related to roadways was performed in the UK by Ross and Russam (1968) on an 11 m x 5.5 m platform. equation (2.14) is reached after testing

two surfaces under various cross slopes with resultant ranging from 0.5% up to about 8% (flow path up to 11 m) and rainfall intensities from 10 mm hr^{-1} to 200 mm hr^{-1}.

$$z_{R\&R} = 0.474(LxI)^{1/2} S^{-1/5} \tag{2.14}$$

where $z_{R\&R}$ is water depth in mm as reported by the Ross and Russam (1968) solution; L is drainage length in m; I is rainfall intensity in mm hr^{-1} and S is the slope of the flow path. For the comparison, travel times for Ross and Russam (1968) were obtained using equation (2.7) after finding water depth from equation (2.14).

Wong (2005) compared several time of concentration formulas with experimental data they conducted and stated in the paper that Chen and Wong (1993) equation estimates the best. However, based on the coefficient of determination values, R^2 — the quotient of the sum of squared errors and the total sum of squares—United States Corps of Engineers' (1954) (USACE) formula estimates the time of concentration better than the Chen and Wong (1993) solution. USACE equation provides travel time to be:

$$z_{R\&R} = 0.474(LxI)^{1/2} S^{-1/5} \tag{2.15}$$

where S_L is the longitudinal slope of the road and b is the width, while I is the rainfall intensity in mm h^{-1}.

Cristina and Sansalone (2003) mentioned the rareness of adopting kinematic wave equation for modeling impervious surfaces subject to traffic loadings. They developed a kinematic wave model and obtained flow depth using the finite difference method. They also compared their results with an experimental model where water moves perpendicular to traffic flow with a 2% cross slope. The data from Cristina and Sansalone (2003) are compared with their results as well as with USACE and Ross and Russam (1968)'s empirical equation as provided in equation (2.14). Results for Cristina and Sansalone (2003) solution and the solution provided in this paper include various Manning's n. Between the Cristina and Sansalone results and the ones in this paper, there is close to no difference in time of concentration (Figure 2.3), yet the solution provided here is easier to apply. In comparison with USACE, the kinematic wave solution outperforms; however, it should be kept in mind that the experiments are limited. Although Wong (2005)

showed that the USACE solution is the best in predicting the time of concentrations, our results adding kinematic wave solutions and solution from Ross and Russam (1968) all outperform the USACE results, which highly overestimate the experimental. The critical point in using the kinematic wave equation is in using correct Manning's number. The general trend in overestimation of the kinematic wave model compared to the experimental data may be attributed to the abstractions in the runoff process.

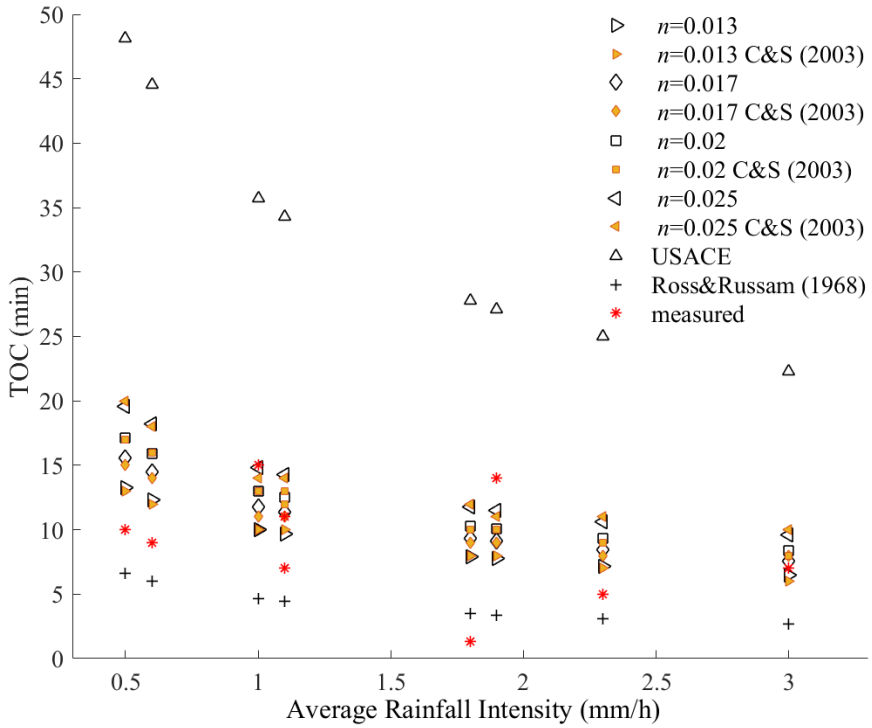

Figure 2.3 : Comparing our kinematic wave model with that of Cristina and Sansalone (2003) model (referred to as C&S in the figure) and of the experimental results for 2% cross slope, 0.4% longitudinal slope for a road width of 20 m which remains within the criteria limits posted by McCuen and Spiess (1995).

2.4 Water Depth Variations under Different Design Rainfalls and Roadway Geometries

Water depths are determined from equation (2.13), and the results are plotted in Figure 2.4 and Figure 2.5. The roadway was considered to have a width of 15 m ($b = 15$ m) with multiple lanes and a Manning coefficient of $n = 0.016$, slightly higher than provided in Table 2.1. Figure 2.4 shows water depths topping over 4 mm even with the shortest paths (i.e. $S_L = 0$) under design rainfall intensities of 200, 250, and 450 mm hr^{-1} with the cross slope ranging from 1 to 6%. Depths reach up to 6 mm with rainfall intensity of 450 mm hr^{-1} while 250 mm hr^{-1} barely reaches 4 mm depth

after the 10th meter hits in from the crown. 10-year frequency constitutes the norm for roadway drainage practices, and it is understood that 450 mm hr^{-1} rainfall intensity is rare for a 10-year design frequency, but it provides a window into how the changes occur. At 15 m, for a cross slope of 1%, the water depth reaches up to 4.7 mm, while for 6% cross slope, the depth remains at 2.7 mm for a rainfall intensity of 200 mm hr^{-1}; for 250 mm hr^{-1}, 5.36 mm and 3.13 mm, respectively (Table 2.2; Figure 2.4). It is obvious from Table 2.2 that at 15 m, while water depth is 5.36 mm for 1% cross slope and 4.35 mm for 2% with a difference of 1.01 mm, the difference is 0.17 mm when the cross slopes are 5 and 6% with water depths 3.3 and 3.13 mm, respectively (Figure 2.4). On the other hand, while water accumulation is 1.87 mm for 1% cross slope at the width of 3.25 m, it increases only to 2.84 mm at 6.5 m with the difference of 0.97 mm. For $b = 6.5$ m under 250 mm hr^{-1} rainfall intensity, the water depth is 2.84 mm for 1% cross slope, 2.04 for 3%, and 1.66 mm for 6% cross slope while for $b = 15$ m the depths are 5.36, 4.35, and 3.85 for cross slopes of 1, 2, and 3%, respectively.

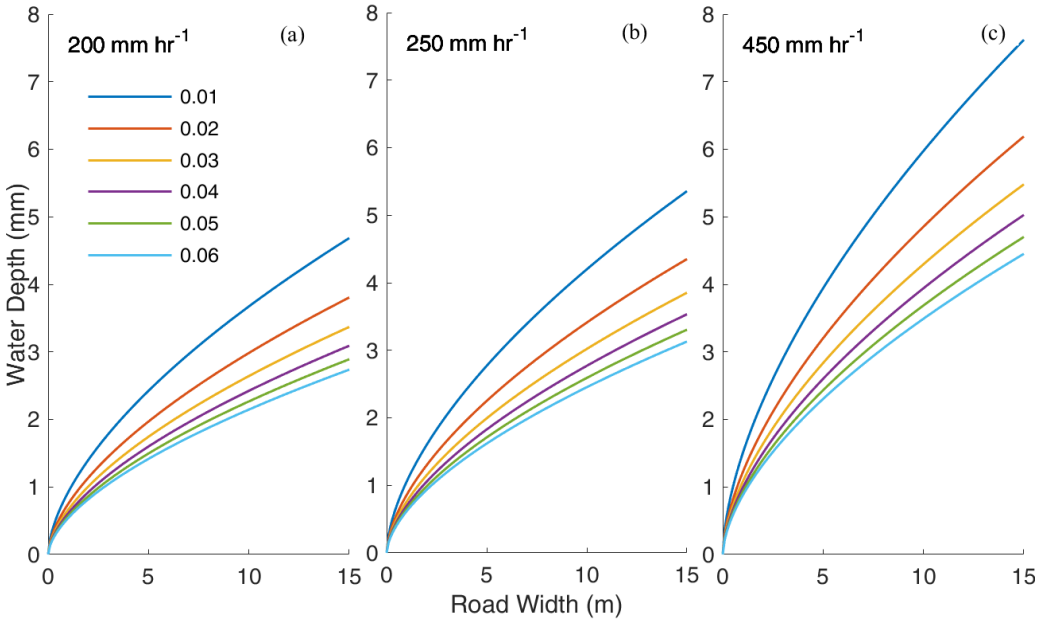

Figure 2.4 : For $S_L = 0$, the effect of changing cross slope on the flow depth as a function of width is provided for rainfall intensities (a) 200, (b) 250, and (c) 450 mm hr^{-1}.

Figure 2.5 shows the water depths for roadway grades of 1, 5, and 10% for rainfall intensities equal to 150, 200, and 250 mm hr^{-1}. Each subplot shows the corresponding water depths for cross slopes ranging from 1 to 6% within the

roadway profile, as in Figure 2.4. Water-depth versus road-width shows that for $b = 6.5$ m, $S_x = 2\%$, and 150 mm hr^{-1} rainfall the water depths are 1.94, 2.00, 2.61, and 3.16 mm for 0, 1, 5, and 10% longitudinal slopes, respectively. For 200 mm hr^{-1} at $b = 15$ m water depths reach to 4.68, 5.2, 7.64, and 9.6 mm for the same longitudinal slopes of 0, 1, 5, and 10%, respectively.

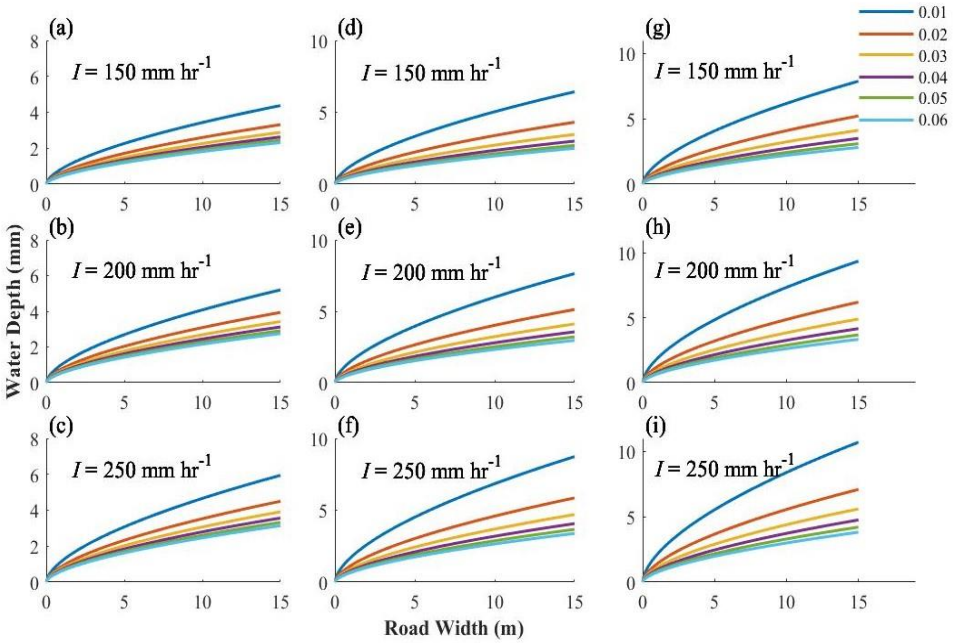

Figure 2.5 : Flow depths through roadway width for $S_L = 0.01$ in (a), (b), and (c); $S_L = 0.05$ in (d), (e), and (f); and $S_L = 0.1$ in (g), (h), and (i) with cross slopes ranging from 1% to 6% under rainfalls of 150, 200 and 250 mm hr^{-1}.

Table 2.2 : Water depth (mm) values with $I = 250$ mm hr^{-1} intensity under flat grades for various roadway widths.

b(m) / S_x	0.01	0.02	0.03	0.04	0.05	0.06
3.25	1.87	1.52	1.35	1.23	1.15	1.09
6.5	2.84	2.30	2.04	1.87	1.75	1.66
15	5.36	4.35	3.85	3.53	3.30	3.13

The sheet flow assumption holds for all these cases, and therefore, the solution provided is valid.

2.5 Discussions

Effectively planning urban traffic involves many factors for the decision-makers, one of which is avoiding hydroplaning during rainstorms. Studies linking water depth and speed document that higher water depths increase hydroplaning risks at lower speeds. Hydroplaning may occur at speeds of above 80 km hr^{-1} with a water depth of 2 mm. However, depending on various factors influencing the conditions, hydroplaning can take place at lower speeds and depths (HEC-22). Gurganusa et al. (2021), in their study to find flow depth using Light Detection and Ranging for existing roads speculated that "hydroplaning speed was at least 16 kph below the posted speed limit." Gallaway et al. (1979) recommend limiting water depths at or below 4 mm to prevent hydroplaning. Risk of partial hydroplaning continues at lower depths; and in fact, Huebner et al. (1986) in an attempt to obtain a single criterion for hydroplaning finds combining a model they obtain using Agrawal et al. (1977) data for below 2.54 mm water film thickness and Gallaway et al. (1979) model for above 2.54 mm that 2.5 mm is the limit for staying safe at speeds higher than 77 kph. It is assumed in this paper that flow depths below 2 mm do not constitute a danger for dynamic hydroplaning around the speeds of 80 km hr^{-1} (55 mph) based on Huebner et al. (1986) model, and 4 mm is the limit depth above which must be avoided based on Gallaway et al. (1979) model. The analyses exhibit the water depth results using kinematic wave equation with various longitudinal and cross slope values as the design rainfall intensity changes. Factors influencing hydroplaning are many. Design rainfall intensity is one that affects water depths immensely, and is fixed for a given region; Figure 2.4 shows how an increase in rainfall intensity leads to an increase in water depths. As a significant factor, the intensity may be obtained via solutions specific to the region under investigation to cope with changing climate. Increases in the longitudinal slope and roadway width are two other factors that lead to higher accumulation of water, and in certain cases, the designer may not have much control over them. Considering a minimum of 1% cross slope exists in any road, Figure 2.6 shows the longitudinal slopes and rainfall intensities that create 4 mm film thickness over-lanes. It shows that for a given lane number, any intensities below the shown profiles and grades are safe and cross slope optimization may not be necessary, but for intensities and grades above these limits, the optimization is unavoidable (i.e., 1% cross slope is insufficient) as the hydroplaning risks increase. For example, for a 3-

lane roadway with a cross slope of 1%, intensities up to 75 mm hr^{-1} require no further evaluation for any grades below 10% (Figure 2.6; Table 2.3)—AASHTO's Green Book (2018) recommends a minimum of 1.5% cross slope on curbed pavements unless a steeper gutter exists, in order to prevent larger intrusions onto lanes.

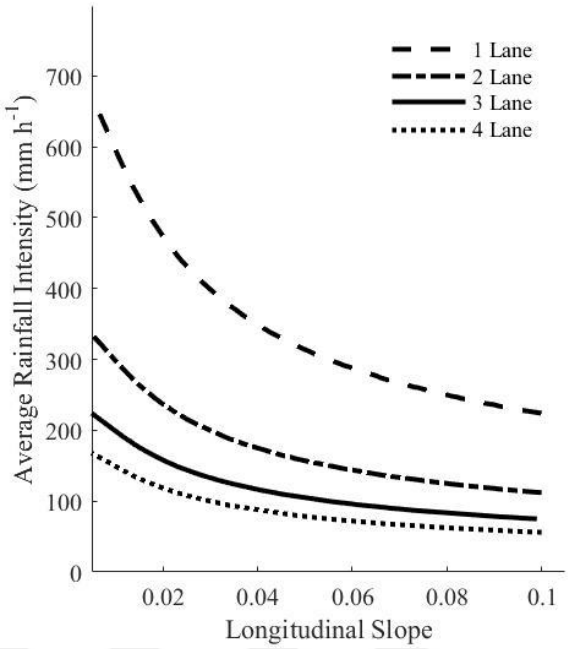

Figure 2.6 : Influence of increasing lane number on reaching a hydroplaning depth of 4 mm for a cross slope of 1%. Lane width is assumed 3.25 m.

Table 2.3 : With a minimum cross slope of 1% and a maximum grade of 10%, hydroplaning free rainfall intensities.

# of Lanes	1	2	3	4
Rainfall intensity	224 mm hr^{-1}	112 mm hr^{-1}	75 mm hr^{-1}	56 mm hr^{-1}

Cross slope is one of the primary factors that help lower water depths. Considering 3.25 m a lane width, a two-lane roadway under 250 mm hr^{-1} design intensity leads to partial hydroplaning risks with water depths tipping 2 mm below 3% cross slopes, while higher values easily eliminate the risks. At the width of 15 m, full hydroplaning risks are faced if the cross slope is below 3% (Table 2.2). Figure 2.7 shows many configurations with the full hydroplaning limit of 4 mm is marked with dashed lines; plots start at 2 mm, the start of partial hydroplaning depth. Adjusting cross slopes prevents hazardous water depths for roadway grades up to 10% under the design rainfall intensity of 250 mm hr^{-1} while for flat grades, cross slopes above 2% are optimal for roadway widths up to 15 m (Figure 2.7). It should be noted that

increasing cross slope by the same percent produces different outcomes depending on the transverseness of the reference slope as increasing it from 1% to 2% with a difference of 1% is more pronounced than the difference between 5% and 6% with 1.01 mm and 0.17 mm depths, respectively, at 15 m with zero grade (Table 2.2;Figure 2.4; Figure 2.7). In other words, increasing the cross slopes after a certain value does not maximize the water depth drops and might be impractical in terms of comfort, but a minor increase at flatter percent leaves higher impacts in avoiding hydroplaning.

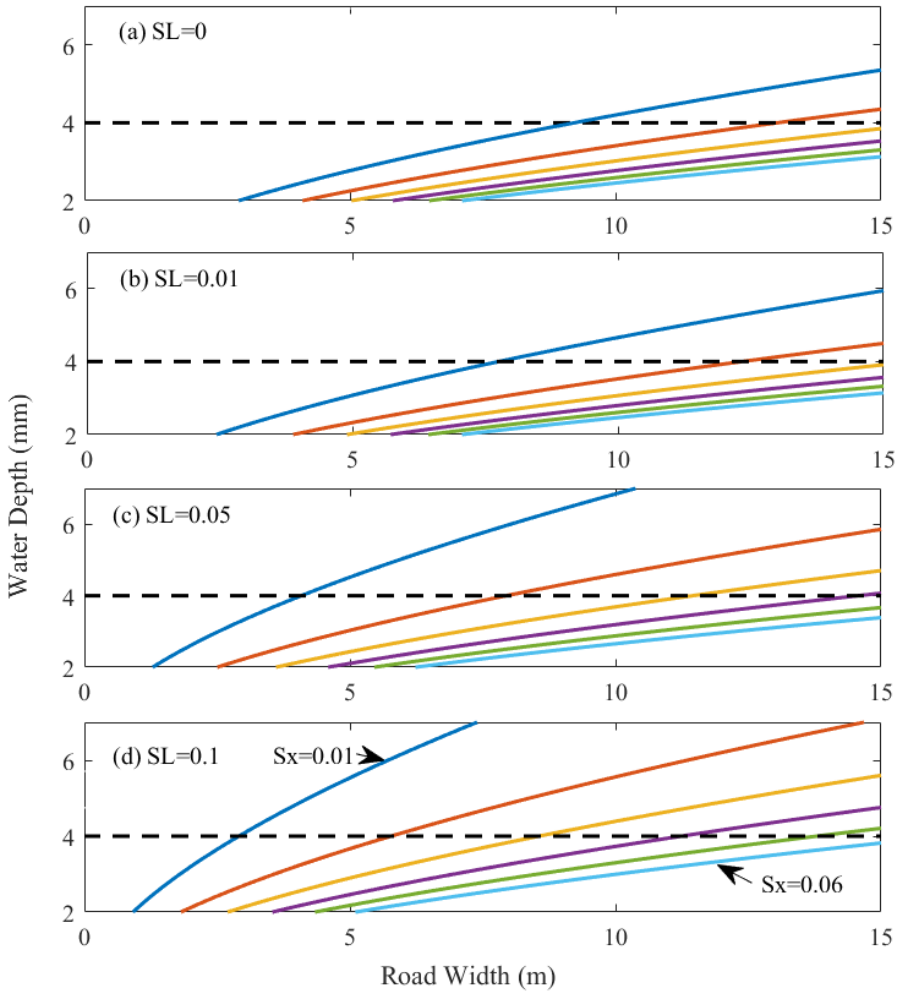

Figure 2.7 : Water depths profiles for 250 mm hr^{-1} rainfall intensity, a 4-mm depth marking is used for ease in interpretation. Each color represents a different cross slope (0.01 to 0.06), with equal increments.

