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Hydrological Considerations in Designing Roadways: Avoiding Hydroplaning

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ABSTRACT

High water levels on lanes poses high risk to the safety on highways. Since drainage structures are mostly focused on water spread issue at sideways, consequences of the build-up flow on the surface is overlooked . This study addresses whether optimizing cross slopes prevents hydroplaning. Water depths obtained using kinematic wave equation were tested against several studies for verification. Wide range of rainfall intensities and cross slopes were covered. Findings revealed that cross slope optimization for grades up to 10% prevents hydroplaning for intensities below 250mm/hr with widths up to 15m. The findings also shows cross slope optimization must be considered simultaneously with inlet design work.

Keywords: Cross slope optimization, hydroplaning, kinematic wave equation, roadway drainage, sheet flow on roadways.

1. INTRODUCTION

High water depths on lanes risk safety that is essential for every highway. When the increased water depths create pressures equal to or more than the pressure due to 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 on traffic lanes that may create dangerous depths: (1) intrusion of flow adjacent to curb into the lanes, and (2) precipitation on the road surface. Latter is reduced through altered pavement cross slopes. Ross and Russam [11] and Gallaway et al. [6] investigated water depths experimentally with different roadway geometries. However, their solutions are limited to the tested conditions. Cristina and Sansalone [5] developed a rainfall-runoff kinematic wave model for highways, but the solution remains in differential form and of no practical use; they were only concerned with the concentration time of the flow. Studies on drainage facilities are concerned with the

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sideway gutter flow and its spread, but the flow contributing to hydroplaning directly from the roadway surface is studied to a lesser extent and the geometric limits to antihydroplaning values are not well established.

This study 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 covered 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. 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 1A), which is the angle roadway centerline makes with the resultant of the cross and longitudinal slopes S_x and S_L, respectively (Figure 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 orthogonality 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.

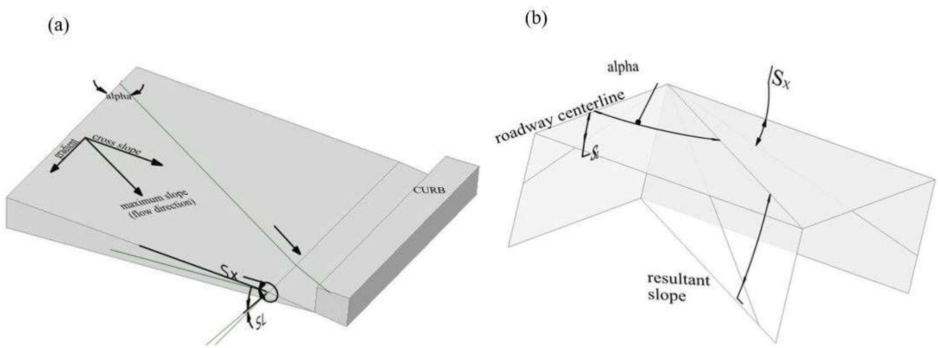

Figure 1 - Longitudinal and cross slopes forming the resultant, which helps obtain alpha and flow direction: (a) on a roadway section and (b) slopes at a magnified view.

2.1. 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. [3] 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 [10] investigated the limiting criteria for sheet flow. They concluded that for the composite parameter $nL/\sqrt{S}$ where n is the Manning roughness coefficient, L is the flow length, and S is the surface slope, values below 30 SI (International System of units) and 100 USC (United States Customary units) gives acceptable errors as a criterion and thus below that limit sheet flow assumption holds. If sheet flow exists, a kinematic wave equation may be employed to estimate the time of concentration, but if it does not, friction slope being different than the bed slope, kinematic wave assumption fails. For the most part, it is safe to assume that the criteria hold 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} .

Table 1 - Manning's roughness coefficient (n) for overland sheet flow (Eq. (9)), after HEC-22 [2].

