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SCUPPER INTERCEPTION EFFICIENCY

MISSOURI HIGHWAY AND TRANSPORTATION DEPARTMENT
UNIVERSITY OF MISSOURI - ROLLA
FEDERAL HIGHWAY ADMINISTRATION

SCUPPER INTERCEPTION EFFICIENCY

FOR

MISSOURI HIGHWAY AND TRANSPORTATION DEPARTMENT

BY

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EXECUTIVE SUMMARY

Public safety on state highways is an important issue. If bridge drainage systems (scuppers) are not properly designed, then accidents will occur. The hydraulic interception efficiency and flow capacity of various shaped scuppers are determined for bridge deck drainage design. Examined scupper shapes are circular and rectangular sections. Parallel and perpendicular orientation to the flow direction of the longest dimension of the rectangular section and recessing (with and without grout) below the pavement were examined. Variables affecting the fraction of gutter flow removed by the scupper (scupper efficiency), and flow capacities; and the basic form of the relationship are explicitly developed for each configuration.

Hydraulic interception efficiency and flow capacity for the various scupper configurations are determined for design of bridge deck drainage. Results show that the 4" x 8" rectangular section oriented with the 8" dimension perpendicular to flow provides the greatest interception for small longitudinal slopes and cross slopes. For steeper longitudinal slopes, some flow overshoots the scupper, thus the orientation with the 8" dimension parallel to the flow direction provides the greatest interception for the rectangular configuration. The circular scupper interception capacity lies between the parallel and perpendicular orientations. For smaller longitudinal slopes and cross slopes the circular scupper interception capacity is less than the rectangular section orientation with the 8" dimension perpendicular to flow and more than the parallel orientation. For steeper longitudinal and cross slopes the circular scupper interception capacity is greater than the rectangular section orientation with the 8" dimension perpendicular to the flow direction and less than the parallel orientation. Generally, recessing the 4" x 8" rectangular section orientationed with the 8" dimension perpendicular to the flow direction increases capacity for smaller slopes but decreases capacity for steeper slopes.

The FHWA method on tangent slopes less than 0.5% estimates greater scupper spacing than is necessary for adequate drainage. At 0.1% the difference is substantial while the difference at 0.5% is inconsequential. For tangent slopes less than 0.5%, the effects of unsteady flow should be considered when spacing scuppers. For tangent slopes greater than 0.5%, the FHWA scupper spacing method may be used with error less than 7.0%. A "C" value of unity should be used with the rational method for more accurate modeling the runoff. The I-D-F method, with modifications, should be used to calculate rainfall intensity for subsequent scuppers. If the first scupper for vertical curve slopes is located on a slope of less than 0.5%, then unsteady, nonuniform flow effects should be considered when spacing scuppers (Hesterberg, 1994).

Neglecting the economics, maintaince, and other engineering aspects of design (considering only the hydraulic capacity) associated with the various shapes studied, it recommended that the 4" x 8" rectangular section oriented with the 8" dimension perpendicular or circular section be used for small longitudinal and cross slopes. For steeper longitudinal slopes the rectangular section orientation with the 8" dimension parallel to flow or circular section should be used. In either case the circular section gives less interception capacity than the rectangular section if the appropriate rectangular orientation for a given slope is used. When using the circular section the slopes are not a consideration.

INTRODUCTION

SUMMARY

This report by the University of Missouri-Rolla (UMR) Department of Civil Engineering contains the hydraulic interception efficiency and flow capacity of the bridge deck drain design used by the Missouri Highway and Transportation Department (MHTD). Hydraulic interception efficiency is defined as the fraction of gutter flow removed by the bridge deck drain (scupper) expressed in percentages. This research ascertained the hydraulic interception efficiency for scuppers of a circular and a 4" x 8" rectangular section. To analyze the rectangular section, parallel and perpendicular orientation of the eight-inch dimension to the direction of flow were both used. Also, the rectangular scupper was recessed three-quarters of an inch below the pavement to improve hydraulic interception efficiency. A further attempt to improve efficiency was to grout a smooth transition to the recessed scupper. These configurations were built and hydraulic interception efficiency was determined. Variables affecting the fraction of gutter flow removed by the scupper, scupper efficiency, and the basic form of the relationship were all determined. These measures of interception efficiency are consistent with and can be used with procedures shown in Federal Highway Administration Report No. FHWA/RD-87/014, "Bridge Deck Drainage Guidelines".

Public safety on state highways is an important focus in this report. If bridge drainage systems (scuppers) are not properly designed, then hydroplaning can occur on bridges during a summer rainstorm, resulting in accidents. During the winter, if scuppers do not drain excess water from the bridge deck, then water will freeze, resulting in accidents.

SCUPPER DESIGN

Drainage is an important issue for bridge designers in regard to both safety and structures. Cost of drainage structures, reinforcement interruption in the bridge deck, and maintenance expense must be considered. Bridge designers must prevent hydroplaning while minimizing drainage structure costs and reinforcement interruption in the bridge deck.

Drainage design and theory have changed dramatically with the evolution of highway engineering. In the past, roads served as the conveyance mechanism for any water they collected. This approach meant keeping the water confined to the edge of the roadway until a suitable drainage location was found. As roadways carried greater volumes of vehicles at increasing speeds, it was necessary to reduce the amount of water on the road and thus prevent hydroplaning. Because higher speed intensified hydroplaning, designers had to decrease the amount of ponding on driving surfaces. Highways were crowned to increase the flow of water off the roadway, and ditches were constructed to collect and convey the water.

Bridge drainage presents problems beyond those of normal highway drainage design. Limited area makes transporting the water away from the pavement a difficult task. Early bridge drainage

consisted simply of slots cut into the sides of the concrete curbing. These slots were subject to corrosion by deicing agents which over time exposed reinforcing steel. Modern bridge deck drainage incorporates scuppers, which are vertical drains typically located on the shoulder of the bridge deck. Scuppers are constructed of steel or corrosion resistant material. They have the advantage over side slots of being replaceable and thus protecting the nearby reinforcing steel. Scuppers direct water away from the bridge superstructure, thereby preventing its corrosion from deicing agents. By means of a piping system, scuppers may also be used to divert water when discharge directly below the structure is prohibited. Disadvantages of scuppers include: clogging, regular maintenance, and interruption of bridge reinforcement for scupper placement.

Interruption of reinforcement near scuppers is a major design concern. Bridge designers must keep this to a minimum while providing adequate drainage. By minimizing the number of scuppers required, bridge designers decrease reinforcement interruption, initial expenses, and maintenance costs. A need thus exists to find the optimal spacing of scuppers to ensure protection from hydroplaning as well as reduceing costs and reinforcement interruptions.

Current design procedure for the placement of scuppers is presented by the Federal Highway Administration in *Bridge Deck Drainage Guidelines*. FHWA uses the rational method for calculating runoff from the bridge deck and a uniform flow equation to calculate where a scupper is needed based on the given design spread (allowable width of encroaching water). This method uses a single time of concentration (t_c) for the entire bridge, but states that a more accurate method would find a time of concentration for each scupper. Nonetheless, their guidelines do not offer a method for determining intermediate times of concentration. Hydraulic Engineering Circular No. 12 (HEC-12), *Drainage of Highway Pavements*, does give the amount of water removed by each scupper for certain configurations of scuppers.

For Missouri bridges, MHTD changed the dimensions of scuppers in order to minimize the effect on reinforcement placement. HEC-12 does not state the amount of flow intercepted with these dimensions. MHTD contracted with UMR to determine flow interceptions of the revised scupper dimensions under different slope conditions. A full-scale bridge deck model was constructed to make this determination for various configurations (orientation and shape).

A hydrodynamic computer program was used to determine the appropriate spacing of scuppers for bridge deck drainage. It includes kinematic sheetflow and dynamic gutter routing. This program was developed to answer two questions that arise when using *Bridge Deck Drainage Guidelines*. First was use of the rational method to determine scupper spacing. Through the computer program, it was shown that the rational method is inadequate for some conditions. Second was dubious use of a single time of concentration for the entire bridge, with a selected design return period to ascertain rainfall intensity, for subsequent calculation of scupper spacing. Through the computer program rainfall intensity values were based on the time of concentration for each scupper, not a single time of concentration for the entire bridge.

EXPERIMENTAL APPARATUS

A full-scale model was constructed and housed in the hydraulic laboratory located in the civil engineering building at UMR. This model consisted of a section of a bridge deck in which the scupper was located as shown in Figure 1 and Pictures 1 through 6. Slope of the gutter as well as the bridge deck was adjusted to duplicate desired conditions. The model's upstream control gate simulated the upstream boundary conditions of flow across the bridge deck and gutter flow next to the edge (parapet or curb).