Flow depth may also be affected by factors besides the flow length, such as texture and tread depths. For bald tires, for example, hydroplaning may occur at depths as low as 0.3 mm (0.01 in.) while the treaded tires reportedly do not hydroplane even on low texture roads with flow depths below 2 mm (0.08 in.) as the grooves allow water

to channel away (Gallaway et al., 1979) (1.6 mm (2/32 in.) is the standard tread depth and usually risks start when water depths are higher than groove depths). Guo et al. (2013) considered the tread depth to be much deeper, but for this study, standard depth was considered in deciding hydroplaning standards.

2.6 Conclusions

The most critical aspect of any roadway design is its capacity to self-drain for avoiding hydroplaning water depths. Factors affecting flow depth on a roadway include design rainfall intensity, roadway width, longitudinal slope, and cross slope. On roads where most or all water originates from precipitation, cross slope gains importance for drainage. The steepness of the cross slope is limited for safety considerations (the vehicle tends to veer towards the low edge of the pavement), and our results for up to 6% cross slopes show that increasing cross slopes have diminishing reduction effects on water depths. Overall, our results show that cross slope optimization is a safe way to avoid hydroplaning depths for grades up to 10% and widths up to 15 m for intensities below 250 mm hr^{-1}. Roadway width, as much as mandated by the need, may be judged based on the constrain, provided that a certain width is inconvenient in terms of drainage. In brief, while some factors that affect roadway water depth may not be controlled, the designer may restrain the cross slope, and the diminishing decrease influence of increasing cross slopes on water depths is shown. Furthermore, the safest slope values are coupled with design intensities without hydroplaning threat beyond which cross slope adjustment is required to prevent hydroplaning as a result of shortened flow paths, and hence, water depths. With a zero cross slope, roads act as channels, not diverting the flow to the sides, and constantly cause an increase in the flow depth until sag is reached and should not be practiced.

3. IMPACT OF SLOPE ORIENTATION ON INLET SPACING: GUTTER FLOW ANALYSES

The ability of a roadway to drain itself is one of the main characteristics looked for (AASHTO). It is essential for the safety and comfort of the users including bicyclists, pedestrians, and the disabled. Standing water and any amount of channelized flow on roadways create nuisances to the listed users, but the extent of encroachment into the lanes and the water film thickness over the lanes are crucial for motorists with relatively high speed. Geometric design and environmental conditions (such as rainfall intensity) alter the consequences road users face. Based on differences in environmental conditions and local cultures and practices, regions from different countries, states, and even cities publish different guidelines for optimal design to minimize the unwanted. The guidelines cover a wide range of subjects from size and type of inlets, which capture the channelized flow to convey into enclosed drains, to the decision of slope orientation. Recent modifications also include planning for climate change which along with continuing urbanization creates new challenges; Walsh (2011), for example, mentions designing with 20% increase for winter rainfall to compensate for the excesses in the UK. On the flip side, planning for extreme situations or under stringent criteria add to the cost or compromise the comfort that may be unnecessary. Thus, the need for continuously evolving standards for proper planning and practice.

Roadway drainage requires considering over-lane drainage, gutter flow, and inlet capacity (HEC-22). Hydroplaning, among other factors, is of the primary concern in which skid resistance between the tire and the pavement drops to zero, and the vehicle loses control. For the most part, the solution entails rearranging roadway geometries for its prevention. Gallaway et al. (1979) and Ross and Russam (1968) conducted experimental analyses to determine the hydroplaning causing water depths (film thickness) experimentally under various parameters for surface flow. Cavdar and Uyumaz (2022) conducted cross slope-rainfall intensity analysis to determine optimal cross slopes for avoiding hydroplaning. The second concern is limiting the

channelized flow to reduce the nuisances to all users. AASHTO recommends off-the-lane-distances should remain constant, to give the driver a sense of consistency with changing terrains; Spaliviero et al (2000) specifically highlight the importance of paying attention to the changes in longitudinal slope to account for the capacity of the channel flow. Thus, without making drastic changes to the width of off-lane distances, the designer should consider the changes to the geometry. There is a large literature that focuses on limiting the flow spread from the curb (AASHTO, 2018; HEC-22) for a range of cross slope values; this leads to various flow depths as a result of altered cross section, but the standards seem to fall shy in checking the depth of flow. HEC-22 endorses limiting the flow depths to curb height (as if the concern is no longer the roadway users) and fixes the criterion for the inlet spacing to maximum allowable flow spreads. Wong (1997) does spacing analysis (dismissing hydroplaning) also using fixed spread for the inlets used in Singapore. Gomez et al. (2002), however, limit the maximum flow depth at the curb to 3 cm, stating that it is the "sole thickness of a normal shoe." HEC-22, being the urban drainage design manual of the US Department of Transportation, provides guidance for locating inlets, starting from the ones that must be prioritized based on geometric control (e.g., at sags and pedestrian crosswalks). Wong (1994) proposes a kinematic wave solution to calculate time of concentration for the gutter flow. Using that he obtains the rainfall intensity to find the amount of flow due to rainfall and then determines inlet spacing for a commonly used Singapore inlet. Wong and Moh (1997), using the results of Wong (1994) analyze the effect of allowable flow spreads on the inlet spacing; they find an exponential relationship in that an increase in the spread results in an exponential increase in the inlet spacing. The concentration time models were then tested against experimental data in Wong (2001) and found to be applicable; however, these studies consider perpendicular flow direction to the curb face, despite nonzero road grades. While distributed models may be of interest, due to the uniformity of the road surface and relatively small size of basins, it is accepted that a lumped model should work.

In this study, the maximum allowable inlet spacing is analyzed via setting three criteria: a fixed maximum to flow depth and spread for the channel flow, and a fixed maximum to over-lane water film thickness; then the results are graphed for various uniform slope orientation under a wide range of rainfall intensities. The impact of

slope orientation in inlet spacing is tested; wondering whether slope orientation governs the inlet spacing or whether other factors rule. The standards vary with the purpose of roadway, so the analyses were done in two groups and the results were analyzed for slope variations in both groups. Those are for high volume-high speed roads with no tolerance to flow accumulation or high water film thicknesses and for low speed-high pedestrian and bicyclist volume roads which also require lower flow depths though the encroachment into the lanes could be tolerated to an extent. Then the inlet spacing is determined for conditions that fit best for the given geometrics (especially cross slopes) and environmental conditions. This was done combining kinematic wave equation solution of Cavdar and Uyumaz (2022) to dismiss the conditions that lead to hydroplaning depths with the Rational Method and Manning's equation put together to obtain water depths and inlet spacings when an inlet of full capture capacity is concerned.

3.1 Obtaining Maximum Inlet Spacing

For any road, its design is important to avoid hydroplaning by preventing high flow depths (i.e., water film thicknesses). For roads with a curb, spread and the depth of channel flow adjacent to the curb are also indispensable, and those are the main criteria that mandate inlet spacing for the removal of flow. The channel flow depth is critical in high speed roads as well as in roads of pedestrian use as in both cases there are different causes of security concern. Table 3.1 shows the models combined in obtaining the maximum inlet spacing under the conditions specified in the following tables.

Table 3.1 : Models used.

Model	Goal
Cavdar&Uyumaz (2022)	Computing over-lane flow depth for obtaining hydroplaning free designs
Izzard-modified-Manning (Izzard 1946)	Obtaining maximum possible channel flow that runs adjacent to the curb and marks the place of an inlet for removal
Rational Method (Kuichling 1889)	With flow from Izzard (1946), finding the maximum permissible inlet spacing

3.1.1 Obtaining proper configurations

Before considering inlet spacing, the designer must first check the over-lane flow depth—usually the depth at the edge of the lane closest to the curb—is low enough to provide hydroplaning-free conditions to the users. There are several methods to calculate the over-lane flow depth, and in this study, the Kinematic Wave Equation method Cavdar and Uyumaz (2022) propose was used:

$$z_r = K_T (Inb)^{0.6} \left(\frac{S_x^{\,2} + S_L^{\,2}}{S_x^{\,4}} \right)^{0.15} \tag{3.1}$$

where K_T is a coefficient equal to 6.92 or 0.933, units in SI and US; I is the rainfall intensity (source term), mm hr^{-1} or in. hr^{-1}; n is Manning's roughness coefficient for sheet flow (Table 3.2); b is the distance perpendicular to the curb from the end of gutter or shoulder to the highest point contributing to the pavement flow, m (ft.) (Figure 3.1), and S_x and S_L are the cross and longitudinal slopes, respectively. The over-lane flow depth, z_r, should be kept within 4 mm for hydroplaning-*free* roadway design.

3.1.2 Determining channel flow (flow depth)

In determining inlet spacing, there are two factors involved in this study: maximum spread and maximum flow depth that could be tolerated. In either case, Manning's equation (Manning 1895) links these to flow amounts; however, presence of cross slope in roads necessitates a modification to Manning's equation, and it is remodeled by Izzard (1946):

$$Q = \frac{3K_M}{8} \frac{S_L^{\,0.5}}{nS_x} z^{8/3} \tag{3.2}$$

where K_M is a unit conversion factor defined as K_M = 1.0 m$^{1/3}$ s^{-1} or 1.486 ft.$^{1/3}$ s^{-1}, Q is in m^3 s^{-1} or ft.^{3} s^{-1} and z is flow depth for a uniform cross section in m or ft. for SI or US, respectively and could be written in terms of spread for the most part; S_L is on-road and S_x is the cross slope, n is Manning's dimensionless coefficient for channel flow (Table 3.2). The maximum value z can take is determined based on a

couple of factors. Now that the maximum flow that may be held is determined, the maximum possible inlet spacing could be obtained.

Table 3.2 : Manning's roughness coefficient (n) for overland sheet flow (for lanes), and channels (equation (3.2)) after HEC-22.

Surface Description	*n*
Smooth asphalt	0.011
Smooth concrete	0.012
Ordinary concrete lining	0.013
Pavement/gutter sections	0.012-0.026 (Manning in open channel, only)

3.1.3 Determining inlet spacing

Rational Method (which states the amount of flow equals the product of rainfall intensity and the impervious surface area) is proper for use in roadways with small catchment areas and is used here to obtain the relationship between the flow amount and the inlet spacing. The equation states (Kuichling 1889):

$$Q = CIA / K_R \tag{3.3}$$

where Q is the peak runoff rate in cms or cfs at a given point; C is a runoff coefficient (Table 3.3); I is the rainfall intensity, in mm hr^{-1} or in. hr^{-1}; A is the drainage area that drains to the location of interest (i.e. inlet opening), in m^2 or $ft.^2$; and K_R the conversion factor is $36x10^5$ or 43200 , all units SI or US, respectively. As evident from dimensionless K_R, while the expression is dimensionally homogeneous, the product IA is not in the standard unit system, so is the need for K_R. Brown et al. (2009) documents C values used in equation (3.3) (Table 3.3)—Cristina and Sansalone (2003) reports C for high intensity storms as 0.6 to 0.9 and for low intensity storms 0.2 to 0.4, abstraction attributed to the traffic.

Table 3.3 : Runoff coefficients for Rational Formula (Brown et al. 2009).

Type of drainage area	Runoff Coefficient, C[2]
Asphaltic street	0.70-0.95
Concrete Street	0.80-0.95

[2] Higher values are appropriate for steeper slopes.

The design rainfall intensity I in equation (3.1) is determined based on rainfall duration equal to time of concentration, t_c, and the design frequency to obtain the rational solution. t_c is the time required for rainfall landing on the farthest point of the *drainage area* to reach to the point of interest, which is the inlet-opening lip and used as the duration of the design storm for calculating peak storm-water runoff rate. Common practice especially in the US and in Australia is to take the rainfall for 10-year frequency and 5-min duration from intensity-duration-frequency curve.

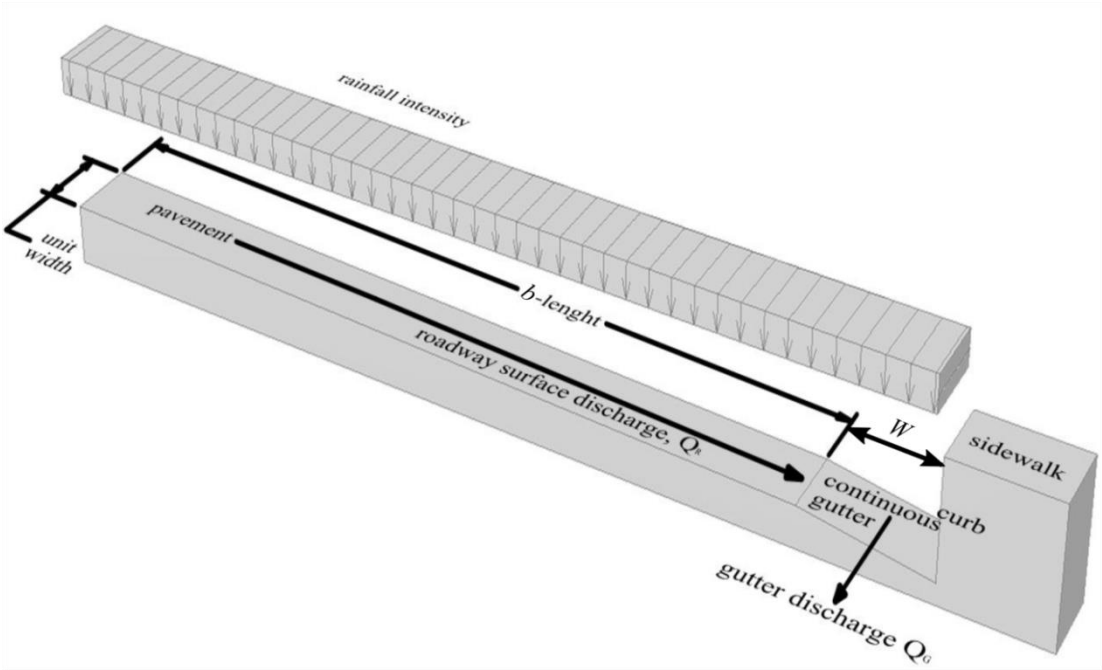

Figure 3.1 : Roadway section with $S_L = 0$ and $S_x \neq 0$. For this study, gutter slope is equal to the pavement slope.

The A in equation (3.3) is essentially equal to the projection of the road section under investigation, for that is the area from which rain falls (**0** top section). For a unit section as in Figure 3.1, which (for our convenience of terms) has the unit *width* of pavement and unit *length* of continuous gutter, the area becomes $(bx1+Wx1)$ and thus the flow for a unit *width* becomes:

$$Q_G = Q_R + Q_W = CI(bx1+Wx1)/K_R \Rightarrow q = CI(b+W)/K_R \qquad (3.4)$$

where, for a given section, Q_R is the amount of over-lane flow, Q_W is the amount of *over-gutter* flow (what falls directly over the gutter), and Q_G is the total precipitation that falls within the centerline of the road and the curb-line. In this work, full

interception of incoming flow by the inlet is considered with a single source term, precipitation; thus: $Q_G = Q_R + Q_W$. The runoff coefficient, C, and the *length* of the pavement area, $y = b + W$ (where b is the length of pavement and W is the length of the gutter or shoulder) are generally known for a given roadway.

Flow at the inlet opening is obtained via multiplying q with the inlet spacing, and as long as the inlet spacing is kept the same (unit width), the area remains the same for a roadway of fixed b and W, regardless of the value of S_L. That flow obtained through multiplying q in equation (3.4) by the inlet spacing gives the same flow as in equation (3.2). If one equates the two and solves for the inlet spacing x:

$$x = K \frac{S_L^{0.5}}{nS_x} \frac{z^{8/3}}{CI(b+W)} \tag{3.5}$$

is obtained where x is in m or ft., K, which is $3/8 K_M K_R$, is 135x10^4 m$^{1/3}$ s^{-1} or 24073.2 ft.$^{1/3}$ s^{-1}, S_L and S_x are the on-road and cross slopes, respectively, n is Manning's dimensionless coefficient (Table 3.2), z is the flow depth at the gutter in m or ft., C is the runoff coefficient (Table 3.3), I is the rainfall intensity in mm hr^{-1} or in. hr^{-1}, and $b+W$ is the distance from the roadway centerline to the curb face in m or ft., all units for SI or US, respectively.

3.1.3.1 Analysis of z in equation (3.5)

In equation (3.5), two potentially unknown parameters are faced: inlet spacing, x, and flow depth at the curb, z. The focus here is to obtain inlet spacing with its maximum allowable values, so the z takes its maximum values. If over-lane flow depths stay within an acceptable range, there are two scenarios for determining flow depth: restricted by maximum spread criteria and/or restricted by maximum allowable depths at the curb. Contribution of the over-lane flow depth to the overall depth at the curb could be neglected as their orders of magnitude differ. Based on this and the uniform transverse slope assumption, potential water depth at the curb face for uniformly sloped surface becomes:

$$z = T_{\max} S_x \tag{3.6}$$

where z is in m or ft., $T_{\max}$ is maximum allowable flow spread in m or ft. (taken equal to W here to prevent intrusion into the lanes), and S_x is cross slope. Any z value calculated using equation (3.6) that is equal or less than $z_{\max}$ is valid for use in equation (3.5); however, if $z > z_{\max}$, then z in equation (3.5) must be replaced with $z_{\max}$ and equation (3.6) should be disregarded as in no case should the depth overtop the set criteria. Again, as long as $z_r < 4$ mm (equation (3.1)), and $z = T_{\max} S_x \le z_{\max}$ (equation (3.6)), inlet spacing in equation (3.5) is solved using the z value in equation (3.6), but if $z = T_{\max} S_x > z_{\max}$, then equation (3.5) is solved for $z = z_{\max}$. Now being left with a single unknown x, equation (3.5) could be solved for inlet spacing.

3.1.3.2 Inlets and their individual spacing

Aranda et al. (2021) mention that Spanish guidelines divide the roadway into small catchments to allow working with individual inlet's flow; using this approach, spacings may be rearranged. In a roadway, if on-grade slope is zero, $S_L = 0$, equally spaced inlets will receive same amount of rainfall for their drainage area is equal (equation (3.3)). That is unlike $S_L \neq 0$ for which only the first and last inlets' spacing needs modification while the inlets placed in between the two continue to receive equal. Lack of respacing the first and the last inlets leads to excess of resources (first inlet) and to potential dangers to the road users as well as the structure via deterioration (last inlet). Respacing is then inevitable in order to keep the inlet design the same while avoiding the downsides of fixed spacing. The area $A1$ in Figure 3.2 is *subtracted* from the first inlet and *added* to the last, assuming that the last inlet is at sump. How large $A1$ becomes is determined by the angle roadway-centerline makes with the resultant vector of cross and longitudinal slopes S_x and S_L, respectively (Figure 3.2):

$$\alpha = \sin^{-1}\left(S_x / \sqrt{S_x^{\,2} + S_L^{\,2}} \right) \tag{3.7}$$

where $\sqrt{S_x^{\,2} + S_L^{\,2}}$ is the resultant slope. To equate the two inlets in flow means equaling them in area, so the inlets must have different spacing for the first and the last. It is straightforward from the geometry that:

$$x_1 = \left(b + W\right) / \tan\alpha \tag{3.8}$$

where b and W are as defined in Figure 3.1. For the first inlet:

$$x_{\text{first}} = x + 0.5x_1 \tag{3.9}$$

The spacing between the last inlet at the sump and the one upstream:

$$x_{\text{last}} \leq x - 0.5x_1 \tag{3.10}$$

It is always safer to design inlets larger for the one placed at the sump or to place in close proximity to each other, so spacing could be shorter than provided above. Equationss. (3.9) and (3.10) gains importance in changing terrains with new spacing needs for each inlet.