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

Using Manning's equation for sheet flow:

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

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

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

Thus, unit discharge for sheet flow can also be expressed as

$$q = kz^m \tag{3}$$

where $k = K_M S^{1/2} / n$ and $m = 5/3$. Considering a Control Volume along the flow direction y with a representative cross section of $z\Delta y$, as depicted in Figure 2, as Δy and $\Delta t \to 0$ inflow and outflow can be written as follows:

$$\begin{aligned} &\text{inflow: } (q_y + I\Delta y)\Delta t \\ &\text{outflow: } (q_y + \frac{\partial q}{\partial y}\Delta y)\Delta t \end{aligned} \tag{4}$$

where q_y is the unit discharge flow in the longitudinal direction, *y*, and I is the source term (rainfall intensity) in mm hr^{-1} (in. hr^{-1}). Difference between the inflow and outflow in Eq. (4) equals the change in storage:

$$(q_y + I\Delta y)\Delta t - (q_y + \frac{\partial q}{\partial y}\Delta y)\Delta t = (z + \frac{\partial z}{\partial t}\Delta t)\Delta y - z\Delta y \tag{5}$$

Simplifying Eq. (5), one obtains the continuity equation for the sheet flow:

$$\frac{\partial q}{\partial y} + \frac{\partial z}{\partial t} = I \tag{6}$$

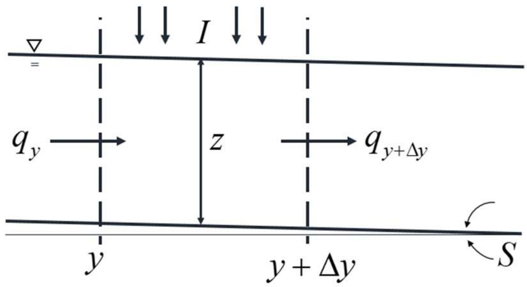

Figure 2 - Definition sketch for the continuity equation in the longitudinal section for sheet flow.

Using Eq. (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: $\dfrac{dy}{dq/dz} = \dfrac{dt}{1} = \dfrac{dz}{I}$. Hence

$$\frac{dz}{dt} = I \rightarrow z = IT_{ti} \tag{7}$$

where T_{ti} is the travel time for the first segment of flow, i.e. travel time for the segment considered within the scope of this study. Since neither z nor T_{ti} is known, using Eq. (3) along with $\dfrac{dy}{dt} = \dfrac{dq}{dz}$ (obtained from the method of characteristics) and inserting z in Eq. (7) one obtains

$$\frac{dq}{dz} = mkz^{m-1} \rightarrow \frac{dy}{dt} = 5/3k(IT_{ti})^{2/3} \tag{8}$$

After inserting k as defined in Eq. (3) and rearranging one obtains

$$T_{ti} = \frac{K_T}{I^{0.4}}\left(\frac{nL}{\sqrt{S}}\right)^{0.6} \tag{9}$$

where K_T is a coefficient equal to 6.92 or 0.933, in SI and USC, respectively. This is also the equation suggested by US Federal Highway Administration.

The angle resultant slope makes with the roadway centerline depicted in Figure 1, α, is the crucial part in determining the length of flow L in Eq. (9), and defined:

$$\alpha = \sin^{-1}\left(S_x / S\right) \tag{10}$$

Since L = b/ sin α with reference to Figure 1, using the aforementioned definition of S, α becomes, $\alpha = \sin^{-1}\left(S_x / \left(S_x{}^2 + S_L{}^2\right)^{1/2}\right)$. Thus, $L/\sqrt{S}$ in Eq. (9) becomes:

$$\frac{L}{\sqrt{S}} = \frac{b/\sin\alpha}{\sqrt{S}} = \frac{b/(S_x/S)}{\sqrt{S}} = b\frac{\left(S_x{}^2 + S_L{}^2\right)^{0.25}}{S_x} \tag{11}$$