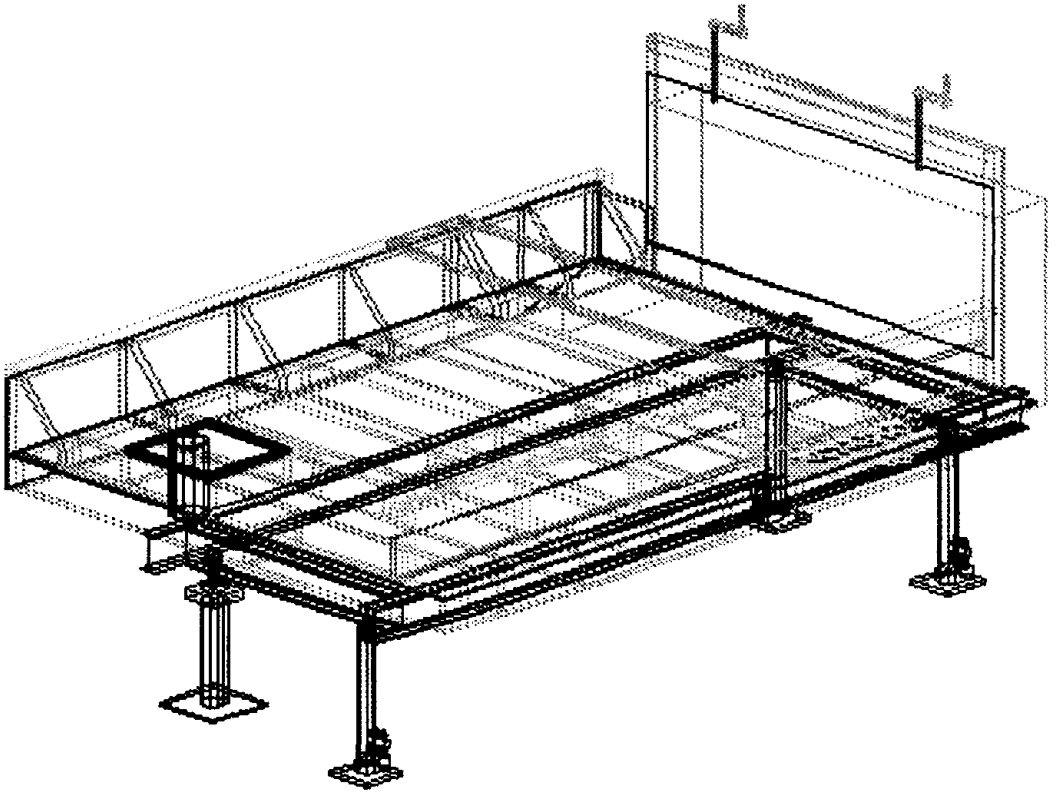
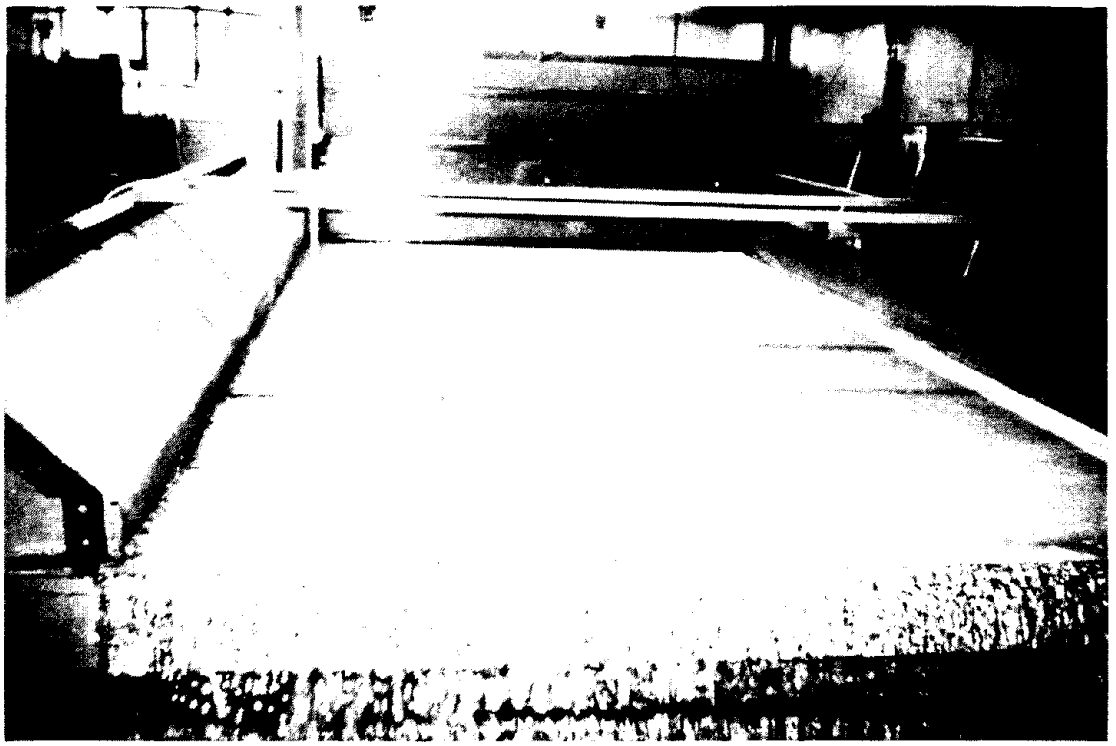
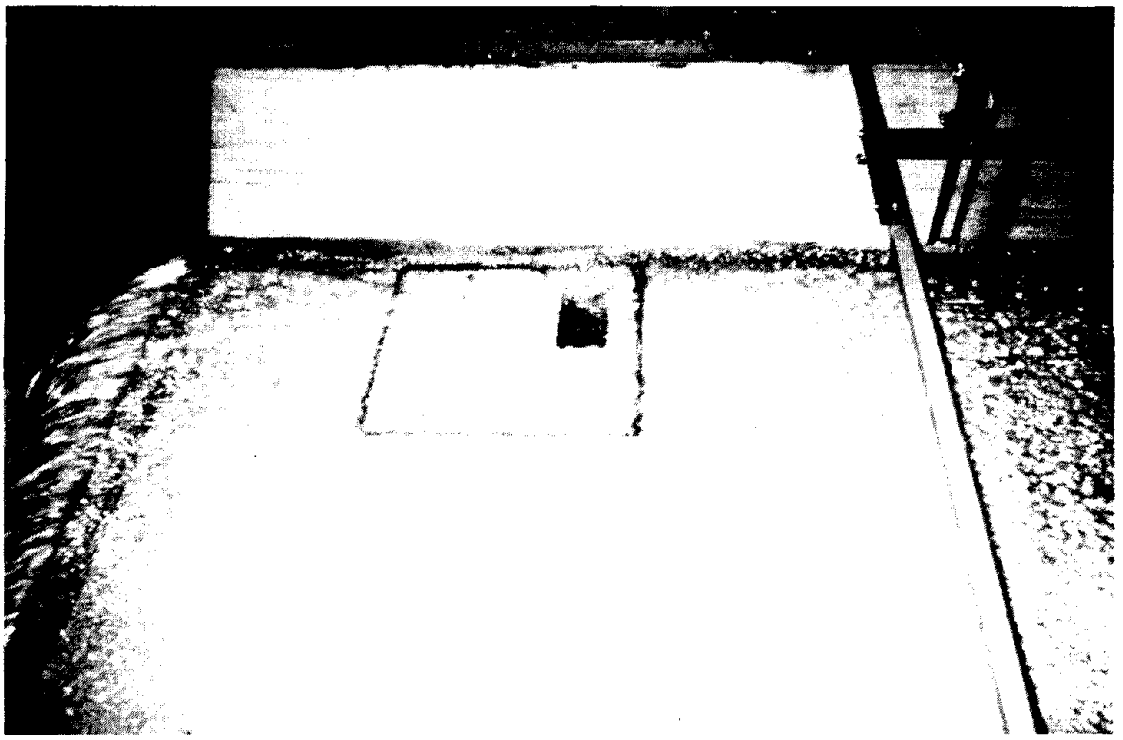


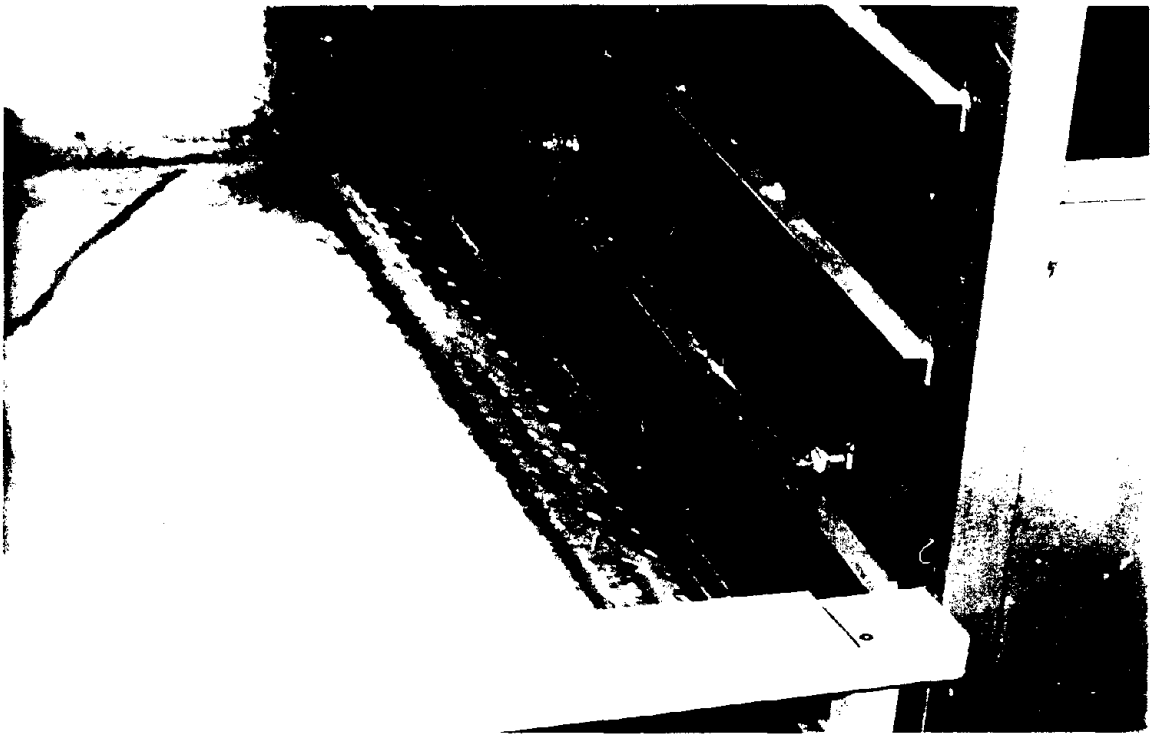
Figure 1



Bridge Deck Model (looking upstream)
Picture 1



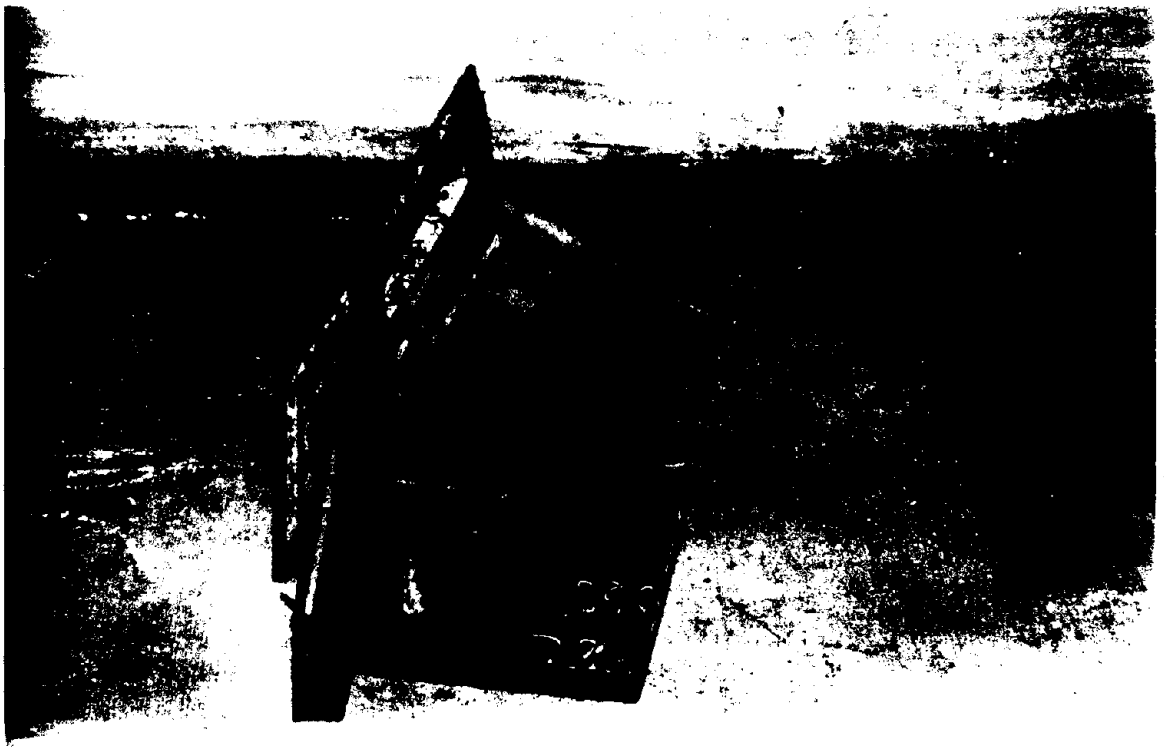
Scupper in Model (perpendicular placement)
Picture 2



Bridge Deck Model Head Gate
Picture 3

Circular Scupper in Model
Picture 4

Scupper Inserts
Picture 5



Scupper Inserts
Picture 6

As noted, an upstream hydraulic control gate served to produce a flow similar to that occurring on an actual bridge deck. Manning's equation was used to estimate upstream flow boundary condition values of velocity and depth for gutter flow under given conditions. These values are then replicated in the bridge deck model by operation of the control gate. When the flow was thus controlled as it entered the hydraulic model, it was maintained at proper depth and velocity in the gutter for slope and roughness of the bridge deck. Flow to the model upstream of the scupper was measured as was flow downstream from the scupper. Flow going down the scupper was also measured. From these measurements, scupper efficiency can be determined and checked.