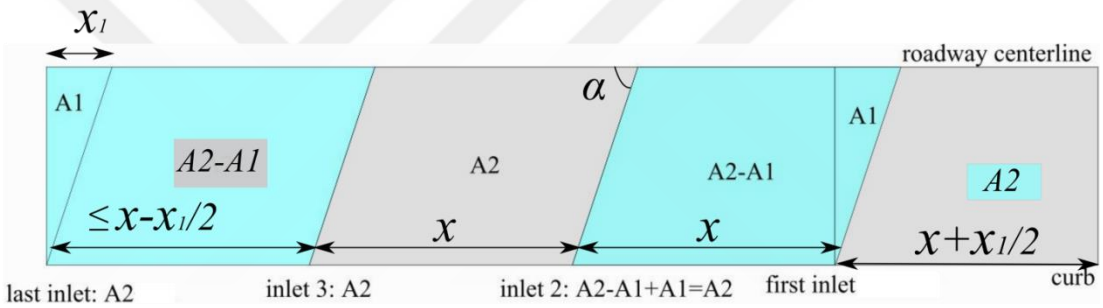

Figure 3.2 : Working from the last inlet to first.

3.1.4 Conditions tested

Curbs are raised elements that help traffic control and roadway drainage; here a vertical curb design is considered. Using the posted solutions, inlet spacing x was obtained under the conditions listed in Table 3.4 and Table 3.5. Since most roads have an average roughness of 0.016 and runoff coefficient of around 0.95, these were employed here along with a 0.011 roughness value for the sheet flow. Lane width impacts the comfort of driving, to an extent prevents accidents, and lessens wheel concentration; thus especially for high speed roads wider widths are preferred (AASHTO 2018), and the maximum common value of 3.6 m is taken here. Shoulders are the part of the roadway flush with the lanes and are needed on highways both in rural and in urban areas serving for refuge in case of emergencies, for bicyclists' use, for avoiding potential vehicle collisions, for reducing effects of precipitation (snow storage in winters and flow redirection in other seasons), for

structural support, and most importantly—to our purpose—for collecting stormwater away from the traveled way which not only protects the road users but road's longevity as well. In order to keep the costs low, the shoulder could be kept narrower than the lane width. Green Book suggests median shoulders to be sloped away from the traveled lane to avoid melting snow in winters for preventing freezing. In the local roadways, due to right of way issues, they are problematic. Under given road conditions, various rainfall intensities and four different roadway widths were used. A range of practical slopes are also tested, which provides insight into sufficient cross slope needed. In case of different considerations, the design code included below can easily be run to obtain the inlet spacing for adequate pavement drainage. the traffic.

Table 3.4 : Assumptions considered.

Property	Tested Conditions
Roadway roughness, channel flow, n	0.016
Roadway roughness, sheet flow, n	0.011 (equation (3.6))
Runoff coefficient, C	0.95
Lane width (m)	3.6
Shoulder width (m)	3 (highway) | 0 (local)
$T_{\max}$	Shoulder width (highway) | ½ lane width (local)
Maximum channel flow depth	10 cm (alternatively 3 cm)
Hydroplaning limit	4 mm (alternatively 2 mm)

Rainfall intensities may reach beyond 300 mm hr^{-1} for 25-year return period—while 10-year return period is picked in this study. For the along-road slope, Westlake (2011) recommends in the Institute of Civil Engineers (ICE) publication the range of 0.5% minimum and 4% maximum for divided highways (dual carriageway, as they refer) and 6% maximum for undivided, but since topography governs these, up to 10% is considered in this study.

Table 3.5 : Conditions inlet spacing is calculated for.

Property	Tested Conditions
Rainfall intensity (mm/hr)	25-300
# of lanes on the road	1, 2, 3, 4
Cross-slope (%)	1, 2, 3, 4, 5, 6, (and 7)
Along-road slope (%)	1, 2, 3, 4, 5, 6, 7, 8, 9, 10

3.2 Results

Maximum allowable inlet spacings are obtained based on various road designs and environmental factors. The range of longitudinal- and cross-slopes, S_L and S_x, are shown in the dual x-axes placed below and above the plot area, respectively as in Figure 3.3. Each subplot in below figures shows the results for a different number of lane up to four. Change in colors shows the change in rainfall intensity, I. The lines that cut short or disappear altogether despite showing in the legend attest to the topping of 4 mm maximum depth criterion for the over-lane flow (except in Figure 3.8 it is for 2 mm).

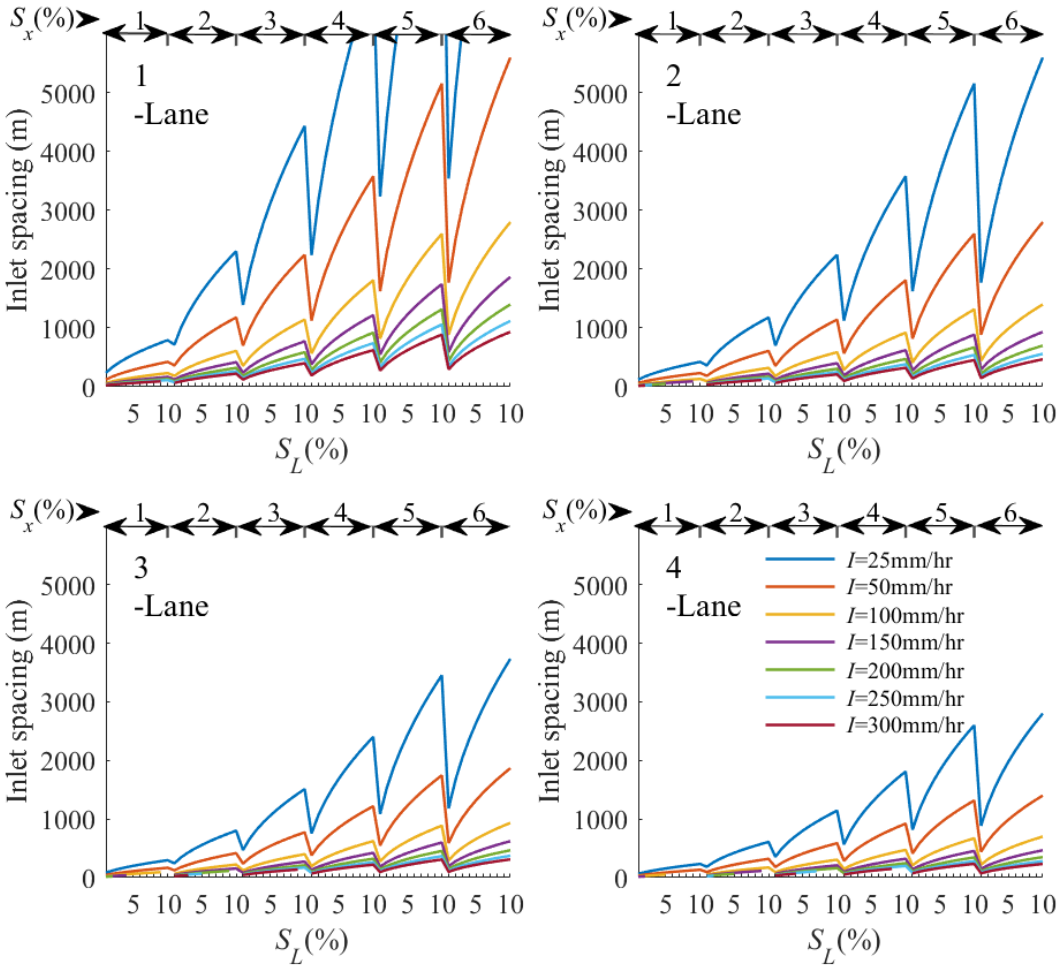

Figure 3.3 : Maximum allowable local road inlet spacings with zero offset between the curb and the lane.

For a single-lane pavement, as high as 293 mm hr^{-1} intensity works for all configurations tested, for the over-lane flow depth. When the lane number doubles,

the intensity halves and when triples, it falls to a third as they are inversely related (equation (3.1); Figure 3.7). This is consistent for a 3-m shoulder, and Table 3.6 shows the corresponding over-lane flow depths for a 4-lane pavement with the rainfall intensities from the figures. These values slightly differ from the earlier chapter because there the sheet flow n values are different from what is used here and the road lane widths are not the same.

Table 3.6 : Water depths (mm) for a 4-lane road with 1% cross slope for a wide range of along-road slope and rainfall intensities. Shoulder: 3 m.

Rainfall intensity, I (mm hr^{-1})	Longitudinal Slope, S_L (%)									
	1	2	3	4	5	6	7	8	9	10
25	1.2	1.3	1.5	1.6	1.7	1.8	1.9	2.0	2.0	2.1
50	1.8	2.0	2.2	2.4	2.6	2.7	2.9	3.0	3.1	3.2
75	2.2	2.6	2.9	3.1	3.3	3.5	3.6	3.8	3.9	4.1
100	2.7	3.1	3.4	3.7	3.9	4.1	4.3	4.5	4.7	4.8
125	3.1	3.5	3.9	4.2	4.5	4.7	5.0	5.2	5.3	5.5
150	3.4	3.9	4.3	4.7	5.0	5.3	5.5	5.7	6.0	6.1
200	3.7	4.3	4.8	5.2	5.5	5.8	6.1	6.3	6.5	6.7
250	4.1	4.6	5.2	5.6	6.0	6.3	6.6	6.8	7.1	7.3
300	4.3	5.0	5.5	6.0	6.4	6.7	7.0	7.3	7.6	7.8
350	4.6	5.3	5.9	6.4	6.8	7.2	7.5	7.8	8.1	8.3
400	4.9	5.6	6.2	6.8	7.2	7.6	7.9	8.3	8.6	8.8
450	5.2	5.9	6.6	7.1	7.6	8.0	8.4	8.7	9.0	9.3

With zero offset and a maximum spread of half the lane from the curb (as in local roads), the spacing limits are pushed further with the increasing on-grade and cross slopes (Figure 3.3). Spacing of kilometers seems applicable for various configurations. But for large intensities and lower slopes, the hydroplaning limit kicks in and starts eliminating certain configurations.

Figure 3.4 captures the orientations that result in shorter inlet spacings. The cross slope values range from 2 to 7 per cent in Figure 3.4 and one sees the increase in maximum spacing values with increase in cross slope slows at 6% and starts to drop at 7%, showing that the turning point is in between 5 and 6%.

The results with 3 cm flow depth restriction are provided in Figure 3.5. Limiting maximum channelized flow depth to 3 cm produces spacings much lower than in the above figures, showing how the channel flow governs spacing. For 3- and 4-lane roadways, higher cross slope values are needed to avoid hydroplaning risks at higher

on-grade slope terrains; however, quite oddly, increasing cross slope values forces to design shorter inlet spacings.

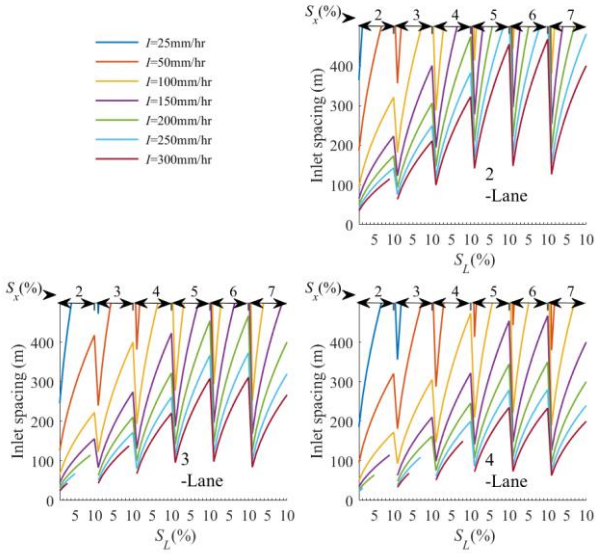

Figure 3.4 : Maximum allowable local road inlet spacings (S_x ranges from 2 to 7).

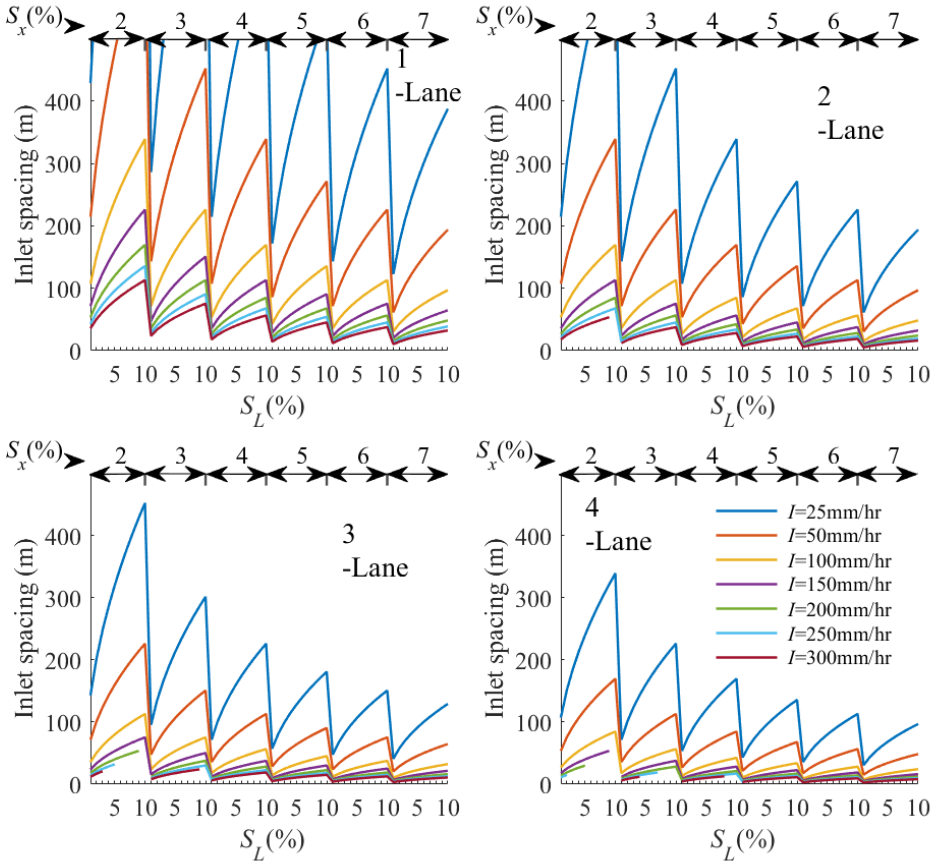

Figure 3.5 : Local inlet spacings with a maximum 3 cm flow depth at the curb (S_x ranges from 2 to 7).

With an addition of 3 m shoulder offset and zero encroachment onto the lanes from the channel flow, the trend observed in Figure 3.3 is observed in Figure 3.6, however, only up to 3% cross slope—and from there on, inlet spacing needs cutting back.

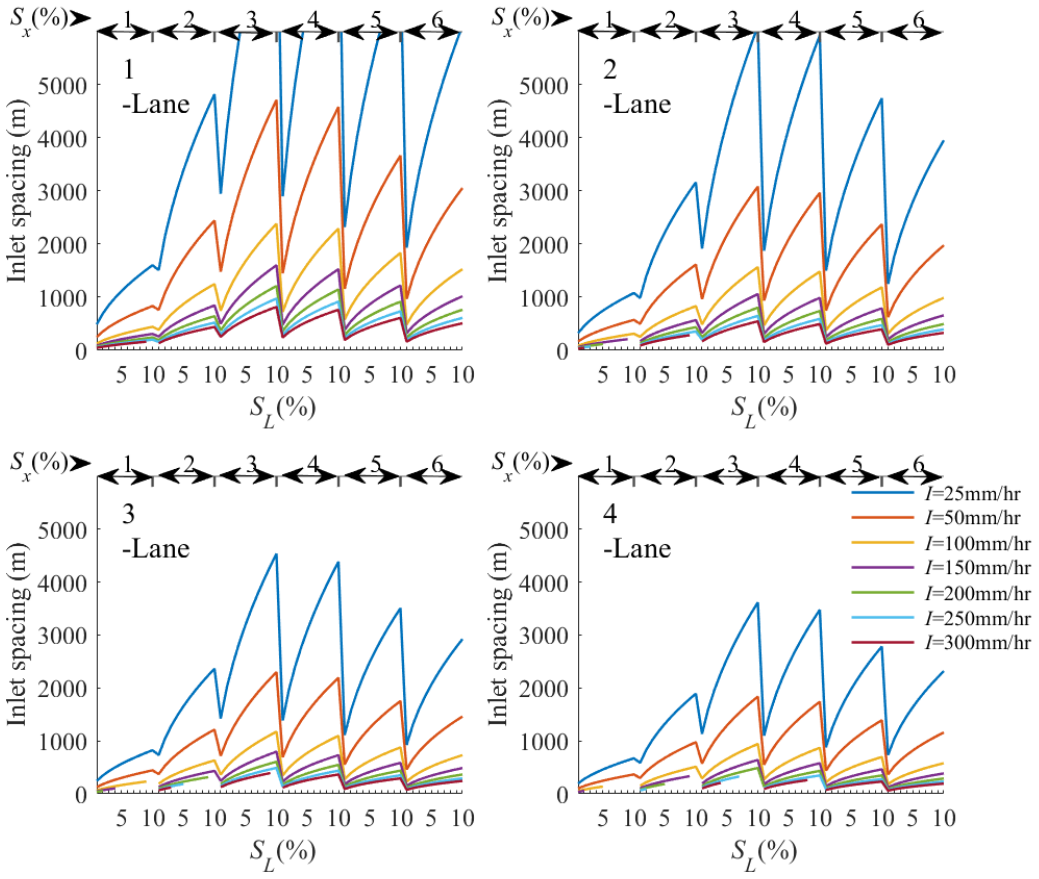

Figure 3.6 : Maximum allowable inlet spacings for highways with a 3 m shoulder ($z_r = 4$ mm).

The zoomed in version for the shouldered case is provided in Figure 3.7. When compared to the zero shoulder in Figure 3.4, hydroplaning becomes increasingly pressing, as the hydroplaning limits are checked half-a-lane beyond what is in Figure 3.4.

Some guidelines consider 2 mm to be the better option for preventing hydroplaning, and thus presented in Figure 3.8 is the outputs with a 2 mm limit to over-lane flow depth. For three- and four-lane cases, 2 mm restricts most configurations for intensities above 100 mm hr^{-1}.

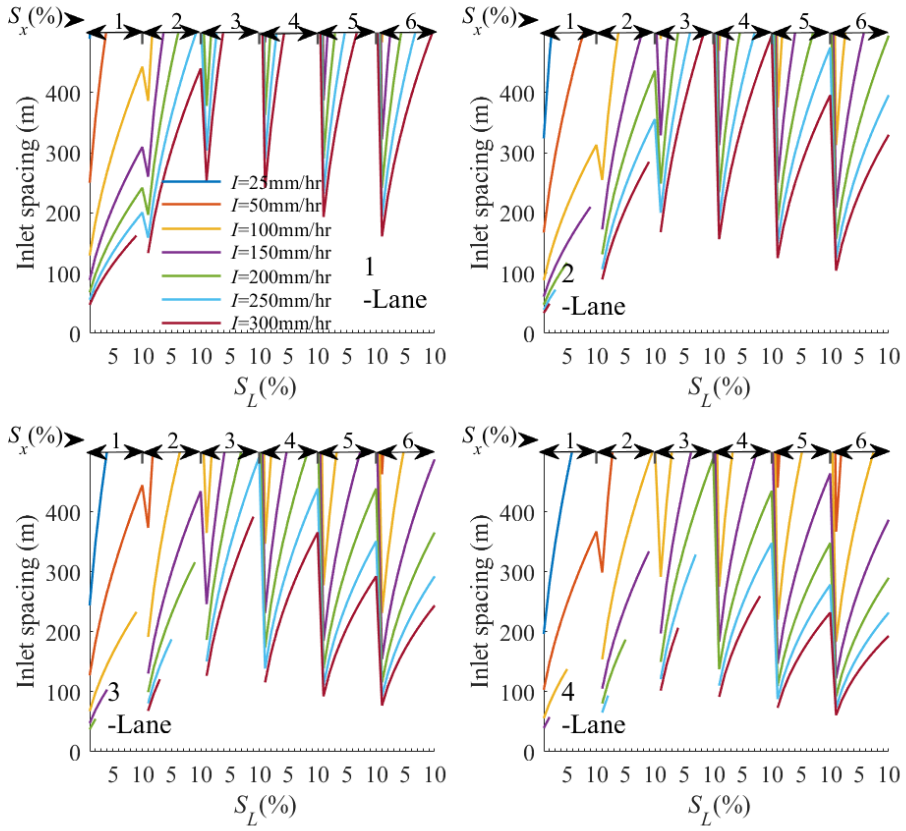

Figure 3.7 : Figure 3.6 zoomed in.