Eq. (11) inserted in Eq. (9) gives:

$$T_{ti} = \frac{K_T}{I^{0.4}} \left(n \frac{b / \sin \alpha}{\sqrt{S}} \right)^{0.6} = \frac{K_T}{I^{0.4}} \left(nb \frac{\left(S_x^{\ 2} + S_L^{\ 2} \right)^{0.25}}{S_x} \right)^{0.6} = \frac{K_T}{I^{0.4}} (nb)^{0.6} \left(\frac{S_x^{\ 2} + S_L^{\ 2}}{S_x^{\ 4}} \right)^{0.15} \quad (12)$$

Using Eqs. (7) and (12) one obtains:

$$z = K_T (Inb)^{0.6} \left(\frac{S_x^{\ 2} + S_L^{\ 2}}{S_x^{\ 4}} \right)^{0.15} \quad (13)$$

Eq. (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 [10], and the relationship is obtained between the travel time and roadway geometry. The flow length, L, one of the major determinants of travel time, ranges from the roadway width for $S_L = 0$ to the roadway length bounded by a sump, for $S_x = 0$. 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. Knowing the depth of water helps setting the optimal cross slopes for a given rainfall intensity to avoid unwanted incidents due to hazardous hydroplaning.

2.2. Verification of the Model

The magnitute of water depth, which is highly dependent on the cross slope of the pavement, is crucial in producing hydroplaning, and the correctness of water depths obtained using the kinematic wave equation depends on how well the travel time is estimated. This study checks whether the solution for travel time compares well with the experimental data made available by the previous work.

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 [11] on an 11 m x 5.5 m platform. After running experiments on two surfaces under various cross slopes with resultant slope 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}, they recommended the use of Eq. (14).

$$z_{R\&R} = 0.474 (LxI)^{1/2} S^{-1/5} \quad (14)$$

where $z_{R\&R}$ is water depth in mm as reported by the Ross and Russam [11]; 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[11] were obtained using Eq. (7) after finding water depth from Eq. (14).

Wong [13] compared several time of concentration formulas with experimental data. They reported that formulation by Chen and Wong [4] estimates the best. However, the formulation by United States Corps of Engineers' [12] (USACE), Eq. (15), based on R^2 (the quotient of the sum of squared errors and the total sum of squares) estimates the time of concentration better than the Chen and Wong [4] formulation. USACE recommendation for travel time is

$$t_{USACE} = \left(10.57 + 0.12 / S_L\right)\left(b / 30.48\right)^{(0.55-0.001/S_L)} I^{-0.43} \tag{15}$$

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}.

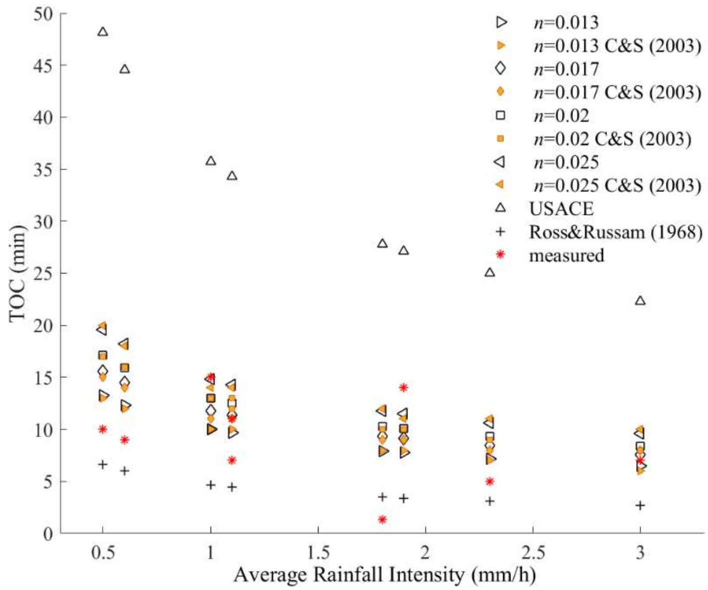

Figure 3 - Comparing our kinematic wave model with that of Cristina and Sansalone [5] (referred to as C&S (2003) in the figure) model and of the experimental results for 2% cross slope, 0.4% longitudinal slope for a road width of 20 m which remains within the criteria limits posted by McCuen and Spiess [10].