Wood, concrete and steel comprised the model, resulting in a pavement surface 6' wide by 11' long (Figure 1). Longitudinal slopes vary from 0.5% to 10% and cross-slopes vary from 1% to 6%, including the standard cross-slope used by MHTD of 1.56% (3/16":1'). Flow rates and depths on the pavement surface were controlled by a valve and a sluice gate. Verification of depths and velocities of flow were made approximately 2' upstream of the scupper.

Scupper flow capacities were measured with the edge of the scupper being located 2" from the face of the curb. Two basic scupper geometries were employed. First was a circular scupper with an interior diameter of 6" and the opening was flush with the surface of the pavement as shown in Figure 2. Second was a rectangular, steel tubing, with exterior dimensions of 4" x 8", and a wall thickness of 1/4". Four configurations were implemented with this scupper: two flush with the surface of the concrete; one of the two having its long side parallel to the face of curb as shown in Figure 3 and the other with the long side perpendicular to the face of curb as shown in Figure 4; two recessed 3/4" below the surface of the pavement, the recess area was 1.5 ft by 1.5 ft, and the long side perpendicular to the face of curb. For these recessed scuppers, one was exposed while the other was grouted from the pavement surface to the rim of the scupper opening, creating a slope to the scupper from all sides as shown in Figure 5. The scupper was located 10 inches downstream of the upstream edge of the recessed area.

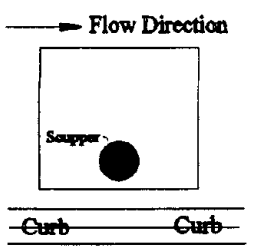


Figure 2

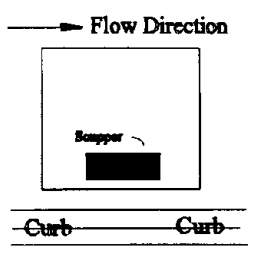


Figure 3

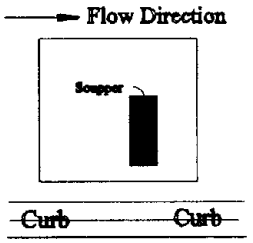


Figure 4

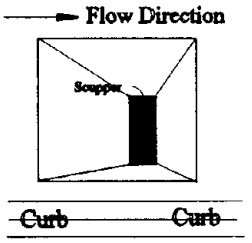


Figure 5

Scupper discharges were determined by measuring the change of weight of the water with respect to time. These values were compared with the change in head on a V-notch weir discharging from a holding tank which collected the water at the end of the model.

Plots of the data were prepared of scupper efficiency and capacity. Expressions were then fit to the data. Equations were selected to yield the best fit. In some cases a curve and equation was fit to just two data points because the general trend of the data can be observed from other plots on the same graph.

SCUPPER INTERCEPTION EFFICIENCY AND CAPACITY

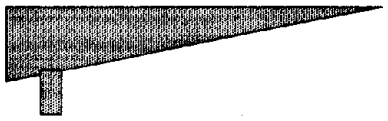
Scupper interception efficiency "E" is that fraction of the gutter flow removed by the scupper. *Bridge Deck Drainage Guidelines* gives an approximation for "E" for small grates and low gutter velocities.

$$E = 1-[1-\frac{w}{T}]^{2.67} \qquad (1)$$

w is the width of deck drainage structure's cross-sectional projected width (ft) normal to the flow and T is design spread (ft). If this equation is applied to a 4" x 8" rectangular section, parallel and perpendicular orientation of the eight-inch dimension to the direction of flow, "E" equals 27% for the perpendicular and 14% for the parallel for a six-foot shoulder. If this equation is applied to 4" x 8" rectangular section, perpendicular orientation of the eight-inch dimension to the direction of flow, "E" equals 17% for a ten-foot shoulder. Although these values are less precise than those determined by this research, examination of appropriate figures shows that both sets values equate reasonably well.

From measured values, it was determined that the controlling factors in scupper efficiency are depth and velocity. This determination is directly related to longitudinal and cross-slope of the bridge. Measured values for the 4" x 8" rectangular, flush mounted, perpendicular to flow, Figure 6 demonstrates that efficiency differs for the same longitudinal slope. For this reason, it was decided to determine scupper capacity directly from slope and depth at the scupper as well as from scupper efficiency.

0.15 ft depth of water over scupper



Q scupper: 0.23 cfs

Spread: 6 ft

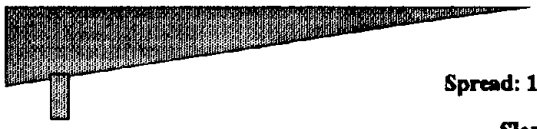
Slope: 0.02

X-Slope: 0.0273

Q total: 1.48 cfs

Efficiency: 15.5%

0.15 ft depth of water over scupper



Q scupper: 0.25 cfs

Spread: 10 ft

Slope: 0.02

X-Slope: 0.0158

Q total: 2.37 cfs

Efficiency: 10.6%

Percent difference in capacity: 8%

Percent difference in efficiency: 32%

Figure 6

SCUPPER INTERCEPTION EFFICIENCY

It was found that the supper efficiency, E_S, could be closely approximated by the equation

$$E_S = a\ S^b \tag{2}$$

where S is longitudinal bridge slope, and **a** and **b** are coefficients dependent on the bridge cross-slope. Expressions were then fit to measured data for each cross-slope to determine the values of **a** and **b** in the above equation. Equations that produced the best fit were selected; values are shown in Tables 1 through 6. Data collected and resulting plots are shown in Figures 3 through 8.

Scupper configuration: rectangular 4"x 8" (flush with deck surface with orientation of 8" dimension perpendicular to face of curb) with 6-foot shoulder

Cross-Slope	**a**	**b**	Coefficient of Determination
0.010	14.58	-0.18	0.89
0.016	6.67	-0.34	0.90
0.020	3.55	-0.45	0.94
0.030	2.08	-0.50	0.99
0.040	2.08	-0.44	0.99
0.050	3.68	-0.28	0.99

Table 1

Scupper configuration: rectangular 4" x 8" (recessed 3/4" below deck surface and grouted with orientation of the 8" dimension perpendicular to face of curb) with 6-foot shoulder

Cross-Slope	**a**	**b**	Coefficient of Determination
0.010	9.47	-0.36	0.99
0.016	5.22	-0.44	0.98
0.020	4.05	-0.45	0.99
0.030	3.12	-0.42	0.99
0.040	2.66	-0.40	0.99
0.050	4.95	-0.23	0.99

Table 2

Scupper configuration: rectangular 4" x 8" (recessed 3/4" below deck surface with orientation of the 8" dimension perpendicular to face of curb) with 6-foot shoulder

Cross-Slope	**a**	**b**	Coefficient of Determination
0.010	7.89	-0.41	0.95
0.016	2.83	-0.58	0.97
0.020	1.77	-0.64	0.99
0.030	1.28	-0.62	0.99
0.040	1.11	-0.58	0.92
0.050	0.86	-0.57	0.99

Table 3

Scupper configuration: circular 6" diameter (flush with deck surface) with 6-foot shoulder

Cross-Slope	**a**	**b**	Coefficient of Determination
0.010	12.93	-0.17	0.87
0.016	8.06	-0.27	0.90
0.020	5.10	-0.35	0.89
0.030	3.60	-0.38	0.98
0.040	2.65	-0.39	0.99
0.050	3.88	-0.28	0.99

Table 4

Scupper configuration: rectangular 4"x 8" (flush with deck surface with orientation of the 8" dimension parallel to face of curb) with 6-foot shoulder

Cross-Slope	**a**	**b**	Coefficient of Determination
0.01	9.17	-0.23	0.94
0.016	7.06	-0.28	0.90
0.020	5.62	-0.32	0.91
0.030	4.67	-0.32	0.94
0.040	3.06	-0.37	0.99
0.050	3.66	-0.30	0.99

Table 5

Scupper configuration: rectangular 4"x 8" (flush with deck surface with orientation of the 8" dimension parallel to face of curb) with 10-foot shoulder

Cross-Slope	**a**	**b**	Coefficient of Determination
0.01	1.78	-0.53	0.87
0.016	1.97	-0.45	0.99
0.020	1.50	-0.46	0.99

Table 6

Scupper configuration: rectangular 4"x 8" (flush with deck surface with orientation of the 8" dimension perpendicular to face of curb) with 6-foot shoulder