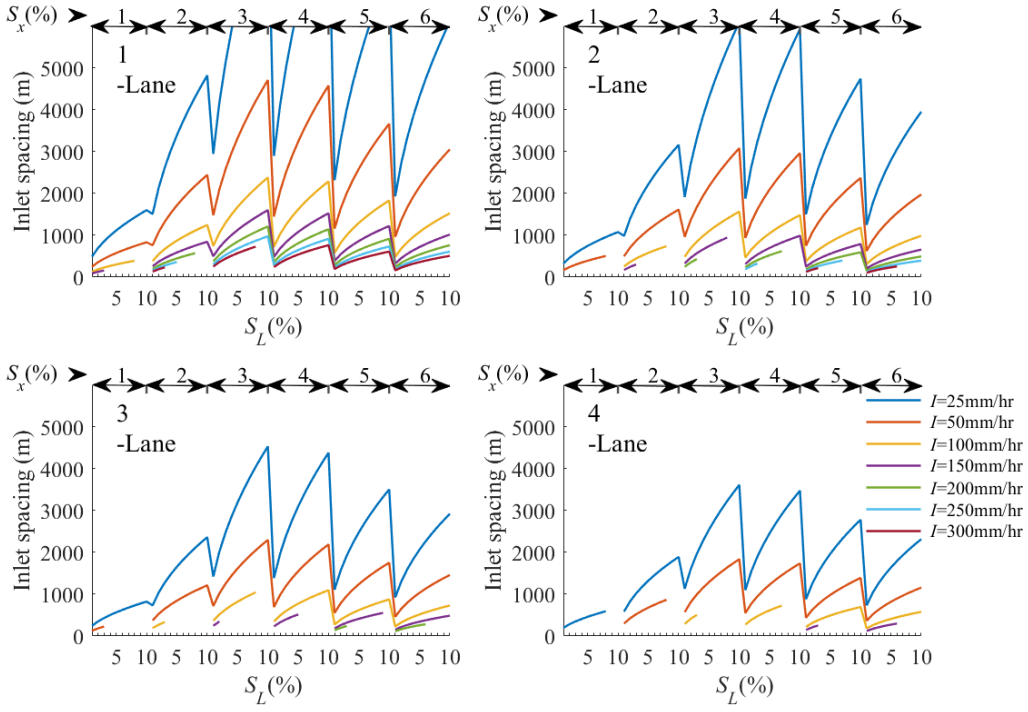

Figure 3.8 : Maximum allowable inlet spacings for highways with $z_r = 2$ mm.

Usually inlet spacing increases with increasing S_L while disappears altogether at certain cases especially for the higher values. Cross slope, S_x, increase, on the other hand, seemingly increases the inlet spacing up to 3%, beyond which the opposite trend takes place with a 3 m shoulder.

Despite lower water film thicknesses on road surface for higher cross slopes (below 4 mm), 3% marks the maximum inlet spacing possible under the considered conditions (Figure 3.6; Figure 3.8).

Figure 3.9 shows that the lower cross slopes hit the hydroplaning depths for lower rainfall intensities; that the maximum spacings are obtained with a 3% cross slope and shows the difference between $S_x = 3\%$ and $S_x = 4\%$, and how the design configurations are limited with the flow depth on road surface when 4 mm threshold is hit. This figure was produced for 3% on-road slope, which is what Westlake (2011) recommends for ideal design in an ICE publication.

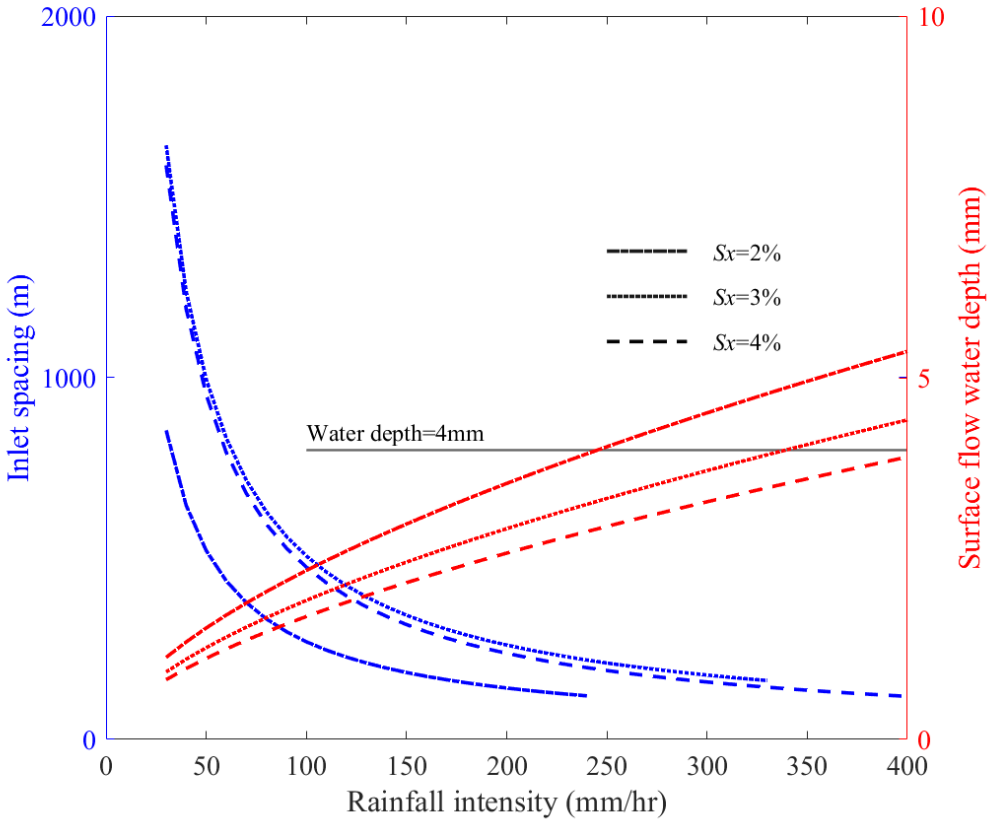

Figure 3.9 : Maximum allowable inlet spacing (in blue) and over-lane flow depths (in red) for a 3-lane road with 3% along-road slope ($S_L = 3\%$) and a 3 m shoulder.

3.3 Discussions and Conclusions

Maximum allowable inlet spacings are obtained for roadways of various purposes under different slope orientations and found that increasing longitudinal slopes increase maximum allowable spacing as a general trend as in Figures **0**-Figure **3.8**. For the cross slope variations, however, the relationship is not as clear at first sight and varies depending on the flow depth criteria. While one sees increasing spacing in Figure 3.3 with increasing cross slope, it changes the trend in Figure 3.4 as the slope increases, and Figure 3.5 shows the opposite of Figure 3.3**.** Figures Figure 3.6 and Figure 3.8 also produce results similar to Figure 3.4 in that maximum spacing increases first and then starts decreasing, and Figure 3.9 shows (in blue) neither extremum results the maximum spacing, in line with the former figures. When one looks into equation (3.5), an inverse correlation is captured between the spacing and cross slope, but this is not reflected in the figures except in Figure 3.5**.** To clarify what first seems as contradiction, equation (3.6) should be considered. When equation (3.6) is placed in equation (3.5), spacing changes with $S_x^{5/3}$ instead of S_x^{-1}. However, this still does not explain why the trend changes direction with increasing cross slopes. It is clear from equation (3.6) that cross slope multiplied by the maximum allowable spread produces the depth value, and in this study the general criteria is set to be 10 cm for the maximum flow depth, and when $S_x > z_{\max}/\mathrm{T}_{\max}$, flow depth is replaced by $z_{\max}$ per the criteria. In such a case (i.e. when $S_x > z_{\max}/\mathrm{T}_{\max}$), spacing changes with S_x^{-1} in equation (3.5), as the depth is constant and is not a function of cross slope. Figure 3.5, in which flow depth is limited to 3 cm (as opposed to 10 cm for the rest), clearly depicts the decreasing spacing trend (with increasing cross slope). Flow depth is limited to 3 cm with half a lane maximum spread (180 cm); that limit is reached for an S_x value below 2%. Thus, the depth is kept constant for S_x values 2% and above (to 7% in Figure 3.5). Figure 3.6 shows that (as the 10 cm is reached at 3.33% for a highway setting with 3 m shoulder) the increasing spacing trend plateaus around 3 and 4% cross slope and starts dropping from there on. For the half lane spread criteria (and 10 cm flow depth) 5.55% cross slope is the turning point beyond which inverse relationship sets in. Thus, to address cross slope impact on spacing, one needs to be aware of the

constraints for design; if designing for a fixed spread with no regards to flow depth, then spacing increases with increasing cross slopes (as it does up to 3% in Figure 3.6), but if flow depth becomes a limitation, than an inverse correlation governs the case (as it does beyond 3% in Figure 3.6). The results also show that while higher longitudinal slopes enable wider spacing of inlets, depending on the roadway width and cross slope values, the design may collapse altogether due to hydroplaning risks leaving no reason to analyze the spacing.

Izzard (1950) mentions inlet capacity controlling the inlet spacing along with gutter capacity; while this study looks into the impact slope orientation has on inlet spacing, it does not consider the impact of inlet capacity. The purpose here was to get an idea regarding the impact of slope orientation on the spacing, and the link between these results and inlet capacity is lacking. Untested also is how increasing the cross slope gradually from the crown line could impact the results, which is not favorable for highways but could be considered for local roads, which might help optimize designing wider roads without the hustle of hydroplaning, to an extent. Based on Gallaway et al. (1979) work, limiting water film thickness to 4 mm is reasonable to prevent hydroplaning risks. HEC-22, however, states that at 89 km hr^{-1}, hydroplaning occurs at 2 mm thickness. The drastic change between the 2 mm and 4 mm is obvious looking at Figure 3.6 and Figure 3.8, for a highway setting, which creates less tolerance to hydroplaning due to high speeds. If it is desired to keep the cross slope at or below 4% (as advised by HEC-22 pp 4-4&4-5), designing for multilane highways looks unpromising at environments with above 100 mm hr^{-1} rainfall. This sets a real challenge at steeper terrains.

HEC-22 recommends inlet spacing to be between 90 to 150 m. The results show, in certain cases, spacings can reach kilometers especially under low precipitation considerations. However, the interest in this study is on the limited conditions. Looking at Figure 3.7 (and knowing a cross slope of 1% is in fact unreasonably low and 1.5% is recommended as a practical minimum), apart from a few exceptions with low grades in 3- and 4-lane roadways, the allowable spacing values do not constitute any major restrictions in highway setting (3 m shoulder) in terms of recommended spacing. In the local setting, however, with a maximum spread of 1.8 m, maximum allowable inlet spacing becomes a limitation in many orientations (Figure 3.4) and

slope optimization becomes crucial. If the maximum flow depth is limited to 3 cm, then the limitations are inescapable and become a major restriction to spacing consideration (Figure 3.5), which is consistent with the conclusion of Wong and Moh (1997) who find exponential increase in inlet spacing with increasing maximum spacing. (This means decreasing maximum flow depths (as Gomez et al. 2002 practices) or the spread results in drastic variations.)

In conclusion, in terms of longitudinal slope, high grades promote higher inlet spacing, so its effect is obvious; however, over-lane flow depths reaching hydroplaning limits is the constrain in terms of roadway hydraulics. For cross slope orientation, however, maximum inlet spacings are obtained at z_{max}/T_{max}, and anything below and above that limit lowers the maximum spacing. Inlet spacing is an important criteria cost vise, and in this part, the goal was to lay the impact of slope orientation on spacing.

The results show that while steep cross slope values may be preferred to minimize ponding and confine the channel flow at bay within very small spreads, especially in regions of low longitudinal slopes (AASHTO), increased cross slopes also leads to rise in flow depths and thus limits the spacing. Plus, cross slopes may only be increased beyond certain values in warm climates because frost creates additional problems such as the tendency to drift towards the lower edge. AASHTO also recommends reasonable high cross slopes to prevent encroachment in curbed roads, as the flow travels within the roadway and not in gutter channels.

4. INTERCEPTING ROADWAY FLOW: CURB OPENING INLETS; FOUNDATIONAL MODELS, UNDERLYING ASSUMPTIONS, AND CRITIQUES FOR FLUSH INLETS

In removing roadway flow, vertical openings in the curbs called Curb Opening Inlets (COIs) practiced widely especially in Europe, North America, South Africa, and Australia (Figure 4.1). They are one of the most preferred flow capture structures for they 1) do not interfere with the road making it safe for all users including bicyclists and the disabled and 2) do not get clogged as easily compared to the structures installed along the road surface such as grate inlets or slutted inlets thus operating with the least maintenance any time. To improve capture capacity of these inlets for economical and functional purposes, different designs developed over the years. But even with the simplest type, modeling the flow analytically accurately is too complicated and thus, as in many other water structures, models need experimental verifications and modifications prior to any field adaptation, and that complicates the matters with the multitude of variations in design.

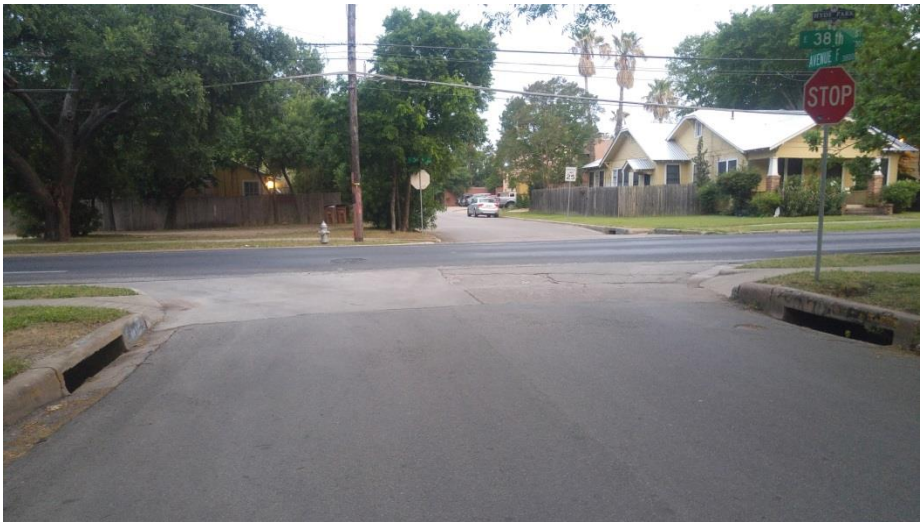

Figure 4.1 : Curb opening inlets located at both sides of the roadway right before a pedestrian pass in Austin, Texas.

One category to group variations in desing is whether the inlet is flush, recessed, or in between regarding inlet's position with respect to the road edge; here, only the designs with flush inlets are considered. Another grouping is whether the inlet is

depressed with regard to approach section or not. Izzard (1950), one of the earliest studies in the subject and the one underlying HEC-22 guidelines, works on these who mentions three sets of experimental data obtained in different geometries but uses only one set to modify his solutions to inlet without depression and inlet with a depression for the scarcity of widely ranged testing conditions from the other two. The hypothesis here is that the slow progress in establishing accurate guidelines for COI design stem from the lack of clearly identifying the differences between the COI types, lack of common terminology, and lack of experimental data. For this, we first introduce the solutions of Izzard (1950), then his work from our perspective followed by HEC's solutions, introduce Muhammad's updated equation to depressed inlets for HEC-22, introduce Soares (1991) and Uyumaz models for continuously depressed models (including Uyumaz's unpublished coefficients) as it is not covered exclusively in HEC-22, briefly mention Izzard (1977) work, and proceed with providing some of the misunderstandings from the literature that lead to misjudged conclusions and incorrect misrepresentations. In reality only the partial solutions of Izzard are based on theory and that his full capture models are not. It is shown that Izzard is the likely source for the equivalent cross slope concept HEC-22 uses to reduce all models down to undepressed model. This work shows that HEC-22 fails to include designs in which continuous and local depression co-exist. We suggest that HEC-22 be updated in line with Muhammad (2018)'s models and that of Soares (1991)' and Uyumaz (1992, 2002)'s.

4.1 Izzard 1950

Most theoretical investigations of COIs are based on weir analogy of various types (e.g. Uyumaz 1994; Zwamborn 1966) and the pioneering work of Izzard (1950) is no exception. His work is adjusted with then available experimental data as empirical adjustments are needed to obtain reasonable predictions. However, for any adjustments to make sense, the real problem at hand, analytical model, and the set-up experimental data is collected should all be as close to each other in design as possible. Only by then, the model fulfils the attempted goal, and because of this, all three components are looked into to do justice to the analyses.

Izzard (1950) possessed three different sets of experimental data. One set he describes "the flow intercepted was never greater than 50 percent," and another

"reported only a discharge Q_a at which the inlet barely intercepted the entire flow" (latter also had an unconventional section at the inlet). It is safe, after carefully reading the paper, to conclude that Izzard's solutions (for inlet without depression and inlet with a depression) at the core are based on the third set (referred to as Illinois data in Izzard (1950)). The crucial part is then to understand how Izzard obtained the empirical solutions to different designs using a single type of experimental data when the depressions are concerned; but prior to that, Izzard (1950)'s analytical and empirical models are presented.

4.1.1 Izzard (1950)'s models

Using linearly dropping head at the opening gives $h = a + y - \left(x / L_a\right) y$ where a is the inlet depression depth, y is the flow depth at the approach section, x is the distance from the upstream end of the opening, and L_a is the opening length for a fully captured flow with the inlet depressed a. This coupled with the critical flow assumption at the inlet lip, $V = \left[g\left(2/3h\right)\right]^{1/2}$, gives captured flow for dx to be $dQ = V\left(2/3h\right)dx$ using continuity, which when combined with the above expressions becomes:

$$Q = 0.218 g^{1/2} L_a y^{3/2} \left(\left(\frac{a}{y} + 1 \right)^{5/2} - \left(\frac{a}{y} + 1 - \frac{L}{L_a} \right)^{5/2} \right) \tag{4.1}$$

where g is the gravity acceleration. From here, Izzard first takes the *inlet without depression* case (i.e., $a = 0$ thus $L_a = L_0$) with an inlet length that captures the entire approach flow ($L = L_0$) and gets

$$Q_0 = 0.218 g^{1/2} L_0 y^{3/2} \tag{4.2}$$

for the theoretical full capture capacity Q_0. Izzard then sets up the ratio Q / Q_0 using equation (4.1) for $a = 0$ with installed inlet length $L \neq L_0$ and equation (4.2) to avoid working purely with the full capture measurements which yields:

$$\frac{Q}{Q_0} = 1 - \left(1 - \frac{L}{L_0} \right)^{5/2} . \tag{4.3}$$

Using partial capture measurement on a cross section as given in Figure 4.2 (where approach section has two cross slope values as a result of having a gutter at the curb side with a higher transverse slope and inlet has no additional depression in its vicinity), Izzard (1950) finds that if

$$L_0 = \frac{Q_0}{0.124 g^{1/2} y^{3/2}}, \tag{4.4}$$

then the experiments fit equation (4.3) well.

Izzard (1950) provides full capture solution for the *inlet with a depression* as:

$$\frac{Q_a}{L_a} = 0.124 g^{1/2} \left(a + y\right)^{3/2} \left[1 - \left(1 - \left(\frac{y}{a+y}\right)\right)^{5/2}\right]. \tag{4.5}$$

Using equation (4.3) to replace what is within square brackets in equation (4.5) (applying similarity between heads at the ends and inlet lengths) results in equation (4.6) for the full capture of inlets with a depression—Izzard tests this equation with the full capture data he had and concludes the results to be satisfactory.

$$\frac{Q_0}{L_a} = 0.124 g^{1/2} \left(a + y\right)^{3/2} \tag{4.6}$$

For the locally depressed inlet, Izzard (1950)'s solution for the partial capture is obtained via the use of equation (4.1) directly, and thus:

$$\frac{Q}{Q_a} = \frac{\left(\frac{a}{y} + 1\right)^{5/2} - \left(\frac{a}{y} + 1 - \frac{L}{L_a}\right)^{5/2}}{\left(\frac{a}{y} + 1\right)^{5/2} - \left(\frac{a}{y}\right)^{5/2}}. \tag{4.7}$$

4.1.2 Analysis of Izzard's solutions (1950): analytical basis, measurements he uses, measurements we use and more

4.1.2.1 Inlet without depression

Among the experimental data sets Izzard employs, the third set, known as Illinois study, has an unchanged section in the flow direction with distance, i.e. the approach section for flow and the section inlet opening is located are about the same (Izzard 1950, p.40). This clearly means no (extra) local depression at the inlet opening; however, the road adjacent to the curb (0.46 m (1.5-ft) wide) depressed evenly

continually (perhaps "raised road" describes it better, because (author infers) instead of *adding* $A2$ to $A1$, Izzard *excludes* $A3$ from the sum with an approach depth of y as in Figure 4.2). In other words, the data Izzard (1950) uses a *composite* geometry in section view with (to our understanding) a flow depth of y as in Figure 4.2. Based on partially intercepted data collected on that setup, Izzard proposes that if equation (4.2) is updated as equation (4.4), the partial solution for the inlet without depression in equation (4.3) provides accurate outputs and thus equation (4.4) is *perceived* to be Izzard's empirical equation to *undepressed* inlets for full capture.