Cristina and Sansalone [5] mentioned the rarity of adopting kinematic wave equation for modelling 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 experimental data from Cristina and Sansalone [5] are compared with their results as well as with USACE and Ross and Russam [11]'s empirical equation as provided in Eq. (14). Comparison between the Cristina and Sansalone findings and the findings of the present work are in good agreement as depicted in Figure 3. Yet the formulation presented in this work is easier to be used. In comparison with USACE, the kinematic wave solution outperforms; however, it should be kept in mind that the experiments are limited. Although Wong [13] showed that the USACE solution is the best in predicting the time of concentrations, both the present work's and Ross and Russam [11] formulations are better than USACE results which highly overestimates. It is worth noting that selection of correct n value is very crucial for the implementation of kinematic wave approach. The tendency for the overestimation of the time of concentration with kinematic wave approach may be attributed to the abstractions in the runoff process.

2.3. Water Depth Variations under Different Design Rainfalls and Roadway Geometries

Water depths are determined from Eq. (13), and the results are plotted in *Figure 4* and Figure 5. The roadway was considered to have a width of 15 m with multiple lanes and a Manning coefficient of $n = 0.016$, slightly higher than provided in Table 1.

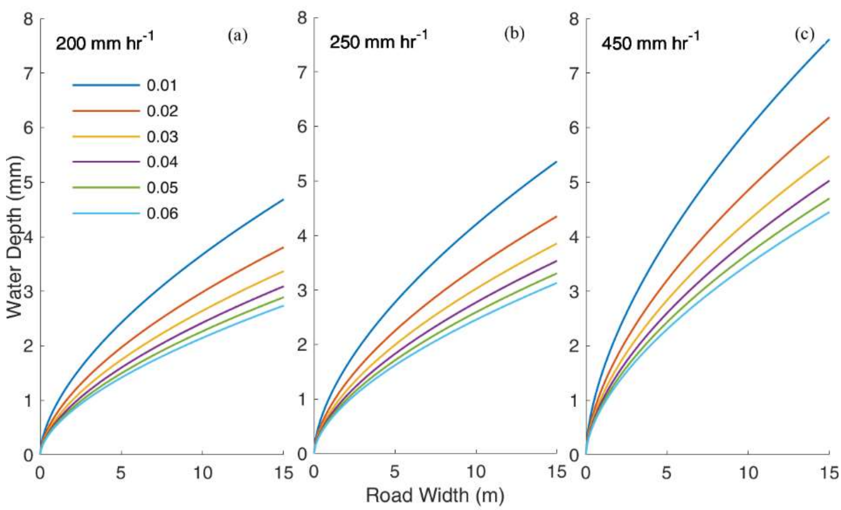

Figure 4 - For $S_L = 0$, the effect of changing cross slope on the flow depth as a function of width is provided for rainfall intensities (a) 200, (b) 250, and (c) 450 mm hr^{-1}.

Figure 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; Figure 4). It is obvious from Table 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 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.