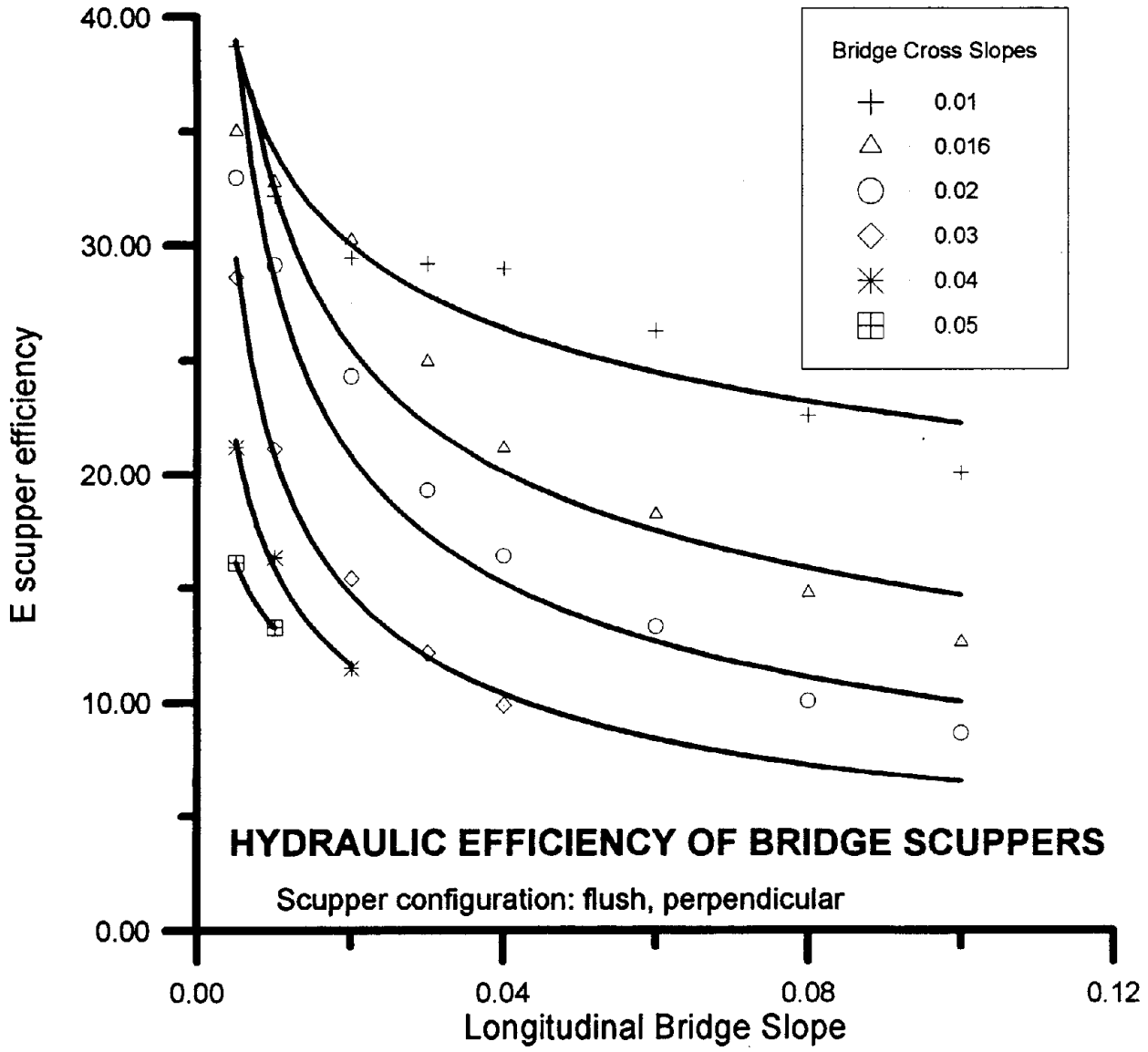


Figure 3

Scupper configuration: rectangular 4" x 8" (recessed 3/4" below deck surface and grouted with orientation of the 8" dimension perpendicular to face of curb) with 6-foot shoulder

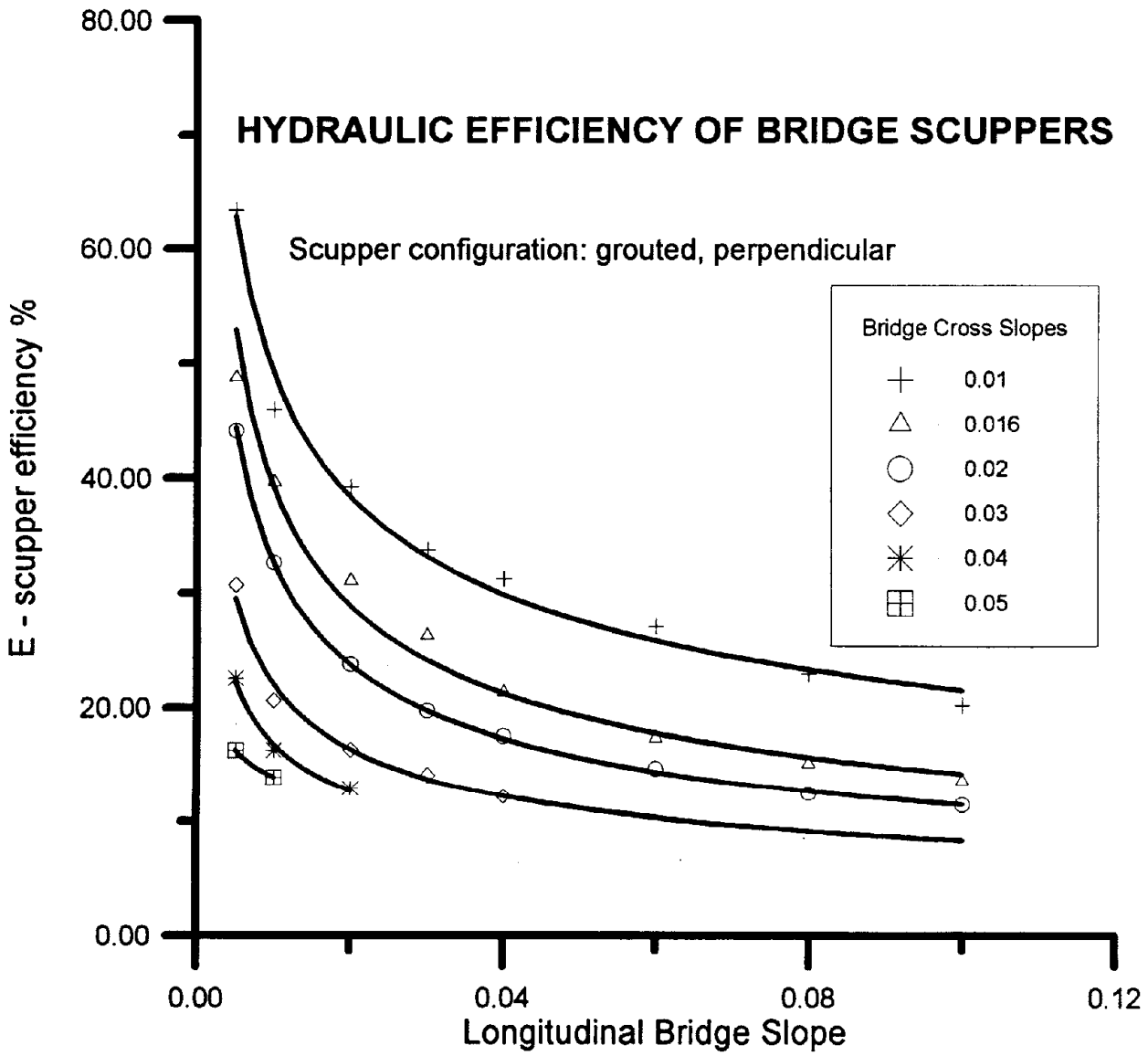


Figure 4

Scupper configuration: rectangular 4" x 8" (recessed 3/4" below deck surface with orientation of the 8" dimension perpendicular to face of curb) with 6-foot shoulder

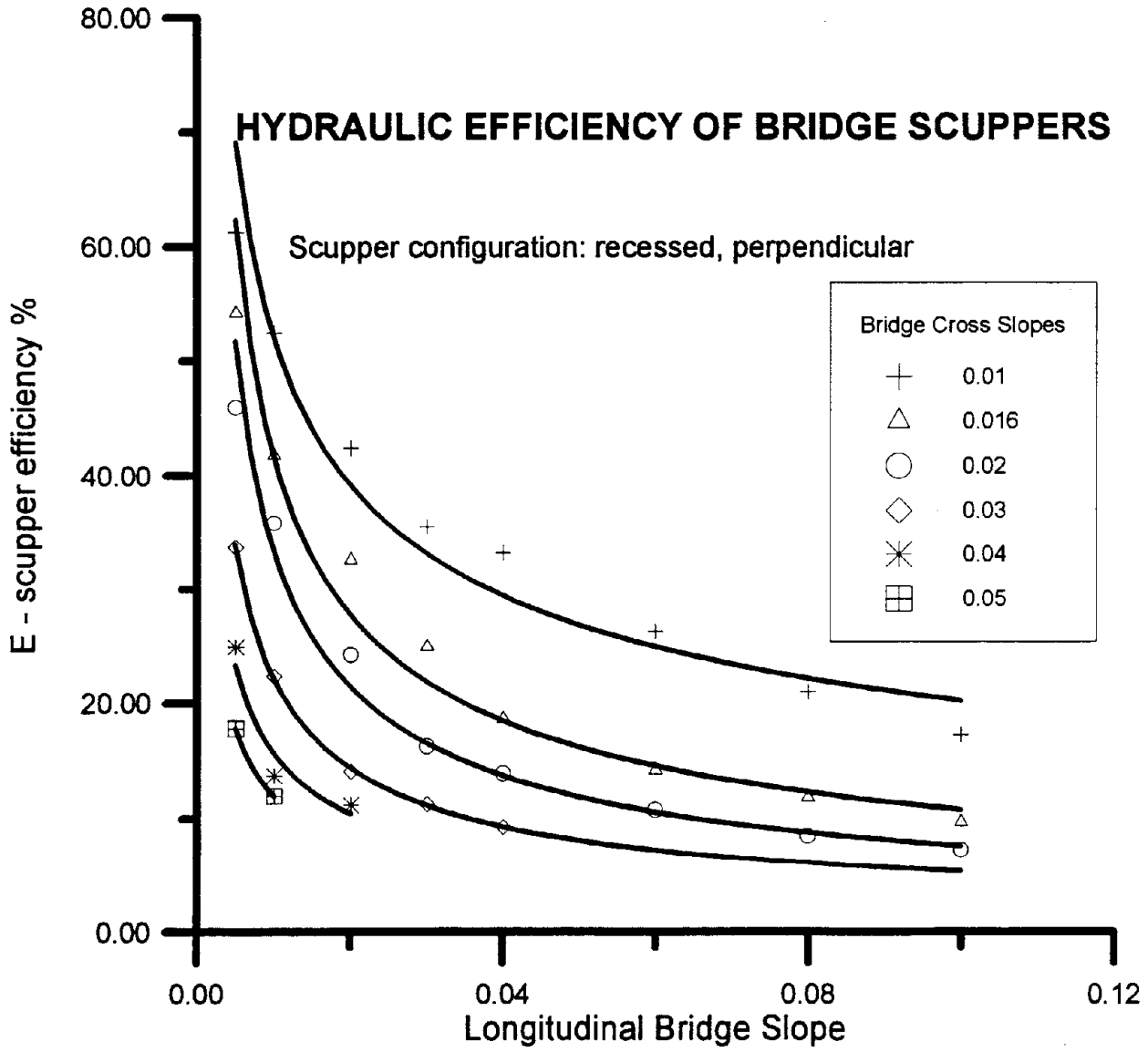


Figure 5

Scupper configuration: circular 6" diameter (flush with deck surface) with 6-foot shoulder