The conclusion for the empirical full captures begs to be criticized for two main reasons: 1) full capture formula is not modified with 100% capture data; and 2) Izzard is not entirely clear with the terminology used. Latter is perhaps more striking, because not only does the final model in equation (4.4) diverge from the set-up for the analytical solution, but because Izzard deals with this case under the title "inlet without depression," most of his readers perceive this to be an unchanged cross section with a single cross slope—unlike the one shown in Figure 4.2. That is how undepressed inlets are understood in the literature that follows Izzard (1950): "inlet without depression" equals undepressed inlets on uniform sections. Thus, analytical model differs from the empirical in terms of design, and because users do not realize the difference due to the confusion in terminology, users' set-up for which the empirical model is used differ for a second time and in final it leads to chaos (partly due to lack of clarity with the terminology used). Below are detailed analyses to these.

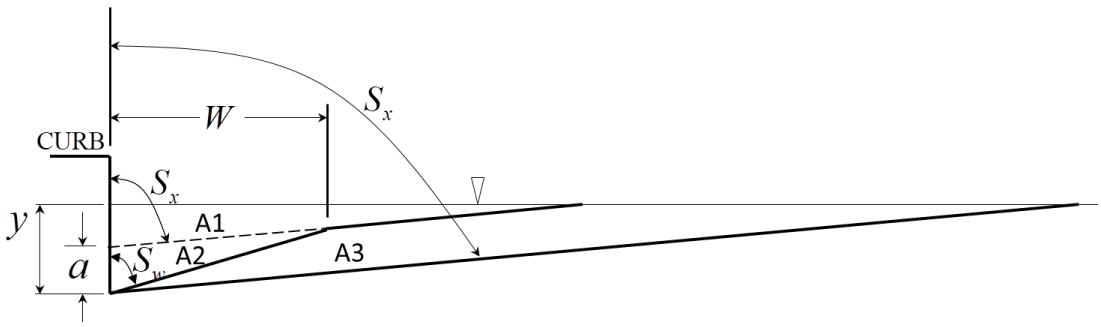

Figure 4.2 : Sketch of section view in Izzard's empirical model upstream of inlet without depression where $a = a_g$, the gutter depression.

In sync with Figure 4.2, Izzard (1950) needed inlet lengths that captured flow within $A1 + A2$ (i.e. Q_{A1+A2}) fully in order to use equation (4.3) safely as its accuracy

necessitates *correct*[3] prediction of L_0; however, using equation (4.2) to obtain L_0 did not work for Izzard because—besides any inaccuracies in the underlying assumptions in derivation—flow depth y corresponds to the depth of $Q_{A1+A2+A3}$ in the analytical solution. To overcome this, Izzard adjusts the multiplier 0.218 to 0.124 empirically in $L_0 = \frac{Q_{A1+A2}}{0.218 g^{1/2} y^{3/2}}$ to keep the ratio reasonable with Q_{A3} eliminated, and that gives equation (4.4). That was Izzard's work; for the rest of us, using depth y meant using $Q_{A1+A2+A3}$ for it was presented as the inlet without depression and understood as undepressed plane upstream of the inlet; but because Izzard was not explicit with the proposed 0.124, $\frac{Q_{A1+A2+A3}}{0.124 g^{1/2} y^{3/2}}$ resulted in astronomically high inlet length predictions in the case of a single cross slope.

Hammonds and Holley (1995) conducted full capture experiments on undepressed gutters with no inlet depression using various slope configurations. Comparison of length forecasts obtained from equation (4.2) and equation (4.4) based on measured flow depths in the approach section yields MAPE values in Table 4.1. It is clear that equation (4.2) performs much better than equation (4.4) does and that both models seem to predict better with increasing inlet lengths (i.e. flow depths) at least for up to 4.57 m. This drop in errors for equation (4.4) lines with the fact that with the increase in flow depth y, the flow within *depressed* part a (i.e. Q_{A2}) remains (almost) constant and the ratio $A3/A1$ decreases leading to smaller errors; in other words, the fact that the set-up equation and the measurements put in test are for different designs, the difference closes as flow depth increases. So what is expected is that lowest flow depths lead to higher errors which closes or almost closes as the flow depth reaches an optimum point and then the errors may reverse and start increasing depending on the performance of the analytical model.

Hammonds and Holley (1995) conducted partial capture experiments also and an analysis of Izzard's solutions and concluded that if equation (4.4) is used in obtaining full capture length for the approach flow, equation (4.3) works satisfactory. This leaves one perplexed knowing the adjustments were made for the composite slope

[3] Because equation (4.3) is not modified with empirical data, equation (4.4) tries to optimize any inaccuracies in equation (4.3) which is beyond correctly forecasting full capture lengths.

with the empirical fit which is binding for both equation (4.3) and equation (4.4). It seems as though, inserting equation (4.4)'s astronomically large predicted L_0 in equation (4.3) by luck balances the inlet capacity of a continuously depressed inlet for partial captures. This also means that if the full capture length information is known through experiments, equation (4.3) very likely does not work.

Table 4.1 : Mean absolute percentage errors (MAPE) for forecasted inlet lengths based on Equation (4.2) and Equation (4.4) Hammonds and Holley (1995) data prototype values).

	MAPE (%)	
Inlet length (m)	Equation (4.2)	Equation (4.4)
1.52	66.2	192.2
4.57	9.8	65.9
overall	38.0	129.1

But could one explain the 66% error or overall 38% (Table 4.1) in the analytical solution? Now that the approach flow for the theory and the experiments match, Izzard's assumptions may be put in trial. For undepressed inlets, Izzard assumed the head at the upstream end of the inlet is equal to the approach flow depth and at the downstream end zero for the full capture lengths, and that the variation in head is in direct proportion along the inlet length. Schalla et al. (2017) and Muhammad (2018) investigated these in some of their figures as in Figure 4.3 where Zwamborn (1966)'s measurements were used (all in black). Notice however that in the derivation equations above, Izzard uses $2/3h$ as he assumes critical flow at the lip, so the red dashed line is added in this study to show a more accurate comparison in Figure 4.3. (Additionally, approach flow should be the basis for comparison with Izzard's assumption, so keeping both the inlet length and the flow depth same is not as accurate—reason the red line ends at 19.51 ft instead of 20 ft). Comparison of the assumed flow profile along the inlet length in red with the measurements in solid line should be about the difference one observes in Table 4.1 and that an accurate empirical adjustment is necessary despite changes with road design.

In conclusion, for "inlets without depression:" 1) full capture lengths in equation (4.4) did not work for the users because inlet without depression is perceived to be a uniform section while the case presented by Izzard is for continuously depressed (thus theory and experiments diverge); 2) surprisingly, use of equation (4.4) in

equation (4.3) results reasonable predictions of equation (4.3) (Hammonds and Holley 1995) despite design difference; 3) for undepressed full capture inlet lengths, equation (4.2) is a better approximation on uniform slopes when compared to equation (4.4); and 4) for equation (4.4) to be used in compound gutters, further testing is needed because it was adjusted with partial tests instead of full.

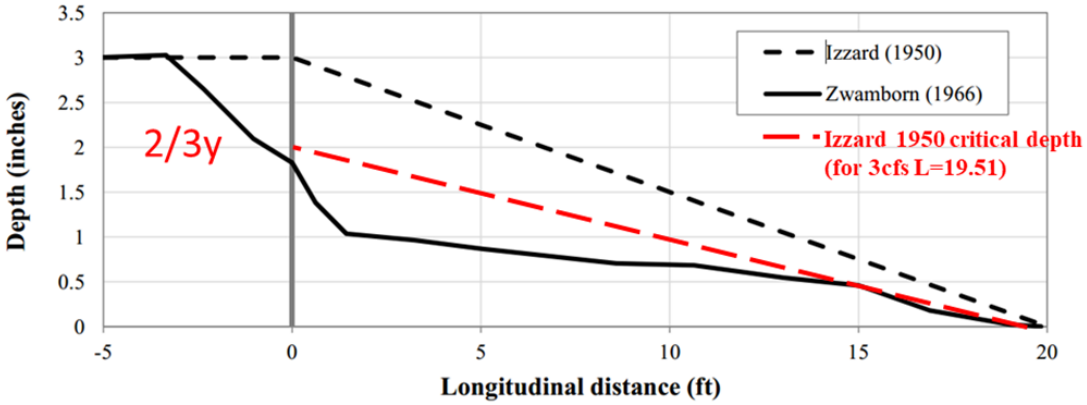

Figure 4.3 : Observed water profile by Zwamborn (1966) is compared to Izzard's critical depth and linear drop assumption (in red) (black dash lines presented by Muhammad 2018 for the normal depth mispresents Izzard's assumption). Modified from Muhammad 2018's Figure 3.1 (red part added).

4.1.2.2 Inlet with a depression

For the depressed inlets, Izzard's assumptions vary from the undepressed in that depression depth is added to the flow depth at the both ends of the inlet so that the head drops from $y+a$ to a along the inlet opening for full capture, where y is the approach flow depth at the uniform section upstream. Rest of the assumptions remains the same in derivation of equation (4.1). Muhammad investigates Izzard's assumptions for depressed inlets also, via obtaining measured and observed flow depths along the lip; unfortunately approach (i.e. normal) flow depths (or the flow amounts) are not reported for an accurate comparison with Izzard assumptions—Muhammad compares measurements to the flow depth at the start of opening (Figure 4.4), but that does not necessarily line with Izzard's assumptions where the expected flow depths vary from $2/3(y+a)$ to $2/3a$ for full capture. However Figure 4.4 is very valuable in that it invalidates the linear head drop assumption close to both ends regardless of where the assumed profile is placed, at least for the road design documented.

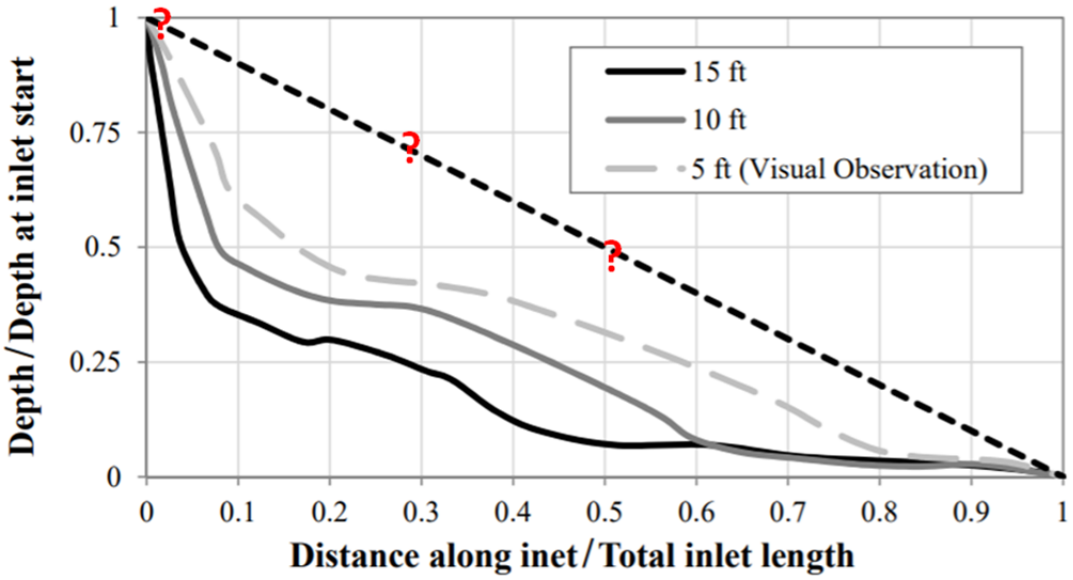

Figure 4.4 : Muhammad's investigation of Izzard's assumptions for the depressed inlets with his experiments along the inlet length. Modified from Muhammad (2018).

The analytical model for full capture model yields from equation (4.1):

$$\frac{Q_a}{L_a} = 0.218 g^{1/2} y^{3/2} \left(\left(\frac{a}{y} + 1 \right)^{5/2} - \left(\frac{a}{y} \right)^{5/2} \right). \tag{4.8}$$

Figure 4.5 shows the forecasts for equation (4.8) (i.e. equation (4.1) when $L = L_a$) and equation (4.6) (empirical) using Hammonds and Holley (1995)'s full capture data on depressed inlets. Hammonds and Holley (1995) (with no explanation) presents Izzard's full capture solution to be $\frac{Q_a}{L_a} = 0.124 g^{1/2} y^{3/2} \left(\left(\frac{a}{y} + 1 \right)^{5/2} - \left(\frac{a}{y} \right)^{5/2} \right)$ which is neither of Izzard's solutions, yet it was included in Figure 4.5 to show that Hammonds and Holley (1995)'s analyses of Izzard deviate from the original models. The analytical forecasts to divert from measured were expected based on Figure 4.4, but is the better predictions of empirical solution also coincidence as in the case of the partial for the undepressed? Additionally, does equation (4.6) align with Izzard's theory and perhaps more importantly, how could it be justified with a coefficient of 0.124 that is obtained for the undepressed inlets, i.e. equation (4.4)? Clearly something different went into this model.

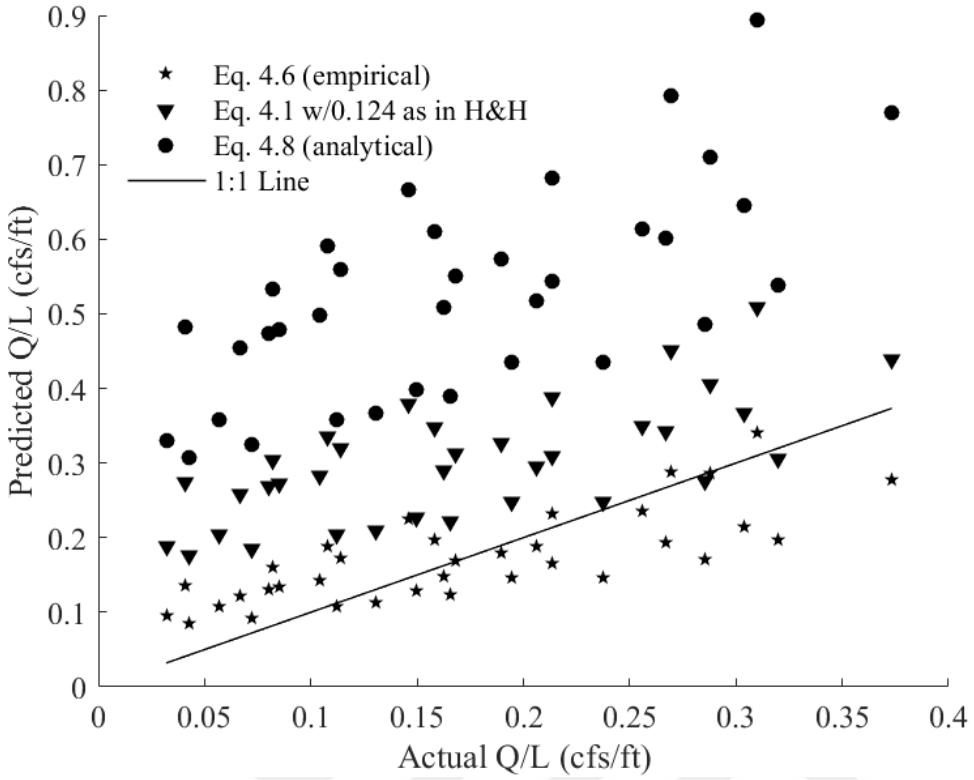

Figure 4.5 : Various solutions vs. Hammonds and Holley (1995)'s locally depressed experimental data with full interception (actual results are Q_a / L_a as measured and *H&H* in figure refers to Hammonds and Holley (1995) study).

Unlike equations (4.2) and (4.4), equations (4.8) and (4.6)—equations for the analytical and empirical depressed full capture inlets, respectively—do not look alike, so equation (4.6) is traced back to understand whether and/or how they are for the same design. Equation (4.1) for $a = 0$ (by definition, L_a the full capture length becomes L_0 for having zero depression) results in:

$$Q = 0.218 g^{1/2} L_0 y^{3/2} \left[1 - \left(1 - \left(\frac{L}{L_0} \right) \right)^{5/2} \right] \tag{4.9}$$

If one considers y to be a fixed depth and wants to increase the approach flow such that the approach depth is $a + y$ as in Figure 4.6, the equation becomes:

$$Q = 0.218 g^{1/2} L_0 \left(y + a \right)^{3/2} \left[1 - \left(1 - \left(\frac{L}{L_0} \right) \right)^{5/2} \right] \tag{4.10}$$

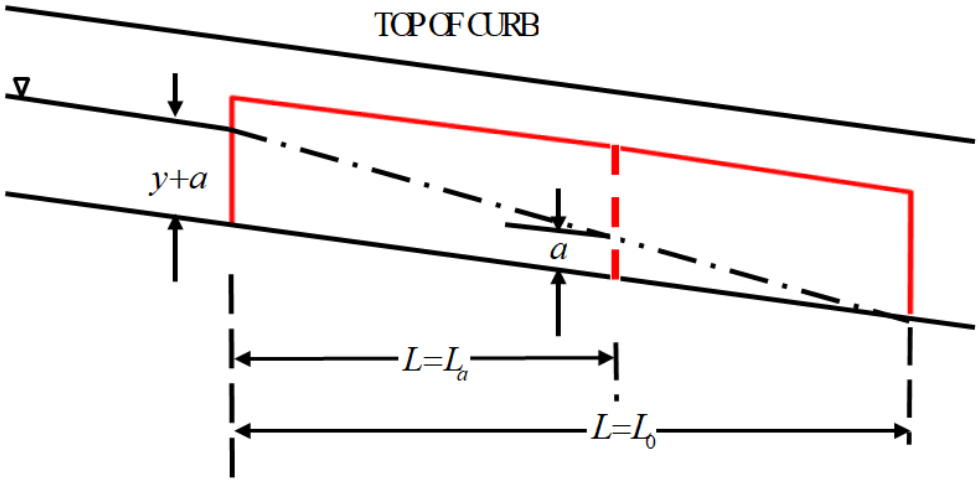

Figure 4.6 : Example figure. Tracing the origin of equation (4.6) for undepressed inlets, modified from Izzard (1950).

Note that inlet depression in equation (4.10) is still zero ($a = 0$), but the approach flow depth is $y + a$ which is independent of whether the inlet has a depression or not. If one now considers that the inlet length is $L = L_a$ and the captured flow is $Q = Q_a$ (just because head drops down to a) then:

$$\frac{Q_a}{L_0} = 0.218 g^{1/2} \left(y + a\right)^{3/2} \left[1 - \left(1 - \frac{L_a}{L_0}\right)^{5/2}\right] \tag{4.11}$$

It is known from equation (4.3) which is also obtained for undepressed inlets that what is within brackets in equation (4.11) is Q_a / Q_0 and inputing that into equation (4.11) (as Izzard does) leads to:

$$\frac{Q_0}{L_0} = 0.218 g^{1/2} \left(y + a\right)^{3/2} \tag{4.12}$$

Equation (4.12) is exactly the same as equation (4.2) but for an approach depth of $a + y$ instead of y, yet that is not what concerns us. Focus here is equation (4.6) which is almost the same as equation (4.12) except for two differences. First difference is the coefficient. equation (4.12) is solely analytical for the approach flow with a uniform cross section, and it is known based on the analyses of Izzard's undepressed inlet solution that if the gutter has a continuous depression that is included in the approach flow depth, then 0.218 is replaced by 0.124, so doing this

turns the approach gutter design into a *compound* section in such a way that y in Figure 4.2 represents the flow depth for $A1$ excluding a whose value is equal to inlet depression (of the attempted case) while its design is continuous gutter under the assumptions made above; thus the approach depth is now $a+y$, and that is exactly what is in equation (4.6). Second difference is that equation (4.6) has the full capture length L_a whereas equation (4.12) has it as L_0; and the author thinks this is a mistake Izzard made that needs correcting for the solution to be accurate analytically because this change is unacceptable theoretically and has no justification as far as one can see.