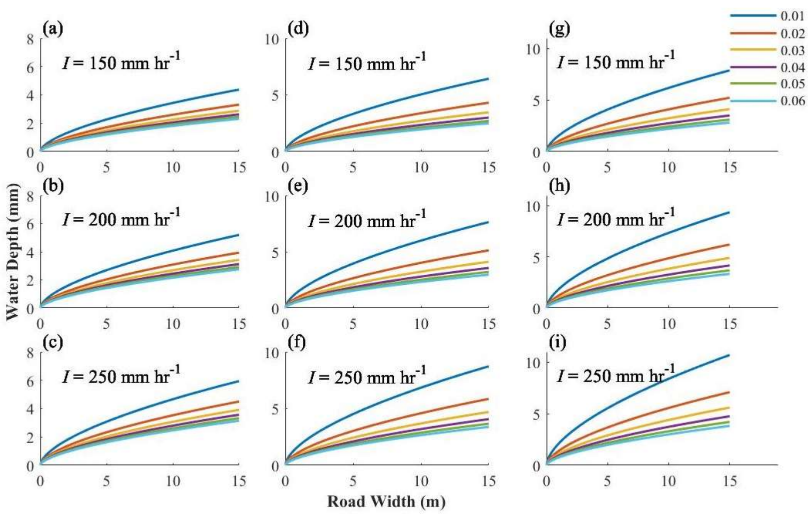

Figure 5 - Flow depths through roadway width for $S_L = 0.01$ *in (a), (b), and (c);* $S_L = 0.05$ *in (d), (e), and (f); and* $S_L = 0.1$ *in (g), (h), and (i) with cross slopes ranging from 1% to 6% under rainfalls of 150, 200 and 250 mm hr*$^{-1}$.

Figure 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 4*. Water-depth versus road-width shows that for $b = 6.5$ m with $S_x = 2\%$ 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.

Table 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.

3. DISCUSSIONS

Engineering most of the time is about setting the criteria and planning around the rare instances that can be highly hazardous, be it earthquakes, volcanic eruptions, winds, or floods. Effectively planning urban traffic involves many factors for the decision-makers [8], 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 89 km h^{-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 [2]). Gurganusa et al. [7], 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. [6] recommend limiting water depths at or below 4 mm to prevent hydroplaning. Risk of partial hydroplaning continues at lower depths. It is assumed in this paper that flow depths below 2 mm do not constitute a danger for dynamic hydroplaning around the speeds of 90 km hr^{-1} (55 mph), and 4 mm is the limit depth above which must be avoided. 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 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 [1]. 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. Figure 6 shows the hydroplaning starting grades up to 10% for a given rainfall intensity; those analyses are conducted for a transverse slope of 1% and show that the area that falls below the lane-line is safe and requires no cross-slope optimization (i.e., 1% is enough), but the area above requires further investigation of proper cross slope to avoid hydroplaning. For example, for a 3-lane roadway with a cross slope of 1%, intensities up to 75 mm hr^{-1} require no further analysis to prevent hydroplaning (Figure 6; Table 3).

The present work revealed that the likelihood of a single-lane road hydroplaning is rare. However, once the roadway width or grade increases, cross slope optimization becomes inevitable because of increased water accumulation.

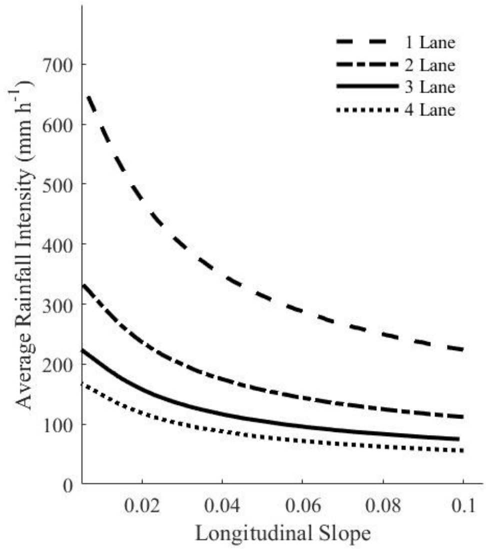

Figure 6 - Influence of increasing lane number on reaching a hydroplaning depth of 4 mm for a cross slope of 1%. Lane width is assumed 3.25 m.

Table 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). Figure 7 shows many configurations with the full hydroplaning limit of 4 mm, 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 7). It should be noted that increasing cross slope by the same percent produces different outcomes depending on the transverseness of the reference slope by 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; Figure 4; Figure 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.