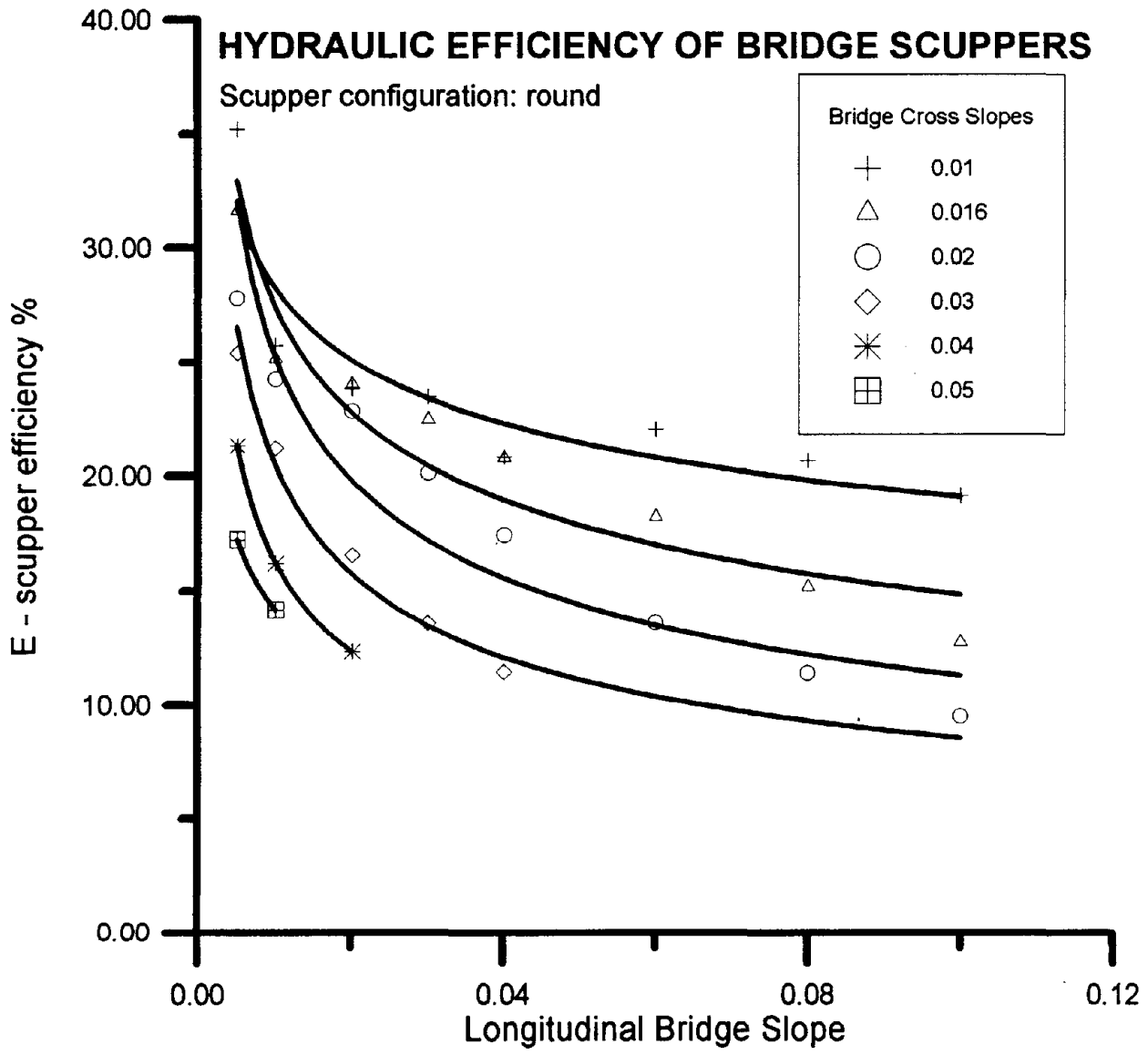


Figure 6

Scupper configuration: rectangular 4"x 8" (flush with deck surface with orientation of the 8" dimension parallel to face of curb) with 6-foot shoulder

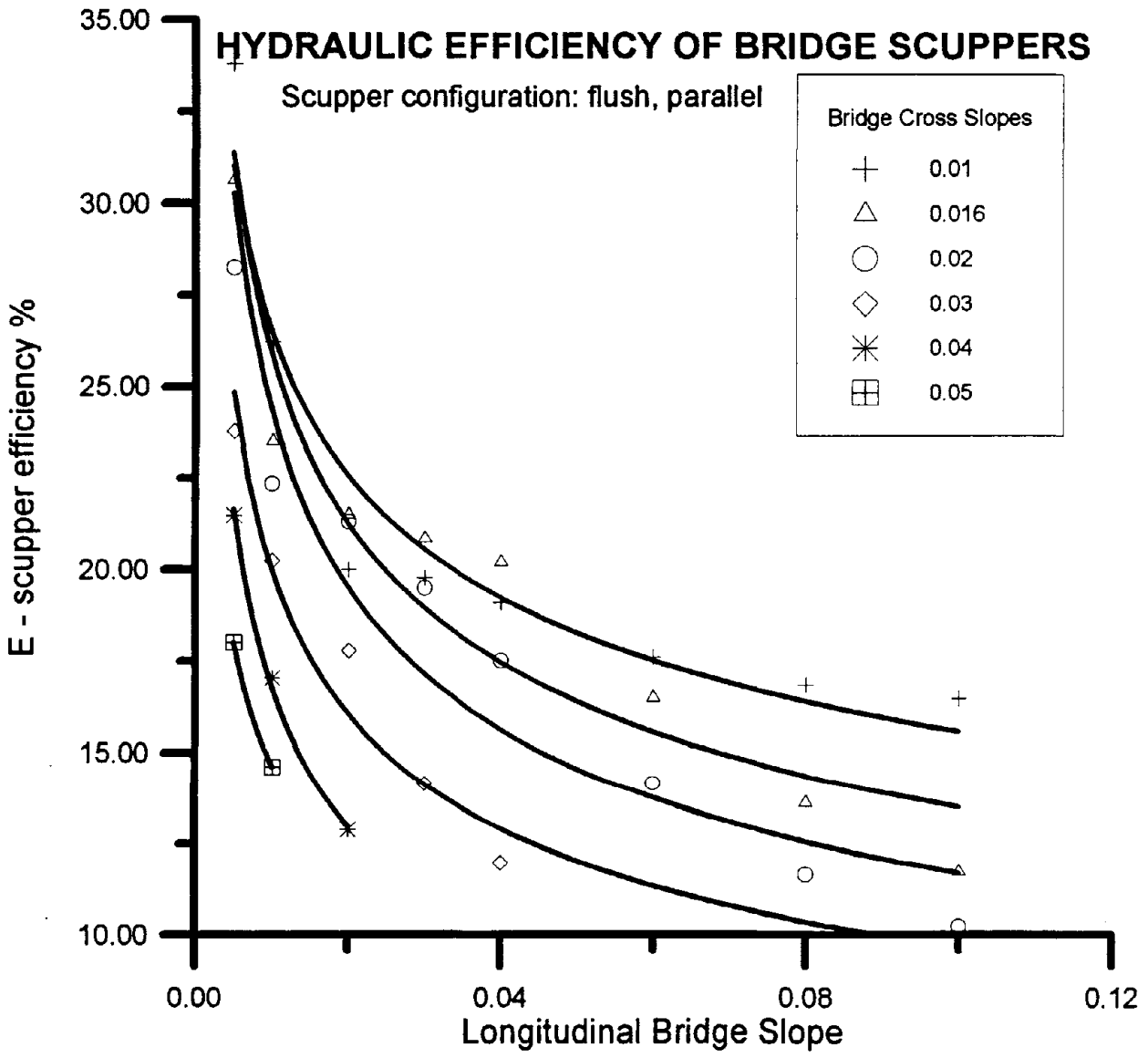


Figure 7

Scupper configuration: rectangular 4"x 8" (flush with deck surface with orientation of the 8" dimension parallel to face of curb) with 10-foot shoulder

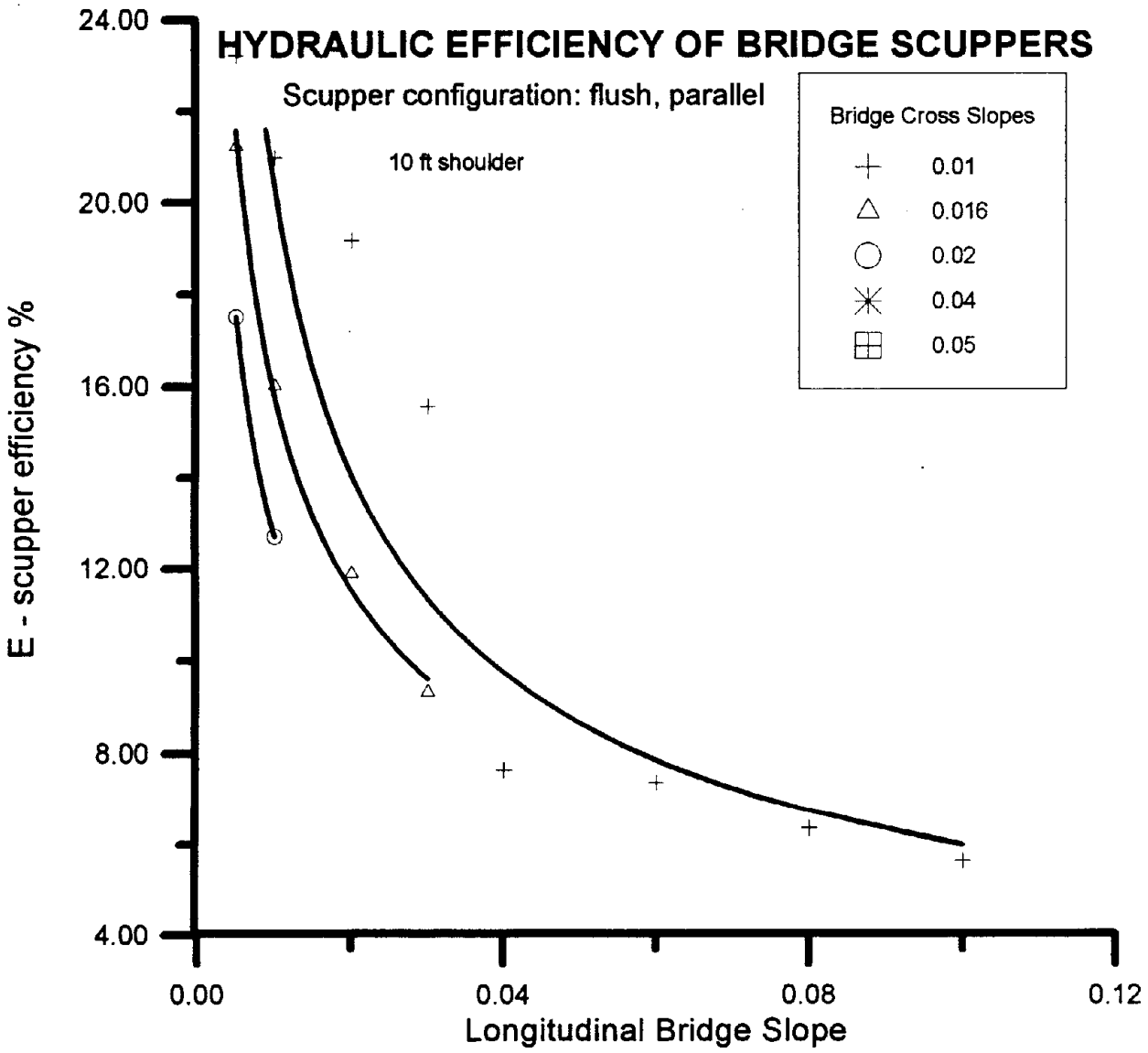


Figure 8

SCUPPER INTERCEPTION CAPACITY

It was found from log-log plots that scupper capacity, $\mathbf{Q_S}$, could be closely approximated by the equation

$$Q_S = a\ H^b \tag{3}$$

where **H** is the depth 6" from the face of curb near the center of the scupper. At lower slopes, the scupper acted more as a weir, with **b** equal to 3/2. As the slope and the amount of water increased, values of scupper capacity were more closely expressed by an orifice equation where **b** is equal to 1/2.

Values of $\mathbf{Q_s}$ were determined by measuring change of weight with respect to time. These values were compared with the change in head of a V-notch weir discharging from a holding tank at the end of the model.

For standard design conditions of $\mathbf{S_x}$=3/16":1', **T**=4', 6', 8', 10' & 12', and **n**=0.015, a design graph of scupper capacity vs. slope is supplied for each configuration as shown in Figures 9 through 13. Most of the plots shown were calculated using the caculated depth for the conditions. Figures 14 through 28 were used to determine the scupper capacity for those depths. The symbols represent measured data where the diamonds are the 12 foot top width; circles are the 6 foot top width; and triangles are the 4 foot top width. All other plots were calculated from depth capacity results shown in Figures 14 through 28.