Based on above analysis, it can safely be concluded that Izzard (1950) used, in his analysis of depressed inlets for full capture, the available solution for the continuously depressed gutters with no inlet depression as covered under *inlet without depression*. This was made possible by fullfiling the only difference in the assumptions of depressed and undepressed inlets in Izzard (1950), hydraulic heads between the two ends of the inlet. Thus, an undepressed inlet with length L_a and approach flow depth $a+y$ as in Figure 4.6 satisfies the inlet end conditions for the depressed inlets (without the existence of inlet depression), as head drops from $y+a$ to a. In other words, Izzard, without wording it, hypothesizes that the inlet capacity for unit inlet length of a depressed inlet with uniform approach section that has flow depth y and inlet depression a is equal to the inlet capacity for unit length of an undepressed inlet with compound approach section that has approach flow depth of $y+a$:

$$\left(Q_a / L_a\right)_y = \left(Q_0 / L_0\right)_{y+a} \tag{4.13}$$

This needs testing against experiments for determining its performance because while Izzard concludes that the model works properly, he had limited data with an unconventional cross section, so it is hard to know what goes into his analysis of equation (4.6). (At this point, the path to finding inlet length for known flow Q_a is: 1) dividing equation (4.11) by equation (4.12) and replacing L_a / L_0 with $a/(a+y)$ to find $Q_0 = Q_a / (1-(1-y/(a+y))^{5/2})$; 2) using equation (4.12) to calculate L_0 (with coefficient 0.124); and 3) using $L_a / L_0 = a/(a+y)$ to obtain L_a.) Additionally,

as much as the assumptions are fulfilled for the depressed inlets and the right hand side of equation (4.6) is obtained *analytically*, if Izzard were to mean Q_0 / L_a in equation (4.6) (in place of Q_0 / L_0 or Q_a / L_a), the difference between the two sides of equation (4.6) introduces an error for missing the multiplier L_0 / L_a (or $(a+y)/a$) on the right hand side whose value is greater than unity. Figure 4.7 shows the difference. If L_a is known experimentally, which is very unlikely, the difference drops (Figure 4.7). On the flip side, for any user applying this solution to a depressed inlet with uniform approach section, all the flow there is Q_a and not Q_0; thus, if the user is trying to obtain L_a and they are not careful with the notations, then they probably make the mistake and insert lower values for the approach flow than Q_0 (i.e. they use Q_a as I did do in Figure 4.5), so in a way just like right handside that is missing multiplier over unity, left hand side is also not being accurate; whether they balance in practice is pending for equation (4.13) to be validated.

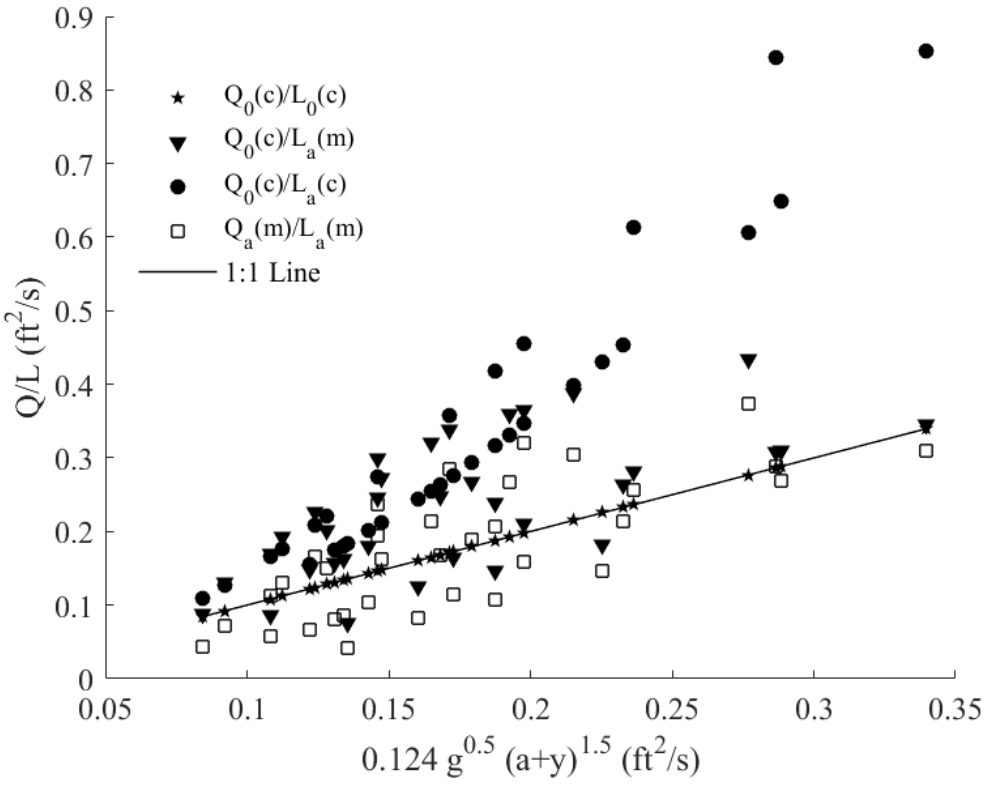

Figure 4.7 : Right hand side of equation (4.6) vs. left hand side of equation (4.6) using Hammonds and Holley (1995)'s depressed full capture data which has no information on L_0 and Q_0.

$0.124g^{1/2}(a+y)^{1.5}$ compares reasonably well with the ratio of measured Q_a and L_a (Figure 4.7; x-axis also equals Q_0 / L_0), and this shows that $\left(Q_0 / L_0\right)_{y+a}$ compares reasonably well with $\left(Q_a / L_a\right)_y$ making Izzard's hypothesis in equation (4.13) reasonable; as a result, using Q_a in place of Q_0 in equation (4.6) and using Izzard's completely unacceptable replacement of L_0 with L_a in practice works.

For the partial capture case, Izzard remains within the design of depressed inlets (unlike the full capture case) and obtains the partial formula using the ratio of equation (4.1) to equation (4.8) that results in equation (4.7), which is purely analytical, except the full capture length used in the partial solution (as detailed) is based on the undepressed empirical model; Figure 4.8 shows the results obtained using equation (4.13) to forecast partially captured flow rates for the depressed inlets. The outputs at zero in Figure 4.8 show that Izzard (1950)'s analytical full captures underestimate the inlet lengths so much that $L / L_a > a / y + 1$. Izzard's empirical equation leads to the best outcomes both in Figure 4.5 and in Figure 4.8. Obtaining partial flows best with the solution that provides best full capture forecasts an indication that equation (4.7) is a reasonable solution in general (unlike (4.3)).

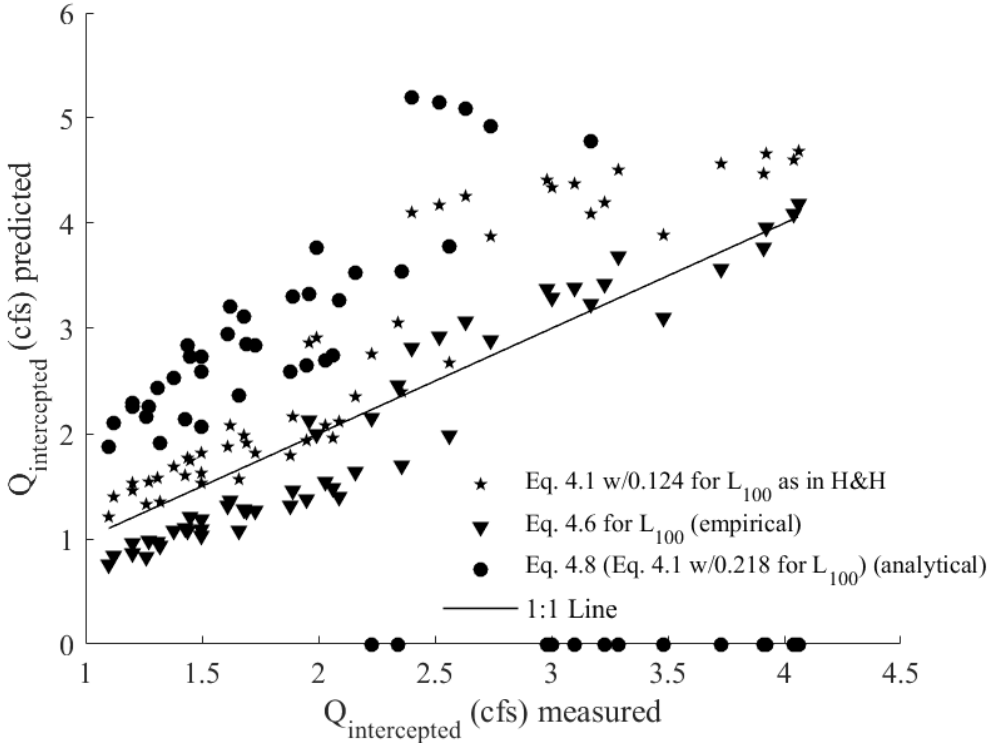

Figure 4.8 : Equation (4.7) for partial flows with full capture models in Figure 4.3.

To summarize, Izzard (1950) starts with the broad-crested weir analogy and obtains the general solution from which the partial models are obtained with no empirical modification for both "*inlet without depression*" and "*inlet with a depression*." Full capture models, which are used in the partial models also, however, divert from the theory. (Most likely) because of the lack of fitting-data for these models without *gutter depression*, Izzard approaches the solutions a bit differently that places the solutions completely off from the theory and eventually leads him to state that a "more complicated expression" could have been obtained was it not for the lack of experimental data (As a matter of fact, when experimental data from Karaki and Haynie (1961) becomes available, Izzard again returns to the subject to update the models (Izzard 1977)). Izzard's *inlet without a depression* model fails in practice, and should rightly so, and the misleading part stems from the (mis)interpretation that the *inlet without depression* means uniform section at the approach gutter as at the opening while it only means no special depression at the vicinity of the inlet. Depressed full capture model is interesting in that, Izzard solves it using similarity from the undepressed by fulfilling the hydraulic head conditions at both ends of the inlet for the depressed. While partial solutions are obtained based on analytical solution, full capture inlet length parameter within those solutions are not obtained based on theory, yet, very surprisingly, without any analytical or experimental consistency, Izzard (1950)'s partial solutions for both cases (as well as the full capture solution for the (locally) depressed case) provide close approximations to experiments when compared to his analytical solutions.

4.2 Hydraulic Engineering Circular No-22 (HEC-22)

Interestingly, Izzard 1950 only provides the section sketches to experiments he uses and stresses the importance of cross section in general, but not even once does he comment on the section he obtains the analytical solutions for. As a result, perceiving Izzard does in fact solve the models for a uniform cross section of a triangular shape is readers' assumption, not of Izzard's. Nonetheless, since the problem at hand needs be solved for the uniform slope (at least the simplest to *start* with), it is not improper to lay the groundwork on that basis and make empirical calibrations to the models accordingly. Benefiting from Izzard's 1946 work, where Izzard uses Manning's equation in a local sense to obtain approach flow (with use of continuity equation),

flow depth y comes into play both from velocity of the flow and the area of the cross section (top width y/S_x) (for details see Izzard 1946 or Muhammad 2018 pp.12-13). And from that modified Manning's equation for Q_a, HEC-22 takes $y = f\left(n, Q, S_x, S_L\right)$ and replaces the one in equation (4.2) (Muhammad 2018). Thus using equation (4.2) not directly as a function of flow depth but a function of its parameters such as S_x and S_L (and other parameters) gives HEC-22 the freedom to approach solving various geometries more systematically than Izzard 1950 does. This is commendable at this point as it is advancement to Izzard 1950 models because HEC-22 defines an equivalent slope S_e which is the slope that represents the section view when it is not uniform. One of the drawbacks of this is that HEC-22 did not consider updating anything equivalent to the longitudinal slope S_L as it also changes when transitioning to the depressed inlets—this was later included in Muhammad (2018)'s solution. Thus, it is clear that HEC-22 uses a (seemingly) different approach to using cross slopes and includes them directly within the solution unlike Izzard (1950), to whom flow depth y is sufficient in representing the changes in the section view but each design required different models.

For the full capture of flow (either local or continuous depression exists or neither does), HEC-22 employs the equivalent slope approach as:

$$L_T = K_u Q_a^{0.42} S_L^{\ 0.3} \left(\frac{1}{nS_e}\right)^{0.6} \tag{4.14}$$

where K_u is 0.817 for SI and 0.6 for US, S_L is on-road slope, n is the Manning's roughness, and S_e is the equivalent (cross) slope that changes based on the gutter and inlet design (Table 4.2). To calculate S_e, the amount of flow on the pavement surface and that on the gutter is considered: $S_e\left(\frac{Q_a}{Q_a}\right) = S_x\left(\frac{Q_s}{Q_a}\right) + S_w\left(\frac{Q_w}{Q_a}\right) = S_x\left(\frac{Q_s}{Q_a}\right) + \left(S_x + \frac{a}{W}\right)\left(\frac{Q_w}{Q_a}\right) = S_x + \frac{a}{W}E_0$ where a is the gutter depression, W is the gutter width, and $Q_a = Q_w + Q_s$, total approach flow (Figure 4.9). Table 4.2 shows three cases covered for different geometries in obtaining E_0 (which equals Q_w/Q_a); one case that is not covered explicitly however

in HEC-22 is when local depression exists on a composite cross section, i.e. when the two co-exist. Based on the examples HEC-22 provides, they seem to endorse using E_0 for the composite gutter for such a case.

Table 4.2 : E_0 for various COI designs for obtaining equivalent slope $\left(S_e = S_x + a/WE_0\right)$. S_w is the gutter depression.

Case:	Composite gutter ($S_w \neq S_x$ for the approach flow)	Local depression only ($S_w = S_x$ for the approach flow only)	Undepressed $\left(S_w = S_x \,\&\, a_{local} = 0\right)$	Composite &Local depression ($S_w \neq S_x$ &$a_{local} \neq 0$)
E_0:	$1/\left\{1+\frac{S_w/S_x}{\left[1+\frac{S_w/S_x}{\frac{T}{W}-1}\right]^{8/3}-1}\right\}$	$1-\left(1-\frac{W}{T}\right)^{8/3}$	Not needed ($S_e = S_x + \frac{a}{W}E_0 = S_x + 0$ thus $S_e = S_x$)	Unclear

Unlike Izzard, who calibrated partial solution via adjusting the full capture equation to an unrealistic form, HEC-22 changes the exponent in equation (4.3) as

$$\frac{Q}{Q_a} = 1-\left(1-\frac{L}{L_T}\right)^{1.8} \tag{4.15}$$

where L_T is the full capture length obtained from equation (4.14). Muhammad 2018 shows that equation (4.15) is the empirical form of equation (4.3), which should be no surprise when accurate full capture lengths are used since equation (4.3) only worked if the (incorrect) estimation of inlet length from equation (4.4) was used for uniform sections. So instead of adjusting the full capture lengths to some unreasonable values, HEC-22 adjusts the exponent in the partial solution. In other words, seemingly large difference between 1.8 and 2.5 shows about the difference in the calculated L_T and L_0 between two approaches. It should be more complicated of a case to obtain partial captures with the equivalent slope assumption when it comes to the depressed gutter and/or depressed inlet, but surprisingly, HEC-22 seems to endorse the same idea for the partial solution also, ignoring the theoretical basis

for the depressed inlets in Izzard (1950). Thus it is almost liberating to know that full capture and partial capture models do not differ based on road or inlet geometry as long as the equivalent slope is obtained accurately. Until it is confirmed otherwise, this concept should only be confined to flush inlets.

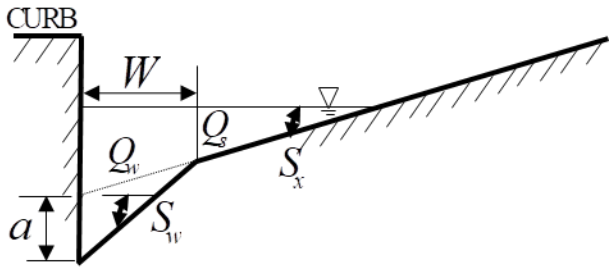

Figure 4.9 : Example figure. Equivalent slope parameters for HEC-22.

Based on these, it is safe to conclude that by the use of equivalent slope, HEC-22 treats all cases as having a uniform (equivalent) cross slope and uses Izzard's analytical solutions to inlet without a depression for full capture as well as for the partial. This is perhaps not very far from Izzard's use of equation (4.4) where he combines undepressed solution with the depressed gutter experiments, or the fact that Izzard's full capture for the depressed inlets also in its soul employs the undepressed solution via manipulating the head values in inlet ends. HEC-22 does not adopt any of Izzard's empirical considerations, and unlike Izzard, HEC-22 modifies the exponent in the partial solution and obtains the empirical fit accordingly. In addition to that, with use of flow depth parameters in place of flow depth in equation (4.2), HEC-22 solution should have the capacity to handle subtle changes in various parameters. Having said this, Muhammad (2018) finds that equivalent slope assumption has limits; if the local depression is too large, then the length of transition becomes very important as shorter lengths delay the spread to narrow with a flat surface.

One case that is not covered explicitly in HEC-22 is the design where both continuous (gutter) and local (inlet) depression exist; luckily, studies from Clemson University and Istanbul Technical University cover this case, which are included below, but prior to them, a couple of misunderstandings on HEC-22 mentioned first.

As earlier stated, the confusion based on terminology is not uncommon and requires care for accuracy. An example to this is from Schalla et al. (2017) who state "the local depression was set to zero, and the gutter depression was set to 3 in" in their

analyses, however, the opposite was the set-up of their experiments thus the results obtained as HEC-22 outputs are (very likely) different than what HEC-22 with the correct design would predict and so HEC-22 results and their experiments are of different designs—whether this causes large differences and impact conclusions is unclear and requires redoing analyses. Same situation occurs in the study of Comport et al. (2009) where the authors aim to find a design model for the specific curb inlet used by Colorado Department of Transportation (CDOT). The inlet used, based on the construction drawings and photographs provided, has a local depression with uniform approach flow section; however, the calculations are done using composite gutter; the difference is clear in Table 4.2 for the design equations. Thus, just as in Schalla et al. study, Comport et al. (2009) also does the analyses for a different design and their new model they propose is also based on that incorrect E_0 calculation. Whether they correct this in Comport and Thornton (2012) study is unclear as they write: "The reader is referred to HEC 22 for calculation of additional parameters," which include E_0. Another interesting example from Comport et al. (2009) is that they introduce the Uyumaz (2002) study as one with local depression, only, which is not the case. The mix of compound gutter with local depression also appears in Muhammad (2018) who explains "derivation of gutter conveyance" stating composite gutter is such that the "edge of the gutter *at the curb inlet* is depressed" while in reality it has nothing to do with the inlet itself.

Use of *gutter* is another problem that outcrops frequently as some studies differentiate between gutter and pavement while others use it as the combination of the two or as synonyms. These are examples to vague use of terminology that easily lead to incorrect analyses and/or conclusions as in the case of Izzard 1950 for *inlet without depression*, so the reader should always be careful. When it is traced back, the source for some confusions seems to be HEC (if one disregards the subtle language of Izzard), for HEC uses the exact same symbols for both the inlet- and the (continuous) gutter-depression, but it is just hard to pinpoint the source for the mistakes made in Hammonds and Holley (1995) in which Izzard's models are misrepresented.

4.3 Bowman 1988 and Soares 1991

While some attempts are available (e.g. Izzard 1977), the case where both the gutter and the inlet are depressed was not studied independently until around 1990s. First two studies are from Clemson University in South Carolina in the US (Bowman 1988; Soares 1991) to our knowledge followed by studies from Istanbul Technical University in Turkey (Uyumaz 1992; Uyumaz 2002). Bowman complies with the guidelines of South Carolina Department of Highways and Public Transportation and conducts experiments based on those geometries for the gutter. Soares later provides the empirical equation to inlet efficiency based on Bowman's data as:

$$\frac{Q}{Q_a} = \left(\frac{KL}{T} \left(\ln\left(0.1 S_L\right) \right)^2 \right)^{0.8} \tag{4.16}$$

where K is a function of inlet length L and cross slope of the road S_{x1} as given in Table 4.3 which could be interpolated for different values, and T is the spread for the approach flow and could be obtained (on a composite section) from $Q = \left(\frac{1}{n} S_L^{1/2} \right) \frac{3}{8} \left(\frac{\left(W S_{x2} + \left(T - W \right) S_{x1} \right)^{8/3}}{S_{x2}} + \left(\left(T - W \right) S_{x1} \right)^{8/3} \left(\frac{1}{S_{x1}} - \frac{1}{S_{x2}} \right) \right)$. Experimental model details of equation (4.16) are listed in Table 4.4.

Table 4.3 : *K* values for equation (4.16), adapted from Soares (1991).