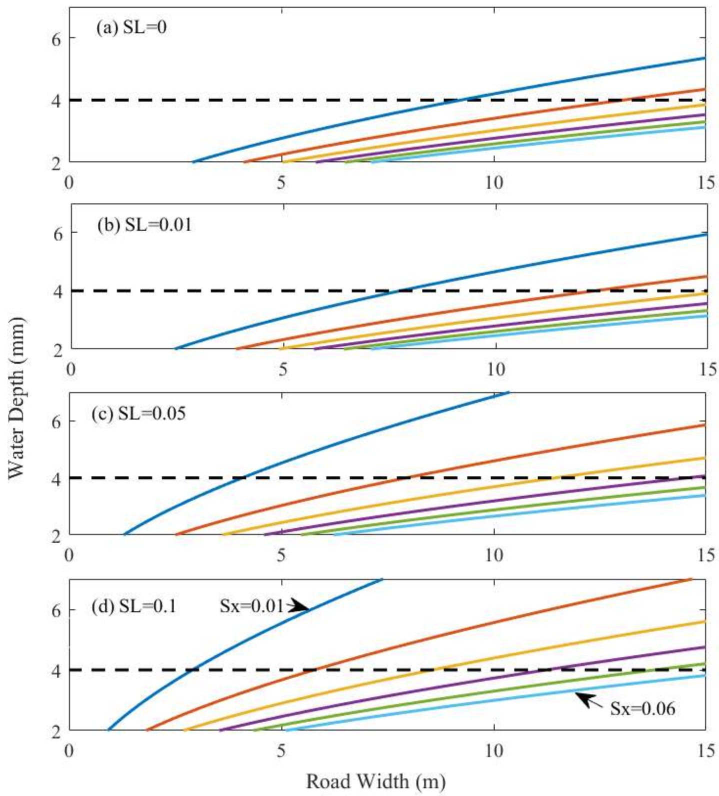

Figure 7 - Water depths profiles for 250 mm hr^{-1} rainfall intensity, a 4 mm depth marking is used for ease in interpretation.Each color represents a different cross slope (0.01 to 0.06), with equal increments.

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 [6] (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. [8] considered the tread depth to be much deeper, but for this study, standard depth was considered in deciding hydroplaning standards.

4. 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 the present work revealed that for up to 6% cross slopes show that increase in cross slope results in diminishing reduction effects on water depths. Overall, the present work has shown that the cross slope optimization was a safe way to avoid hydroplaning depths for grades up to 10% and widths up to 15m 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; increasing cross slopes have diminishing reduction effects on water depths. Furthermore, the safest slope values are coupled with design intensities without hydroplaning threat beyond which cross slope adjustment is required to prevent hydroplaning because of shortened flow paths, and hence, water depths. With a zero-cross slope should not be used since then roads act as channels, not diverting the flow to the sides, and constantly cause an increase in the flow depth until sag is reached.

Symbols

α : the angle between resultant slope and the roadway centreline, $\sin^{-1}\left(S_x / S\right)$

b : the width of the road, m (ft.)

I : rainfall intensity, in mm hr^{-1} (in. hr^{-1})

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

K_M : unit conversion factor (1 for SI and 1.486 for US units)

K_T : a coefficient equal to 6.92 (0.933) in SI (US)

L : flow length m (ft.)

m : $5/3$

n : Manning's roughness coefficient

S : energy grade line m m^{-1} (ft. $ft.^{-1}$), equal to $\left(S_x{}^2 + S_L{}^2\right)^{1/2}$ (Figure 1B)

S_L : road grade m m^{-1} (ft. $ft.^{-1}$)

S_x : cross slope m m^{-1} (ft. $ft.^{-1}$)

q : unit discharge m^2 s^{-1} ($ft.^2$ s^{-1})

T_{ti} : sheet flow travel time min

t_{USACE} : travel time for USACE equation min

z : flow depth m (ft.)

$z_{R\&R}$: water depth in mm as reported by the Ross and Russam [11] solution

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