Log-log plots of scupper capacity vs. depth were prepared , and relationships were established. Expressions were then fit relating longitudinal slope to values of **a** and **b** in Eq. (3). In order to make substitutions for different values of Manning's **n** and widths of runoff, **T,** equations with the best fit and with S raised to the power of 1/2 were selected.

For each of the scuppers and configurations a pair of equations was established. They vary for low (weir flow) and high (orifice flow) depths as just discussed. Depths described as low or high are relative for each slope, and the change from one equation to the other occurs at varying depths for different slopes. To find which equation is appropriate for the scupper's design depth, consult Figure 14 through 28, or refer to both equations and take the lower value of scupper capacity. Design equations for each scupper's geometry and configurations are as follows.

H is the depth of gutter flow 0.5 feet from the face of the curb and **a** and **b** are equal to:

Scupper configuration: rectangular 4"x 8" (flush with deck surface with orientation of the 8" dimension perpendicular to face of curb) with 6-foot shoulder for low depths less than 0.1 foot or at change in slope on plot

$$a = 2.09 + 28 S^{1/2}$$
$$b = 3/2$$

and for depths greater than 0.1 foot or at change in slope on plot

$$a = 0.6$$
$$b = 1/2$$

Scupper configuration: rectangular 4" x 8" (recessed 3/4" below deck surface and grouted with orientation of the 8" dimension perpendicular to face of curb) with 6-foot shoulder for low depths less than 0.1 foot or at change in slope on plot

$$a = 1.43 + 4.32 S^{1/2}$$
$$b = 1$$

and for depths greater than 0.1 foot or at change in slope on plot

$$a = 1.26 - \frac{0.0242}{S^{1/2}}$$
$$b = 3/4$$

Scupper configuration: rectangular 4" x 8" (recessed 3/4" below deck surface with orientation of the 8" dimension perpendicular to face of curb) for low depths less than 0.1 foot or at change in slope on plot

$$a = 2.59 - \frac{0.0566}{S^{1/2}}$$
$$b = 1$$

and for depths greater than 0.1 foot or at change in slope on plot

$$a = 0.75 - 0.98 S^{1/2}$$
$$b = 1/2$$

Scupper configuration: circular 6" diameter (flush with deck surface) with 6-foot shoulder for low depths less than 0.1 foot or at change in slope on plot

$$a = 1.45 + 28S^{1/2}$$
$$b = 3/2$$

and for depths greater than 0.1 foot or at change in slope on plot

$$a = 0.6$$
$$b = 1/2$$

Scupper configuration: rectangular 4"x 8" (flush with deck surface with orientation of the 8" dimension parallel to face of curb) with 6-foot shoulder for low depths less than 0.1 foot or at change in slope on plot

$$a = 1.03 + 28S^{1/2}$$
$$b = 3/2$$

and for depths greater than 0.1 foot or at change in slope on plot

$$a = 0.6$$
$$b = 1/2$$

When the Manning's **n** value differs from 0.015, replace **S** in the above equations used to compute **a** with the equivalent value of velocity of a one-foot wide section of flow over the scupper. From the Manning's equation

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} \qquad (5)$$

solve for V where

$$R = A/P = \frac{S_x (T - 0.5)}{1 + (S_x T)} \qquad (6)$$

where **V** is average velocity for a one-foot section of water nearest the curb, **R** is hydraulic radius, and **n** is the Manning's coefficient. Then replace **a** in the above equation(s) as follows.

Scupper configuration: rectangular 4"x 8" (flush with deck surface with orientation of the

8" dimension perpendicular to face of curb) with 6-foot shoulder

$$a = 2.09 + 28\frac{Vn}{1.49R^{2/3}}$$

Scupper configuration: rectangular 4" x 8" (recessed 3/4" below deck surface and grouted with orientation of the 8" dimension perpendicular to face of curb) with 6-foot shoulder for depths less than 0.1 foot or at change in slope on plot

$$a = 1.43 + 4.32\frac{Vn}{1.49R^{2/3}}$$

and for depths greater than 0.1 foot or at change in slope on plot

$$a = 1.26 - \frac{0.0361\,R^{2/3}}{Vn}$$

Scupper Configuration: Rectangular 4" X 8" (recessed 3/4" below deck surface with orientation of the 8" dimension perpendicular to face of curb) for depths less than 0.1 foot or at the change in slope on plot

$$a = 2.59 - \frac{0.0843\,R^{2/3}}{Vn}$$

and for depths greater than 0.1 foot or at the change in slope on plot

$$a = 0.75 - 0.98\frac{Vn}{1.49R^{2/3}}$$

Scupper configuration: circular 6" diameter (flush with deck surface)

$$a = 1.45 + 28\frac{Vn}{1.49R^{2/3}}$$

Scupper configuration: rectangular 4"x 8" (flush with deck surface with orientation of the 8"

dimension parallel to face of curb)

$$a = 1.03 + 28 \frac{Vn}{1.49 R^{2/3}}$$

Equations were selected that produced results with an average error less than 10%. Due to some inaccuracy of lab measurements and to the small values of scupper capacity, 10% error is not significant. The test model was found to have a Manning's **n** value of 0.015. Further investigation is required for values of **n** that vary widely from this. These equations are empirically formulated for scupper dimensions and configurations specified by MHTD. Further investigation is also required for any change in dimensions or alignment of scuppers.

Scupper configuration: rectangular 4"x 8" (flush with deck surface with orientation of the 8" dimension perpendicular to face of curb), cross-slope: 3/16" per foot and n=0.015.

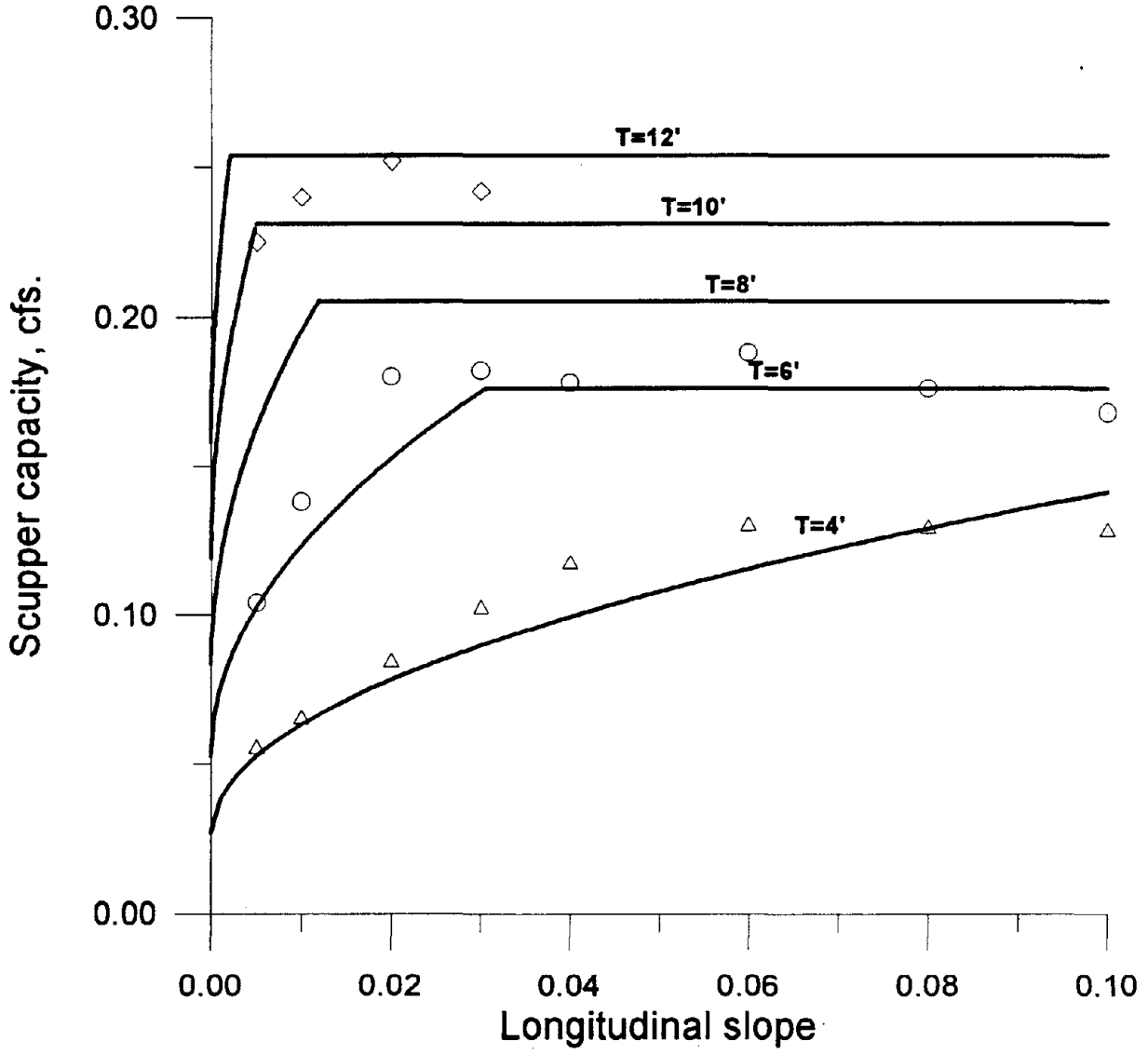


Figure 9

Scupper configuration: rectangular 4" x 8" (recessed 3/4" below deck surface and grouted with orientation of the 8" dimension perpendicular to face of the curb), cross-slope: 3/16" per foot and n=0.015.