Cross slope S_{x1}	Inlet length L (m)			
	1.22	2.44	3.66	4.88
0.021	0.037	0.021	0.016	0.013
0.028	0.031	0.019	0.014	0.011
0.042	0.027	0.016	0.012	0.01
0.063	0.018	0.012	0.01	0.008

4.4 Uyumaz Models

Uyumaz conducts vast amount of experiments and based on dimensional analysis proposes:

$$Q / Q_a = \begin{cases} A\left(\frac{L}{FT} \right) & \text{for } Q / Q_a < 0.6 \\ B\left(\frac{L}{FT} \right)^2 + C\left(\frac{L}{FT} \right) + D & \text{for } Q / Q_a \geq 0.6 \end{cases} \tag{4.17}$$

where $A, B, C,$ and D are the empirical coefficients that change based on slope orientation and inlet length (Table A.1); L is the installed inlet length; T is the top width of the approach flow; and F is the Froude number at distance W from the curb, $F = \frac{V}{\sqrt{gy}}$ where V is the average flow velocity for the approach flow obtained from continuity equation $V = Q/A$, g is the gravity acceleration, and y is flow depth $y = A/T$. The area, A, is needed at W to find V and y in Froude number, and it is obtained using road cross slope S_{x1} and the experimentally measured T (area: $A = T^2 S_{x1}/2$).

Table 4.4 : Gutter information for the experimental models (prototype).

	Bowman & Soares studies	Uyumaz studies
Scale	1:2	1:4
Continuous gutter width, W (m)	0.43	0.61
Continuous gutter cross slope S_{x2} (m/m)	0.059	0.04;0.06;0.08
Local depression (vertical:horizontal)	3:18	2:24
Inlet lengths (m) prototype	1.22;2.44;3.66;4.88	1.52;3.05;4.57

Uyumaz conducts one of the most complicated of experiments for curb opening inlets with three types of cross slopes involved; unfortunately the raw readings from the experiments are not available in the published works. Nonetheless, I digitized the 89 measurements provided in a figure in Uyumaz paper for $S_L = 4\%$ and used equation (4.17) to calculate the efficiency Q/Q_a (that ranged from 0.15 to 0.98) using $\frac{L}{FT}$ and then compared the results with the digitized (experimental) measurements of Q/Q_a. In average 3.81% MAPE is found, which shows the model for its tested data works reasonably. Only six combinations were provided in the 1992 paper, and in total 72 combinations were tested—89 readings for only six combinations opens a window into the multitude of experiments conducted by Uyumaz (1992). The details of experiments are in 0Table 4.4 and Table A.1 where S_L is along-road slope, S_{x1} road and S_{x2} gutter transverse slopes, L is inlet length, and h is the constant value of curb-opening height. While the width of the gutter depression was not written anywhere, Dr. Uyumaz recalls it to be 2-ft prototype.

The coefficients used in the solution ($A, B, C,$ and D) were published in a project report in 1991 in Turkish and they are included here in Table A.1 for the wider audience. Table A.1 shows that Uyumaz created the models for fixed length, albeit for a few lengths, and changed the amount of flow with which FT changes. While $\frac{L}{FT}$ is a dimensionless number, this was different from Karaki and Haynie (as in Izzard 1977) experiments who used multiple inlet lengths for fixed FT to study for a range of capture efficiencies—which is how Izzard came up with L_2 and L_3 that will be defined below. L_2 is not as clearly identifiable with the Uyumaz model, but this could be because the setup is more complicated with the existence of gutter depression.

In addition to above presented model, Uyumaz publishes 2002 paper providing a single third degree model, which is an improvement to equation (4.17); surprisingly, the results do not seem to differ between the two models, and the detailed coefficients are lacking for the 2002 model. Also in 2002 paper, Uyumaz provides a theoretical setup using energy equation, which is derived from the side-weir analogy for a triangular channel and could be used for undepressed analysis. Uyumaz did not test this solution because it is too simple for his experiments; however, he did provide a graph showing the numerical solution to this work while there was no attempt to provide the final analytical solution. Solving Uyumaz's model analytically yields:

$$x = -\frac{1}{4mS_x}\left(5\sqrt{y(E-y)} + 3E\sin^{-1}\sqrt{\frac{y}{E}}\right) \tag{4.18}$$

where S_x is the cross slope (single), y is the flow depth, E is the specific energy of the approach flow, and m is the discharge coefficient (for subcritical flows $m = 0.407\sqrt{\frac{1+0.5F^2}{1+2F^2}}$ and for supercritical $m = \frac{2}{3}(0.36 - 0.008F)$ where F is the Froude number at the beginning of the COI)—Uyumaz was not the first to use side-weir analogy; Zwamborn (1966) found, using linear head drop at the inlet opening $L = 3.05Qy^{-1.25}$.

Izzard 1977, despite its fame as the updated version of Izzard (1950) states that the data is lacking for gutters with higher cross slopes when compared to street sections. For this Bowman 1988, Soares 1991, and Uyumaz studies are attempts to close that gap to a large extend.

4.5 Li 1954

Li (1954) considers the elevation view of the flow at the end of an open channel with uniform velocity distribution and supercritical slope (Figure 4.10a) to be similar to the plan view of the flow on a uniform approach section with zero friction entering into a COI (Figure 4.10b). With this, the width of flow and the depth is correlated and the inlet length required for full capture is obtained. Li then applies an empirical coefficient to compensate for the ignored friction. From Figure 4.10a, the length is obtained as: $L_0 = Vt$ where V is the velocity and t the time, and flow depth from: $y = at^2/2$ where a is the acceleration. Using these two, Li obtains:

$$L_0 = V\sqrt{\frac{2y}{a}} \;\&\; L = V\sqrt{\frac{2y_1}{a}} \tag{4.19}$$

and from the ratio:

$$\frac{L}{L_0} = \left(\frac{y_1}{y}\right)^2 \tag{4.20}$$

Applying equation (4.19) to Figure 4.10b, Li finds inlet length to be $L_0 = v\sqrt{\frac{2y\tan\theta_0}{g\cos\theta_0}}$; using v from this in continuity equation along with the geometry in Figure 4.10b, in inlet length becomes: $L_0 = \sqrt{\frac{8}{\sin\theta_0}}\frac{1}{\sqrt{g}}Q_0 y^{-3/2}$. For gutters used in practice $\sin\theta_0 \to 1$, and because Li (1954) obtains this expression by neglecting friction, he applies an empirical coefficient K to the equation and obtains:

$$L_0 = \frac{1}{K\sqrt{g}}Q_0 y^{-3/2} \tag{4.21}$$

From experiments, Li finds $K = 0.23$ for $S_x = 8.33\%$ and $K = 0.2$ for $S_x = 2.083\%$ and 4.167%. Because Li considered a uniform velocity distribution, $Q/Q_0 = A/A_0$ (where A and A_0 are the cross sectional areas for the corresponding flows) which

using section A-A in Figure 4.9b and equation (4.20) leads to $\frac{Q}{Q_0} = 2\left(\frac{L}{L_0}\right)^2 - \left(\frac{L}{L_0}\right)^4$.

Li approximates this (for any bypass below 40%) to:

$$\frac{Q}{Q_0} \approx \frac{L}{L_0} \text{ for } \frac{Q}{Q_0} \geq 0.6 \tag{4.22}$$

Using equation (4.21) and equation (4.22) leads to:

$$L = \frac{1}{K\sqrt{g}} Q y^{-3/2} \tag{4.23}$$

which gives the inlet length that captures flow Q when the approach depth is y. In other words, as long as $Q/Q_0 \geq 0.6$, equation (4.23) is valid.

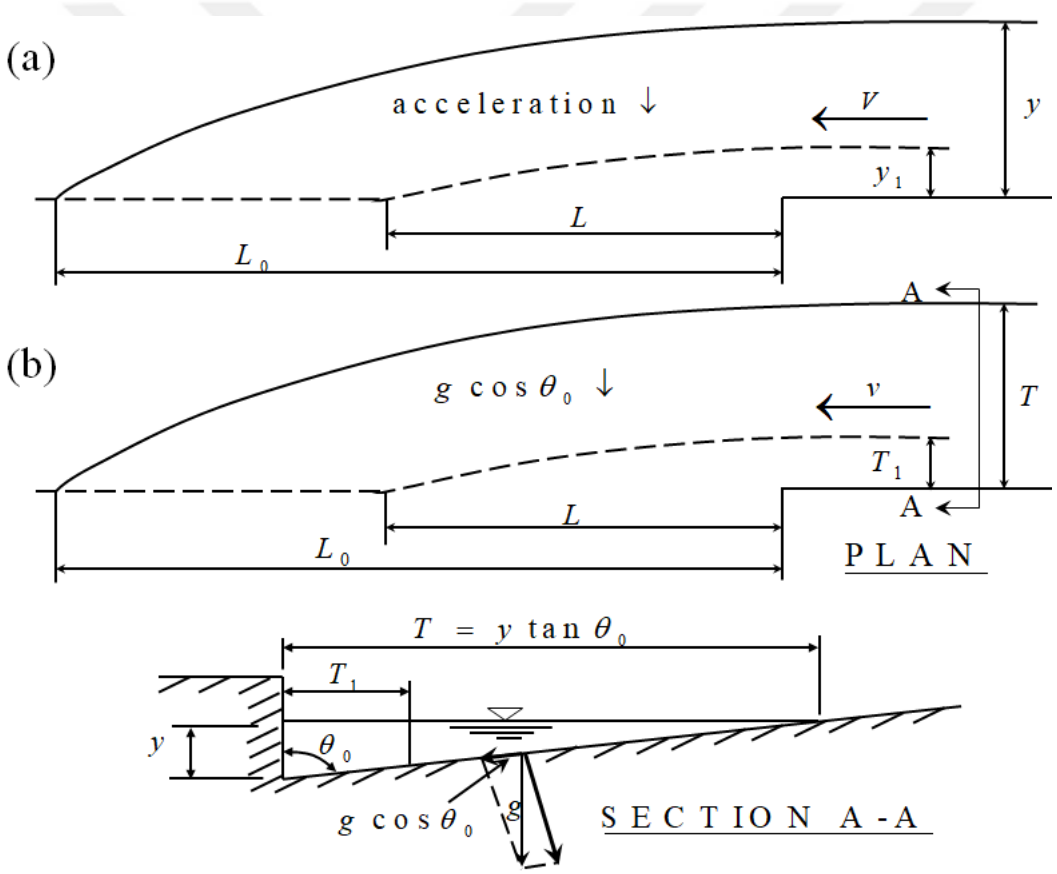

Figure 4.10 : (a) free drop of flow and (b) flow entering into UCOI (adapted from (Li 1954).

For the locally depressed case, based on the experimental data, Li et al. (1951) proposes:

$$L = \frac{1}{(K+C)\sqrt{g}} Q y^{-3/2} \tag{4.24}$$

where C is a function of several dimensionless numbers based on flow and inlet and approach section geometries, which becomes zero when no depression exists giving equation (4.23); thus, equation (4.24) is the general model C accounts for the extra flow captured due to the local depression.

4.6 Muhammad 2018

While HEC-22 updated Izzard's solution by using equivalent slope, the inconsistencies introduced overall was not removed entirely and discrepancies especially under certain conditions made it necessary to revisit HEC-22 solutions. Muhammad 2018, using large number of experimental data available in literature and the ones they conducted provides statistical corrections to HEC solutions using correlation analyses. One parameter that was entirely disregarded by HEC-22 for the locally depressed inlet solution—information on transition length and grade—was considered by Muhammad for full capture flow. He proposed that the right amount intercepted by a COI with local depression could be obtained via

$$Q = C_F Q_{HEC} \tag{4.25}$$

where Q_{HEC} is the flow calculated using HEC solution in equation (4.14) for the locally depressed inlet and $C_F = 2.8\left(\frac{a}{y}\right)^{0.24}\left(\frac{W}{T}\right)^{0.8} S_{UT}^{-0.13} S_W^{0.22}$ where a is the local depression depth, y flow depth (computed using Q_{HEC} in Manning's modified equation), W is depression's width, T is the spread of flow (computed using Q_{HEC} in Manning's modified equation or from $T = y / S_x$), S_{UT} is the longitudinal slope of the locally depressed gutter at the upstream transition ($S_{UT} = S_L + \frac{a}{L_{UT}}$ where S_L is uniform section's longitudinal slope and L_{UT} is upstream transition's length-- $\frac{a}{L_{UT}}$ is the longitudinal slope of the upstream transition measured from the longitudinal slope of the pavement), and S_W is the cross slope of the locally depressed gutter at the upstream transition ($S_W = S_x + \frac{a}{W}$ where S_x is uniform section's cross slope -- $\frac{a}{W}$ is the cross slope of the upstream depression measured from the cross slope of

the pavement). This shows clearly that C_F encompasses detailed information regarding the upstream transition. If L, the inlet length, is to be obtained for the full capture, then an iterative calculation is needed starting with an initial guess in obtaining Q_{HEC} until Q equals the expected approach flow.

Muhammad--unlike HEC-22 model in equation (4.15) where the approach flow is central to partial capture analysis with its amount and the inlet length needed for its full capture—takes the existing inlet as the key component and uses information about inlet's capacity to propose the captured flow model for any approach flow greater than that of inlet's full capture. His approach is wise as the information could easily be obtained experimentally adjusting the flow in laboratory conditions (because to re-construct a locally depressed inlet for the given approach flow is expensive and time consuming leaving one with approximations for the full capture inlet length that introduces new uncertainties to the model; Izzard's full capture solutions are classic examples to this). For the partially captured flow, Muhammad proposes:

$$Q = Q_{OF}\left[1 - m\left(1 - \frac{T}{T_{OF}}\right)\right] \tag{4.26}$$

where Q_{OF} is the flow existing inlet opening may intercept at full capture, T is the width of flow corresponding to approach flow, T_{OF} is the width of flow corresponding to Q_{OF}, and $m = \left(\frac{L_{UT}}{y_{OF}}\right)^{0.22}$ where L_{UT} is the length of upstream transition and y_{OF} is the normal depth in the approach section for flow Q_{OF}. Muhammad's models in equations (4.19) and (4.20) are improvements to HEC-22 solutions for the locally depressed inlets. However one should be mindful that all of the experimental data (bar one) used in obtaining and validating these models had equal up- and down-stream transition geometries, and it is not entirely clear how these solutions work with unequal transitions; while it is not the same geometry, Bowman finds that flow spreads are much affected by the downstream transition on a continuously depressed COI and states that "it is the length of downstream transition that which significantly alters the efficiency of the inlet," so in a scenario where the both are the same, it is not as easy to differentiate which impacts more. Li et al.

(1951)'s efficiency data, based on Muhammad's plots, was under-predicted both by HEC-22 and to an extent by Muhammad models, and a likely reason could be that its downstream transition is shorter when compared to the upstream's, indicating Bowman's conclusion might be valid for locally depressed case also. Li et al. (1951) mentions the hydraulic jump formed at the downstream transition which increases the amount of flow captured. Thus, leaving a room for improvement to Muhammad's model.

4.7 Follow ups by Izzard: the 1977 Paper

Schalla et al. (2017) mentions Izzard 1977 being adapted to FHWA-TS-79-225 (Jens 1979) and later being removed in HEC-22. Indeed a very detailed analysis based on Izzard 1977 was placed in 1979 version by Izzard himself as Jens writes in the preface of 1979 document that these were "prepared by Carl F. Izzard." Izzard (1977) analyses differed from the 1950 in that Izzard (1977) states that Q_i / Q changes linearly with L_i / L_1 up to a certain length $L_i = L_2$ where L_i is the installed inlet length and Q_i is the intercepted flow. L_2 changes with the dynamics of the approach flow and the road geometry, constrained mostly within the half of the full capture length and is equal to $3.01W^{-1/6}S_x^{1/2}FT$ where W is the width of inlet depression in m, S_x the cross slope of the pavement, F Froude number at distance W from the curb for the approach flow, and T the top width of the approach flow while $L_1 = 2.29W^{-1/6}S_x^{0.3}FT$ and is also known as the inlet length where $Q_i / Q = 1$ if it were to continue changing linearly with L_i / L_1. For any inlet length longer than $L_i = L_2$, however, Izzard finds the linearity is interrupted and Q_i / Q changes proportional to $\left(L_i / L_3\right)^{0.4}$ where $L_3 = 1.65FT$, the length that captures the entire flow (i.e. $Q_i / Q = 1$, the full capture case where Q is the approach flow). While Izzard states in his 1977 work that "not all sets of runs were used … because certain sets departed from the norm for unexplained reasons" –thus leaving us with doubt regarding method's consistency in general—it sheds great light into the behavior of depressed inlets and the unreasonableness of Izzard 1950 assumption that at the inlet lip water profile drops linearly.

In applying this work, the break point at $L_i = L_2$ is very likely misapplied in Schalla et al. (2017) for determining the full capture inlet length range at which the equations do fair predictions. This is because L_2 exists for any approach flow for a depressed inlet ($3.01W^{-1/6}S_x^{1/2}FT$) and takes values smaller than L_1, which is a length already shorter than the full capture inlet length. While (if modeled correctly)[4] Schalla et al.'s study is very insightful in identifying Izzard 1950's and HEC-22's limitations for long inlets, using L_2 to determine the range these models fail predicting *full* captures accurately is a misinterpretation of this concept. Any inlet length smaller than or equal to L_2 lacks capturing the approach flow fully, so any work on the subject of COIs should be read with special care.

4.8 Conclusions

In order to capture road surface flow efficiently, different designs of COI's developed over the years, and each design requires investigating its performance. With the use of equivalent slope concept, HEC-22 treats all designs as planar surfaces tagged with a cross slope that is equivalent to considering all pieces separately. This reduces all designs down to undepressed curb opening inlets with a uniform approach gutter regardless of the amount and the type of depression that exists. Because of this, Izzard 1977, which only covers depressed inlets, finds no place in HEC-22 despite its fame as the updated version to Izzard (1950), and Izzard (1950) becomes the mecca of criticism even to date. However, it is very interesting to note that Izzard (1977) using locally depressed inlets finds that up to an inlet length L_2, Q_i/Q changes linearly with inlet length L_i/L_1, which is somewhat consistent with Uyumaz's conclusion on the continuously depressed COIs with local depression that Q_i/Q changes linearly with the installed inlet length as long as $Q_i/Q < 0.6$. This, however, is entirely different from Li (1954)'s conclusion that for a COI with no depression $\frac{Q}{Q_0} \approx \frac{L}{L_0}$ for $\frac{Q}{Q_0} \geq 0.6$ (i.e. bypass below 0.4). Obviously all three have different depression designs from none to inlet (local) depression to

[4] Design change in analyses, see last paragraph under HEC-22.

inlet and gutter depression combined (composite); however, this depicts an entirely different direction from HEC-22's equivalent slope approach.

Izzard 1950 is one of the foundational studies in the subject, and this study makes it clear that Izzard's solutions were partly misapplied over the years which shadowed its fair evaluation and were partly misleading themselves for *inlet without depression* case, either due to vague titling the work or because of the difference between the analytical model and the experimental setup. Quite oddly, Izzard 1950 itself might be the inspiration to HEC-22 in putting forward the equivalent slope concept because, as shown in detail in this work, Izzard solved compound approach gutter problem with uniform cross slope assumption for the partially captured flow despite keeping it true to its analytical derivation balancing the changes via unrealistic full capture length model that is used as a parameter in the partial solution (that is equation (4.4)) and that Izzard's full capture model for the locally depressed inlets is already adapted from the undepressed using similarity. However, the use of equivalent slope concept and HEC-22's modification to equation (4.2) via replacing approach flow depth with its parameters as in equation (4.14) with the help of Izzard (1946) (and as a result as in equation (4.15)) become so powerful that HEC-22 stands by its own. Muhammad (2018) provides adjustments to HEC-22's undepressed as well as (locally) depressed inlet models on uniform pavements using the largest set of experimental data with an extensive data log. However long the solution steps may be, the fact that Muhammad considers the longitudinal slope of the transition in depressed inlets is something of importance especially considering HEC-22's disregard of it. The only difference in different designs in HEC-22, E_0, blurs when different types of depressions co-exist. In such a case where a local depression is placed on a compound slope, the guidelines needs updating also, and for that the works of Bowman, Soares, and Uyumaz can help either to setup an entirely new model or help determine an accurate equivalent slope.

It is shown with various examples how unclear use of certain terminology leads to misunderstandings and how common it occurs. To restate, a clear definition of terminology is necessary to overcome the incorrect evaluations of literature. To be in-line with the legacy of Izzard where he addresses composite design as inlet without depression (because gutter remains unchanged in the flow direction), I suggest using depressed gutter for composite section and depressed inlet for the local

depression around the inlet. Use of pavement for the road's clear section and gutter for the part adjacent to the curb with unequal transverse slope to that of pavement (as oppose to using gutter inclusive of both sections) is also recommended. Another factor that blurs clarity in general is the use of a in place of both continuously depressed and locally depressed segments; perhaps a slight modification (e.g. a_g vs. a_i for gutter and inlet depressions respectively) helps clear that. The author believes carefully crafting the words and analyses are necessary for the right direction in providing optimal models and in their future applications.