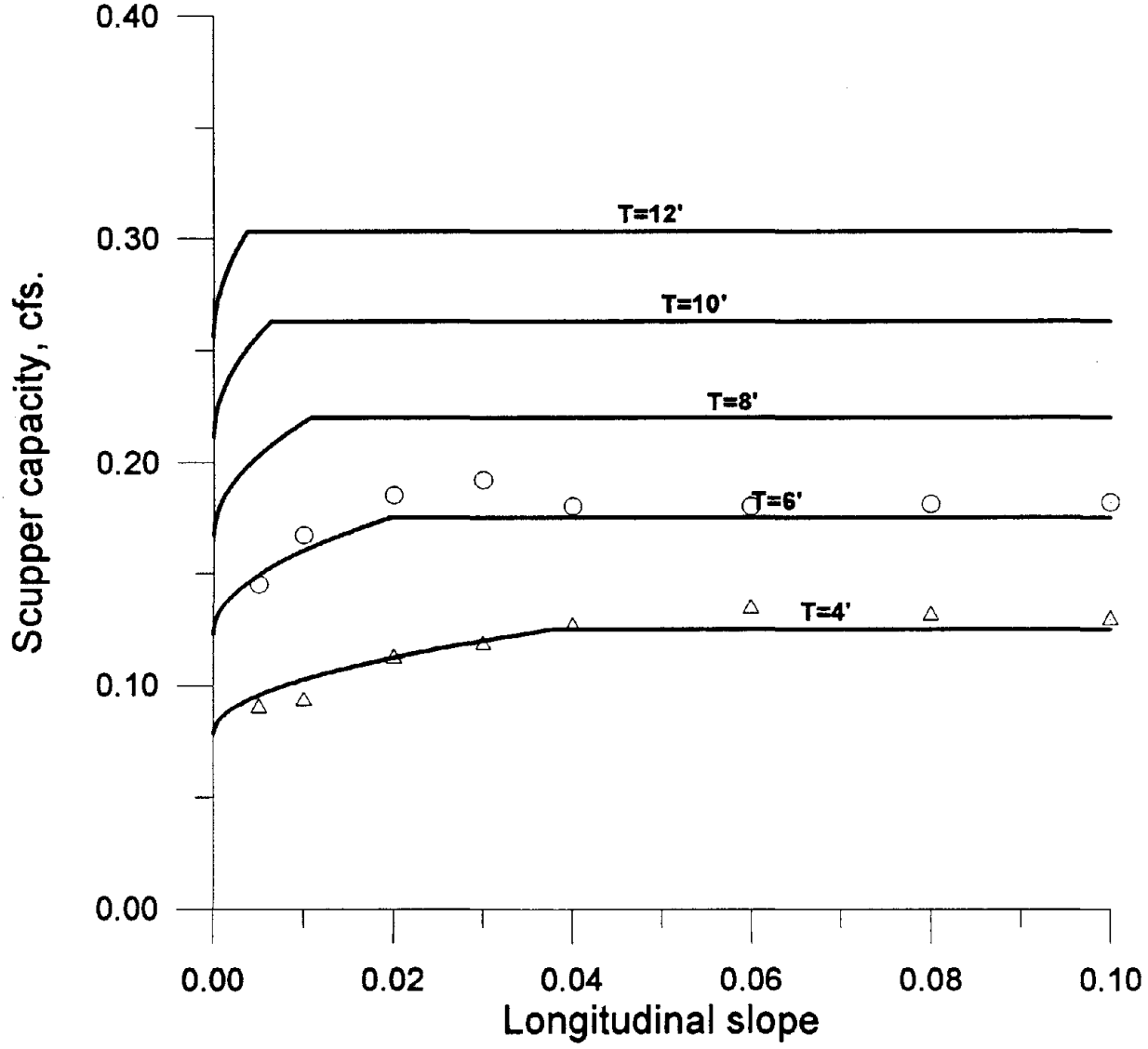


Figure 10

Scupper configuration: rectangular 4" x 8" (recessed 3/4" below deck surface with orientation of the 8" dimension perpendicular to face of curb), cross-slope: 3/16" per foot and n=0.015.

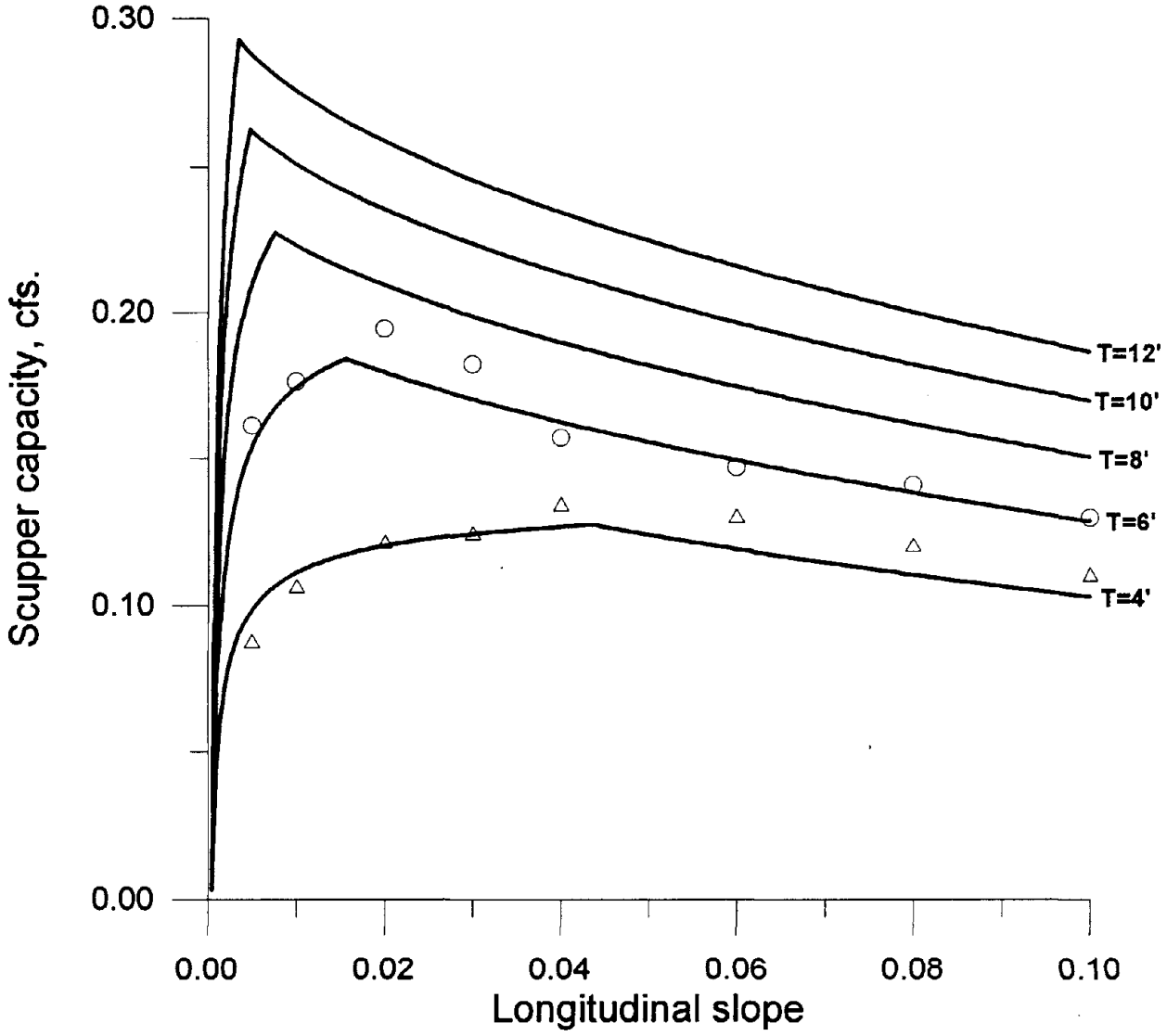


Figure 11

Scupper configuration: Circular 6" diameter (flush with deck surface), cross-slope: 3/16" per foot and n=0.015.

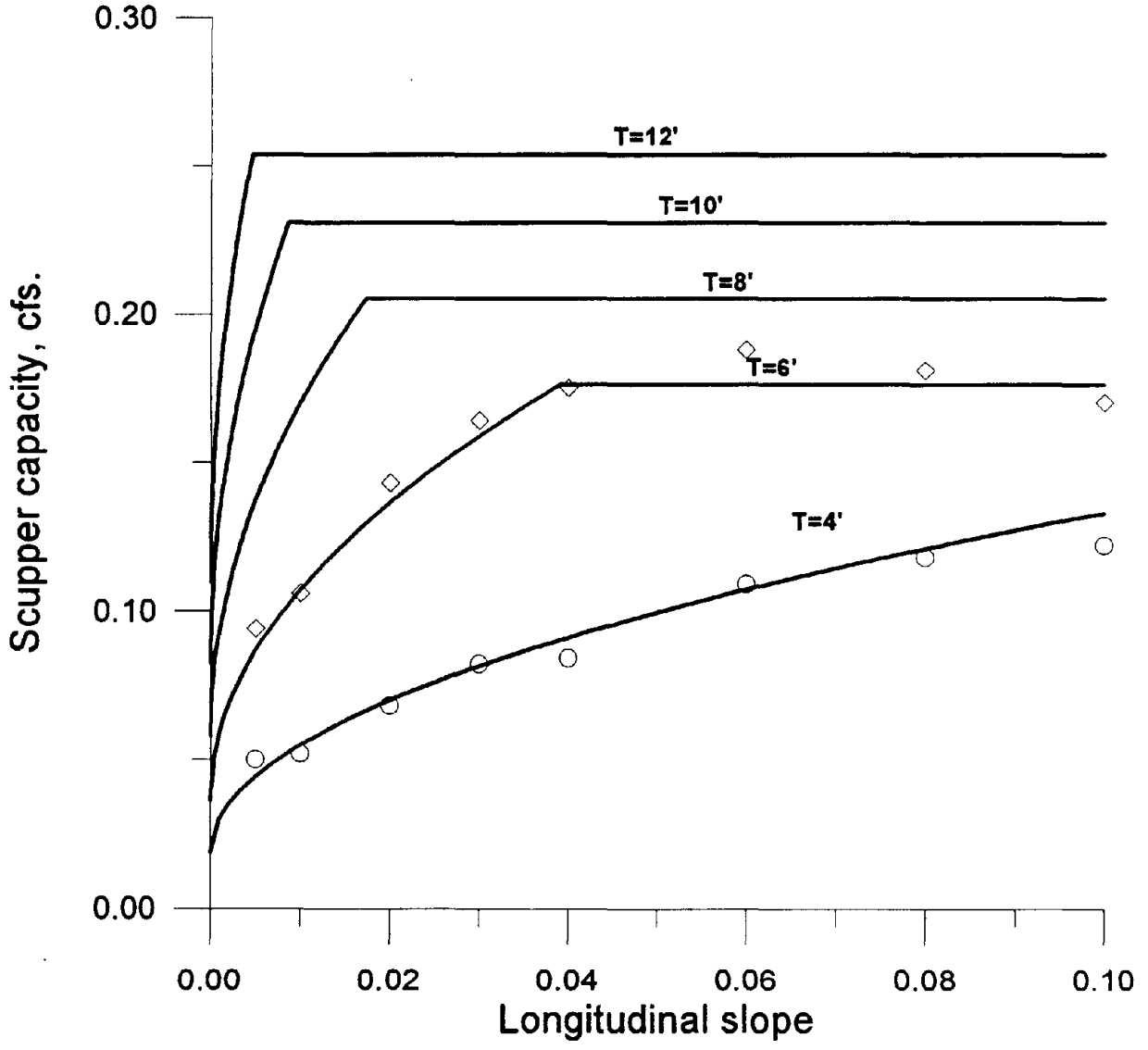


Figure 12

Scupper configuration: rectangular 4"x 8" (flush with deck surface with orientation of the 8" dimension parallel to face of curb), cross-slope: 3/16" per foot and n=0.015.