5. CONCLUSIONS

Data suggest that inclement weather increases the risk of accidents. Making drainage a priority in designing roadways prevents a great portion of these accidents and lower the degree of nuisances to overall road users and increase the longevity of roadways. With the motivation to contribute to these via providing 1) an easy to apply model for computing the over-lane flow depths so the geometries that are safe in terms of hydroplaning could be determined by any designer; 2) a detailed analysis of geometric configurations and putting forward not only which ones are *safe* in terms of hydroplaning but also which require attention in determining inlet spacing beyond the criteria of inlet capacity and maintenance purposes; and 3) the details on curb opening inlet literature and investigating some inconsistencies regarding certain designs in what is probably the most commonly used manual (i.e. HEC-22).

In the second section, a simple yet physically based solution is presented for obtaining road-lane water film thickness, and the model is verified against already existing data from literature. HEC-22 documents that thicknesses of 2 mm could cause full hydroplaning at 80 kph, which is certainly alarming in terms of economics. What is more alarming, however, as the methods improve to obtain accurate observations of flow depths in the field, the studies report inaccuracies of posted speed limits—that they are higher than the hydroplaning speed limits (Gurganusa et al. 2021). This risks the safety of users. The aim in this section was to map the geometries that reduce the chances of vehicles being such dangerous situations, so those are avoided. The present study showed that cross slope optimization is a safe way to avoid hydroplaning depths for grades up to 10% and widths up to 15 m for intensities below 250 mm hr^{-1}. Roadway width, as much as mandated by the need, may be judged based on the constraint, provided that a certain width is inconvenient in terms of drainage. The study shows also the diminishing decreasing influence of increasing cross slopes on water depths; meaning, increasing cross slope does not provide a constant impact on decreasing flow depths and the influence drops as the slope values increase, so in terms of the bigger picture, the cons of increasing cross

slope should also be considered and an optimization becomes necessary. In conclusion, while some factors that affect roadway water depth may not be controlled, the designer may restrain the cross slope values to create safe roads. Section two also provides the safest slope values coupled with design intensities without hydroplaning threat beyond; beyond those cross slope adjustment is required for preventing hydroplaning to shorten flow paths, and thus, water depths. Zero and even 1% cross slope should be avoided at all costs, considering also the changes on road surface over use with time.

Common practice in inlet spacing analyses is to consider inlet capacity (as they come in certain sizes) and maintenance restrictions be it in terms of access points or the spread criteria from the curb. In the 3rd section, this work looks into the inlet spacings in terms of road geometry only rather than the formerly mentioned two to investigate whether it needs consideration in planning. Usually a fixed spread is used as a criterion, but this work shows at locations where the flow depth needs being regulated, spreads may not provide the needed depth restrictions and should be coined with channel flow depth. HEC-22 recommends comparing flow depths with curb heights and our results show that when flow depths are allowed to be that high, roadway geometry does not limit the spacing. However when keeping depths as low as 3 cm is necessary for the channel flow, then road geometry influences inlet spacing. This is another important part of in establishing safe roads in addition to hydroplaning investigated in the earlier section under inclement weather conditions. The analyses were done using Manning's equation along with Rational Method and the solution from 2nd section—latter to dismiss geometries that cause hydroplaning depths. While seemingly spacing changes with S_x^{-1} (equation. (3.5)), because z is also a function of cross slope, it changes with $S_x^{5/3}$, in reality. Thus, for the most part, as the cross slope increase, the spacing should increase. However when the cross slope value tops the ratio of maximum allowable flow depth to flow spread value, $S_x > z_{\max}/\mathrm{T}_{\max}$, then the spacing changes with S_x^{-1} because the maximum depth value must be used in obtaining spacing which is no longer a function of cross slope. That shows that if the flow depths should be kept under control beyond being limited by the curb height, the chances of $z_{\max}/\mathrm{T}_{\max}$ controlling the inlet spacing increase. The results also show that while higher longitudinal slopes enables wider spacing of

inlets, depending on the roadway width and cross slope values, the design may collapse altogether due to hydroplaning risks leaving no reason to analyze the spacing. The conditions should also be investigated superelevations.

Flow on roadways is directed to underground drains via different types of inlets, and in the 3rd section some criteria regarding their spacing were explored. As a follow up, 4th section explored the existing models for curb opening inlets, and their capacities for they do not interfere with the traffic and do not get clogged easily, becoming one of the most advantageous types to collect stormflow. Latter advantage keeps the inlets functioning with least maintenance; former helps driver comfort and helps eliminate driving sideways—a persisting problem with the through-roadway inlets. For a country such as Turkey, where COIs are not used but a potential exists for employing such inlets, manuals such as HEC-22 could be a logical starting point for setting practice standards as the government administrations are responsible for publishing such standards. This is what inspires the detailed investigations of COI models in this work, that the advantages of COIs may spark local or national governments to adopt COIs for drainage.

In the larger picture, existence of cross slope is the starting point of directing flow to a limited portion of the road; in the same manner, slope alterations at the inlet vicinity, (locally) depressed inlets, direct channelized flow towards the inlet; any increase in cross slope close to the curb line (usually <0.9 m) throughout the roadways is considered a continuous depression for the inlet. (If the depression encompasses one (or more) entire lane, than that is unrelated to the inlet depression and such roads are considered compound.). *Goal* of the increase in cross slope in either case is then what seems to determine how the inlet is categorized. Changes in geometry that leads to variations in flow dynamics require different models for the inlets as those alter inlet capacity. One of the earliest models on COIs is Izzard (1950) and it is considered to be an analytical model. Part of this chapter shows that it is not entirely true, and it is a misunderstood or a misleading work in this scope. Impression regarding the Izzard work is that he first derives analytical equations and then empirically adjusts the models. This work shows that none of the empirical full capture models of Izzard has analytical roots. The fact that Izzard obtains models for different geometries implies that he investigates the situations based on the section views; however, Izzard never makes any mention of the sections he considers when

obtaining the analytical models. He presents his models as *inlet with a depression* and *without depression* with no details of what depression or without depression implies; the empirical calibration is done for a continually depressed inlet in a way the approach flow depth includes the continuous depression depth also. This creates an enormous difference between the undepressed and the continuously depressed; thus, when anyone who mistakenly applies his empirical solution to uniformly sloped sections obtains gigantic errors. For the depressed full capture model, Izzard (1950) (without mentioning) uses the model he obtains for the continuously depressed inlet (i.e. the *undepressed inlet*, with his wording). So for the (locally) depressed inlets, Izzard fulfills only difference in his posted assumption, head drops from $y+a$ to a in depressed as opposed to from y to 0 between the two ends of the inlet. But the approach he uses has no analytical basis: he hypothesizes that the flow captured per unit length in an [as he refers] undepressed inlet with approach depth of $y+a$ is equal to the flow captured per unit length in a depressed inlet with approach flow depth of y; put in mathematical form: $\left(Q_a / L_a\right)_y = \left(Q_0 / L_0\right)_{y+a}$. This has no basis analytically. And because he uses these baseless models for full capture inlet lengths in his partial capture models (both depressed and undepressed), his partial models are also a mixture of different inlet types and are baseless analytically. Beyond what Izzard did and what his models respresent, this work tests $\left(Q_a / L_a\right)_y = \left(Q_0 / L_0\right)_{y+a}$ for a dataset collected on a purely undepressed inlets by Hammonds and Holley (1995) and finds it to work relatively accurately, but because giving relatively reasonable results does not change the realities regarding its (non)analytical roots.

This work presents the way HEC-22 limits all designs down to undepressed COI model via obtaining *equivalent* cross slopes for each inlet type. The equivalent slope does not encompass the longitudinal slopes, although for local depressions, transitions up- and down-stream of the inlet changes the longitudinal slopes. Muhammad (2018) traces the roots of HEC-22 models to Izzard 1950. The use of Izzard (1946)'s flow depth formula in Izzard (1950) models enables HEC-22 to incorporate slopes, which are not apparent in the original Izzard models. Equivalent slope concept in HEC-22 might have been inspired by Izzard (1950)'s use of a single type of experimental data to calibrate all COI models—thus unlike Muhammad's comment that HEC-22 departs from the Izzard solution, this work demonstrates how

HEC-22 follows the "footsteps" of Izzard (1950) via unmasking Izzard's use of mismatched data and geometry. The slope is considered *equivalent*, but because the existence of a depression should incorporate both longitudinal and cross slopes, and the fact HEC-22 incorporates only cross slopes within it shows that it is less than equivalent. Nonetheless, this parallels with Izzard's use of continuous depression as the undepressed, where there is no change in longitudinal slope. Muhammad updates models incorporating transitions to the inlet depression. It enhances the accuracy of HEC-22 models; perhaps what makes models more accurate is updating those with different up- and down-stream transitions, as most experimental data used by Muhammad (2018) were collected on equal transitions. Muhammad's updates were for undepressed and locally depressed inlet and did not include depressed inlets with continuous gutter depression. It is not much of a surprise because as this work unveils, HEC-22 lacks providing a clear cut solution to such combined design. This work suggests that the models of Soares (1991) and Uyumaz (1992, 2002) could be incorporated into HEC-22 design equations.

With the use of equivalent slope concept, HEC-22 treats all designs as planar surfaces tagged with a cross slope that is equivalent to considering all pieces separately. This reduces all designs down to undepressed curb opening inlets with a uniform approach gutter regardless of the amount and the type of depression that exists. Because of this, Izzard 1977, which only covers depressed inlets, finds no place in HEC-22 despite its fame as the updated version to Izzard (1950), and Izzard (1950) becomes the mecca of criticism even to date. Interestingly, however, Izzard (1977) using locally depressed inlets finds that up to an inlet length L_2, Q_i/Q changes linearly with inlet length L_i/L_1, which is somewhat consistent with Uyumaz's conclusion on the continuously depressed COIs with local depression that Q_i/Q changes linearly with the installed inlet length as long as $Q_i/Q<0.6$. This, however, is entirely different from Li (1954)'s conclusion that for a COI with no depression $\frac{Q}{Q_0}\approx\frac{L}{L_0}$ for $\frac{Q}{Q_0}\geq 0.6$ (i.e. bypass below 0.4). Obviously all three have different depression designs from none to inlet (local) depression to inlet and gutter depression combined (composite); however, this depicts an entirely different direction from HEC-22's equivalent slope approach of essentially treating all models the same.

In summary, this work considers various aspects to draining roadways with the goal to eliminate inclement weather aftermath. It explores major models for the curb opening inlets, which are a great substitude under many cases to grate inlets. It provided he missing parts of commonly used HEC-22, with are additions to Muhammad's updates to HEC-22 models and provided alternatives to current state; this work also showed that it is not necessaryly analytically-based and that HEC-22 should provide guidance to composite COIs. This work showed that depending on the criterion, inlet spacing should involve beyond inlet capacity and consider the allowable flow depths also, in addition to the spread. Additionally, only the designs that are hydroplaning-safe should be used as a starting point to designing any roadway geometry, and a model to easily obtain the depths that sets the limit is provided. The variations in terms of roadway slopes is framed.

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APPENDICES

APPENDIX A: Uyumaz Model's Coefficients
APPENDIX B: Inlet Spacing Function Code

Appendix A: Uyumaz Model's Coefficients

Table A.1: Uyumaz model coefficients in equation (4.17) for various configurations.

L/h	S_L	S_{x1}	S_{x2}	A	B	C	D
10	0.00	0.02	0.04	1.63	-1.617	3.074	-0.32
10	0.00	0.02	0.06	1.529	-1.509	2.934	-0.324
10	0.00	0.02	0.08	1.409	-1.202	2.562	-0.276
10	0.00	0.04	0.06	1.292	-0.702	1.922	-0.141
10	0.00	0.04	0.08	1.116	-0.631	1.873	-0.224
10	0.00	0.06	0.08	1.007	-0.496	1.568	-0.159
10	0.02	0.02	0.04	1.047	-0.57	1.791	-0.242
10	0.02	0.02	0.06	0.998	-0.462	1.6	-0.196
10	0.02	0.02	0.08	0.964	-0.432	1.526	-0.183
10	0.02	0.04	0.06	0.926	-0.340	1.352	-0.132
10	0.02	0.04	0.08	0.878	-0.292	1.258	-0.123
10	0.02	0.06	0.08	0.822	-0.286	1.224	-0.141
10	0.04	0.02	0.04	1.217	-0.743	2.073	-0.245
10	0.04	0.02	0.06	1.268	-0.486	1.672	-0.13
10	0.04	0.02	0.08	1.116	-0.368	1.464	-0.08
10	0.04	0.04	0.06	1.061	-0.193	1.193	-0.011
10	0.04	0.04	0.08	1.008	-0.264	1.27	-0.062
10	0.04	0.06	0.08	0.923	-0.252	1.222	-0.089
10	0.06	0.02	0.04	1.296	-0.301	1.527	-0.04
10	0.06	0.02	0.06	1.225	-0.337	1.555	-0.08
10	0.06	0.02	0.08	1.183	-0.355	1.547	-0.093
10	0.06	0.04	0.06	1.131	-0.291	1.41	-0.066
10	0.06	0.04	0.08	1.066	-0.348	1.497	-0.132
10	0.06	0.06	0.08	0.978	-0.284	1.354	-0.123
20	0.00	0.02	0.04	1.178	-0.35	1.402	-0.02
20	0.00	0.02	0.06	1.104	-0.362	1.383	-0.044
20	0.00	0.02	0.08	1.033	-0.33	1.327	-0.06
20	0.00	0.04	0.06	0.946	-0.284	1.27	-0.092
20	0.00	0.04	0.08	0.894	-0.25	1.184	-0.083
20	0.00	0.06	0.08	0.833	-0.119	0.904	0.003
20	0.02	0.02	0.04	0.97	-0.435	1.557	-0.198
20	0.02	0.02	0.06	0.915	-0.45	1.6	-0.259
20	0.02	0.02	0.08	0.89	-0.314	1.312	-0.142
20	0.02	0.04	0.06	0.82	-0.21	1.121	-0.108
20	0.02	0.04	0.08	0.778	-0.122	0.931	-0.046
20	0.02	0.06	0.08	0.727	-0.081	0.832	-0.033

Table A.1 (continued): Uyumaz model coefficients in equation (4.17) for various configurations.

L/h	S_L	S_{x1}	S_{x2}	A	B	C	D
20	*0.04*	*0.02*	*0.04*	*1.184*	*-0.357*	*1.455*	*-0.041*
20	*0.04*	*0.02*	*0.06*	*1.09*	*-0.257*	*1.297*	*-0.034*
20	*0.04*	*0.02*	*0.08*	*1.008*	*-0.293*	*1.348*	*-0.098*
20	*0.04*	*0.04*	*0.06*	*0.955*	*-0.247*	*1.253*	*-0.09*
20	*0.04*	*0.04*	*0.08*	*0.902*	*-0.198*	*1.144*	*-0.073*
20	*0.04*	*0.06*	*0.08*	*0.853*	*-0.16*	*1.04*	*-0.054*
20	0.06	0.02	0.04	1.247	-0.269	1.453	-0.034
20	0.06	0.02	0.06	1.182	-0.13	1.224	0.016
20	0.06	0.02	0.08	1.128	-0.153	1.21	0.003
20	0.06	0.04	0.06	1.058	-0.284	1.389	-0.096
20	0.06	0.04	0.08	0.995	-0.074	1.026	0.009
20	0.06	0.06	0.08	0.904	-0.176	1.141	-0.079
30	0.00	0.02	0.04	1.02	-0.236	1.223	-0.019
30	0.00	0.02	0.06	0.964	-0.374	1.462	-0.162
30	0.00	0.02	0.08	0.908	-0.325	1.335	-0.134
30	0.00	0.04	0.06	0.848	-0.295	1.251	-0.133
30	0.00	0.04	0.08	0.787	-0.216	1.083	-0.096
30	0.00	0.06	0.08	0.676	-0.29	1.25	-0.283
30	0.02	0.02	0.04	0.903	-0.312	1.282	-0.105
30	0.02	0.02	0.06	0.849	-0.199	1.043	-0.027
30	0.02	0.02	0.08	0.811	-0.151	0.925	0.008
30	0.02	0.04	0.06	0.77	-0.22	1.095	-0.118
30	0.02	0.04	0.08	0.725	-0.219	1.097	-0.158
30	0.02	0.06	0.08	0.646	-0.135	0.855	-0.079
30	0.04	0.02	0.04	1.12	-0.192	1.205	0.025
30	0.04	0.02	0.06	1.051	-0.278	1.324	-0.06
30	0.04	0.02	0.08	1.008	-0.244	1.242	-0.049
30	0.04	0.04	0.06	0.921	-0.191	1.136	-0.057
30	0.04	0.04	0.08	0.848	-0.228	1.132	-0.086
30	0.04	0.06	0.08	0.742	-0.136	0.871	-0.018
30	0.06	0.02	0.04	1.148	-0.412	1.599	-0.122
30	0.06	0.02	0.06	1.097	-0.411	1.549	-0.122
30	0.06	0.02	0.08	1.061	-0.278	1.327	-0.056
30	0.06	0.04	0.06	1.002	-0.252	1.255	-0.059
30	0.06	0.04	0.08	0.954	-0.295	1.314	-0.109
30	0.06	0.06	0.08	0.837	-0.205	1.137	-0.109

Appendix B: Inlet Spacing Function Code

FUNCTION FOR INLET SPACING

```
%This function outputs the inlet spacing for roads with a single cross slope when
called. It makes sure:
% 1 the flow depth at the gutter-lane intersection does not reach beyond *4mm* of
hydroplaning depths (Eq. a).
% 2 the water depth at the curb is not beyond *10cm*
% 3 It also considers for the surface flow depth in terms of what intrudes into the
lanes from the curb side. It limits the maximum surface water depth reaching
*4mm*.
% $$z=K_T(Inb)^0.6((sx^2+sL^2)/sx^4)^0.15$$ (Eq.(3.1))

function x = inletSpacingupdatedhighway(I,LN,sL,sx)
```

Parameters related to Rational Method

```
C=0.95;
Kr=360*10^4; %SI; use 43200 for US
lane_width=3.6; %m
B=lane_width.*LN; %m; LN is lane number
sh=3 ; %m shoulder width, changes between 4 to 12ft and in general is greater than
6ft.
```

Parameters for Manning's Equation

```
Km=1; %SI; for US use 1.486.
nsf=0.011; %Manning's coef for over-lane flow
n=0.016; %Manning's coef for channel flow
zmax=4; %mm for hydroplaning free lanes
WDmax=0.1; %m maximum water depth at the curb
```

Surface water depth parameter for Eq. a

```
Kt=6.92; %for unit consistency in SI
```

Obtaining inlet spacing

```
x=zeros(length(I),length(LN), length(sL),length(sx));

for r=1:size(I,2)
   for s=1:size (LN,2)
         for k=1: size (sL,2)
            for j=1: size (sx,2)

wd=(Kt./I(r).^0.4.*(nsf*B(s)).^0.6*((sx(j).^2+sL(k).^2)/sx(j).^4).^0.15)/60*I(r);%
mm surface flow (Eq. (3.1))
               if wd<=zmax && (sx(j)*sh+wd/1000)<WDmax

x(r,s,k,j)=3/8*Km*Kr./(C*n).*sL(k).^0.5./sx(j).*(sh.*sx(j)+wd/1000).^(8/3)./(I(r).*(
B(s)+sh));%m Manning and Rational put together; sh in place of W.
```

```
                elseif wd<=zmax && (sx(j)*sh+wd/1000)>=WDmax

x(r,s,k,j)=3/8*Km*Kr./(C*n).*sL(k).^0.5./sx(j).*WDmax.^(8/3)./(I(r).*(B(s)+sh));
%m limits water depth at the curb to a max 10cm.
                end
                if wd>zmax; x(r,s,k,j)=nan; end
              end
            end
    end
end
```

Calling

```
%To produce inletSpacing data as used in one of our figures, a simple main script (or using command window) may include: I=[150 200 250 300 350 400 450];LN=[1 2 3 4];sL=0.01:0.01:0.1+0.0001;sx=0.01:0.01:0.06; x=inletSpacing(I,LN,sL,sx)
```

CURRICULUM VITAE

Name Surname **:** Sevgi Cavdar

PUBLICATIONS ON THE DISSERTATION

- **Cavdar S.,** Uyumaz., A. (2022). Hydrological considerations in designing roadways: Avoiding hydroplaning. Teknik Dergi, 33(5) doi:10.18400/tekderg.989134 (in press)
- **Cavdar S.,** Uyumaz., A. Impact of slope orientation on inlet spacing: gutter flow analyses (under review)
- **Cavdar S.,** Hodges, B. R., Uyumaz., A.Intercepting roadway flow: curb opening inlets; foundational models, underlying assumptions and critiques for flush inlets (draft finished)