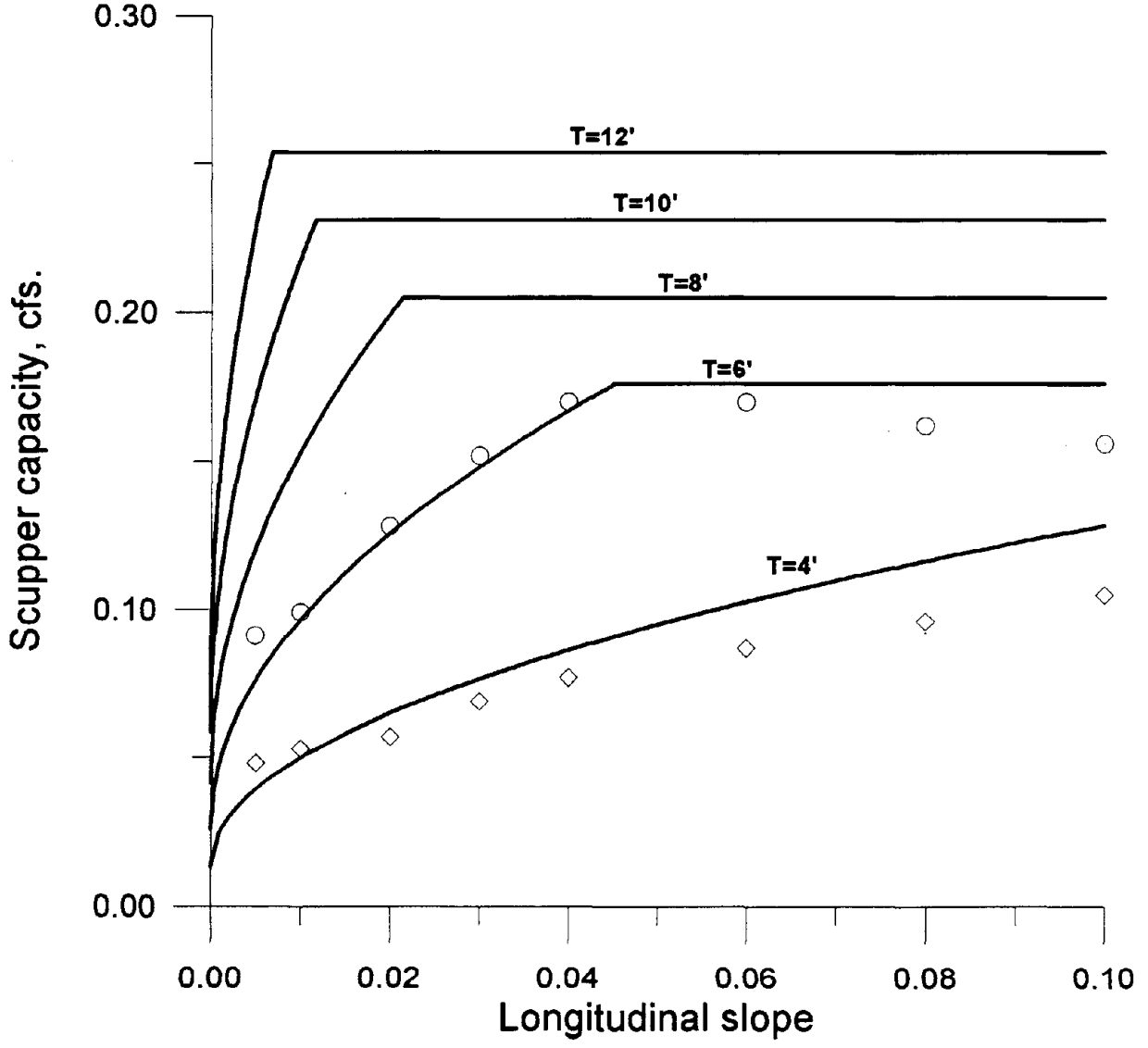


Figure 13

Scupper configuration: rectangular 4"x 8" (flush with deck surface with orientation of the 8" dimension perpendicular to face of curb) with longitudinal slopes: 0.005 to 0.01 and Manning's n=0.015

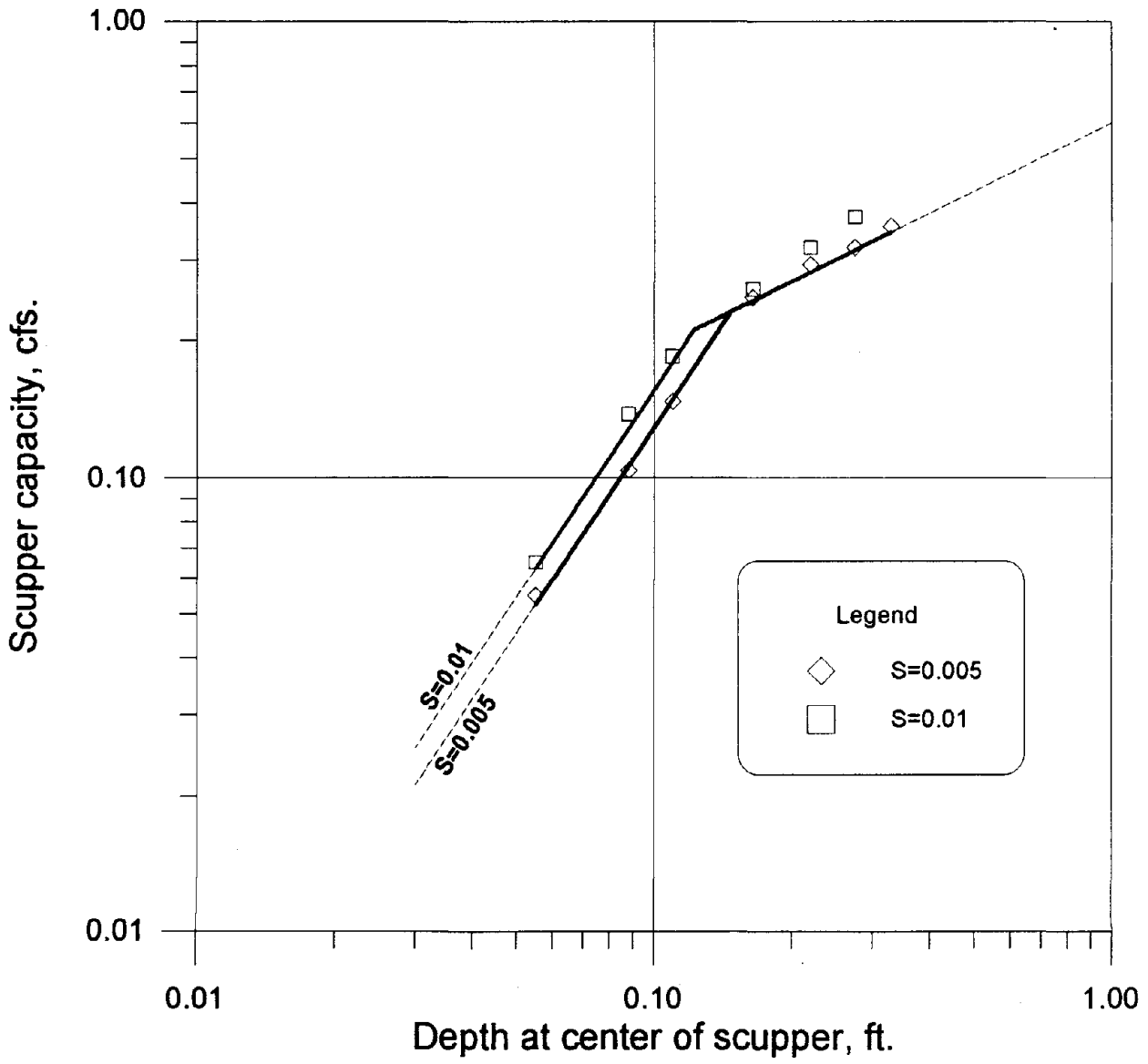


Figure 14

Scupper configuration: rectangular 4"x 8" (flush with deck surface with orientation of the 8" dimension perpendicular to face of curb) with longitudinal slopes: 0.02 to 0.04 and Manning's n=0.015

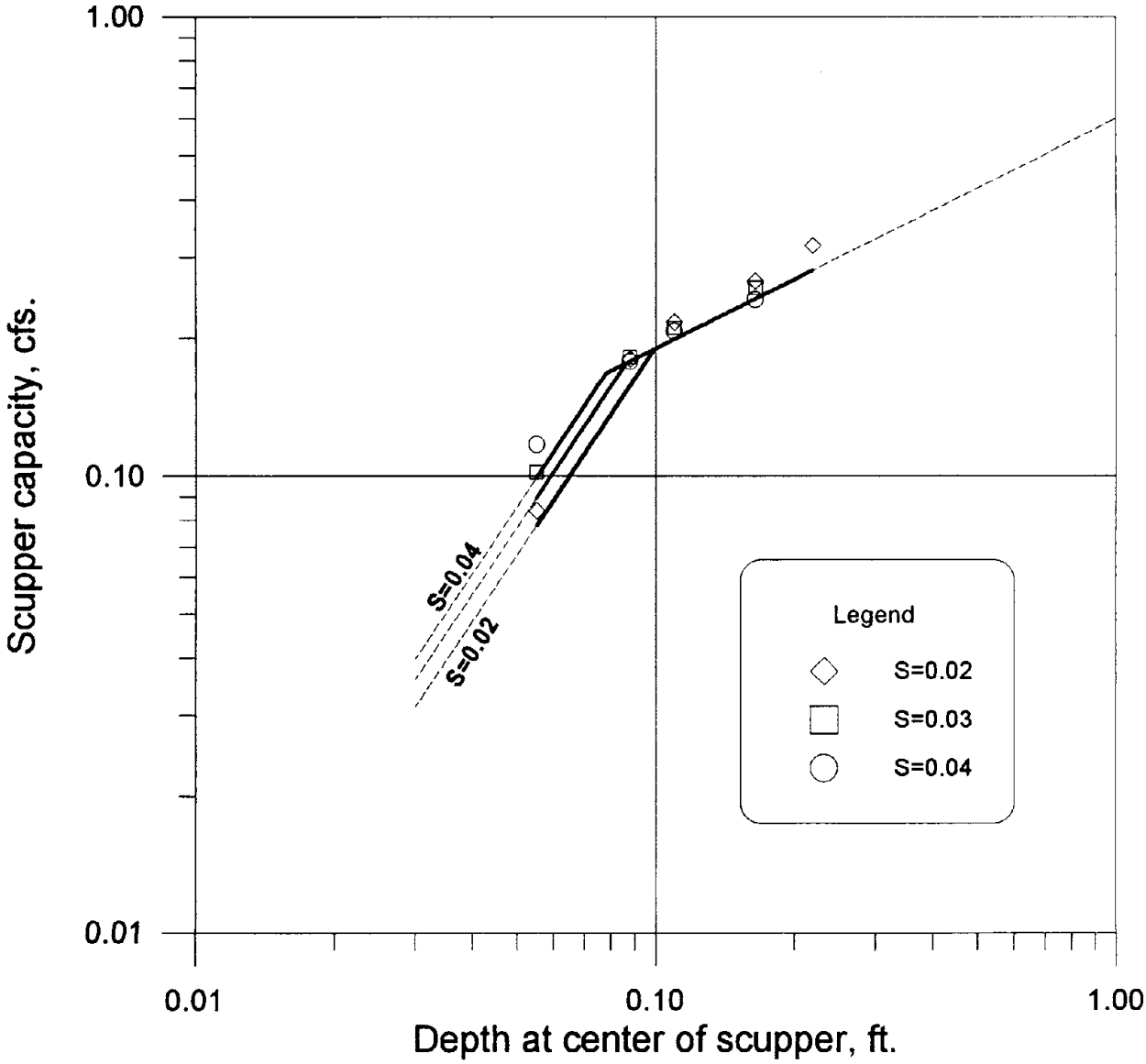


Figure 15

Scupper configuration: rectangular 4"X 8" (flush with deck surface with orientation of the 8" dimension perpendicular to face of curb) with longitudinal slopes: 0.06 to 0.10 and Manning's n=0.015

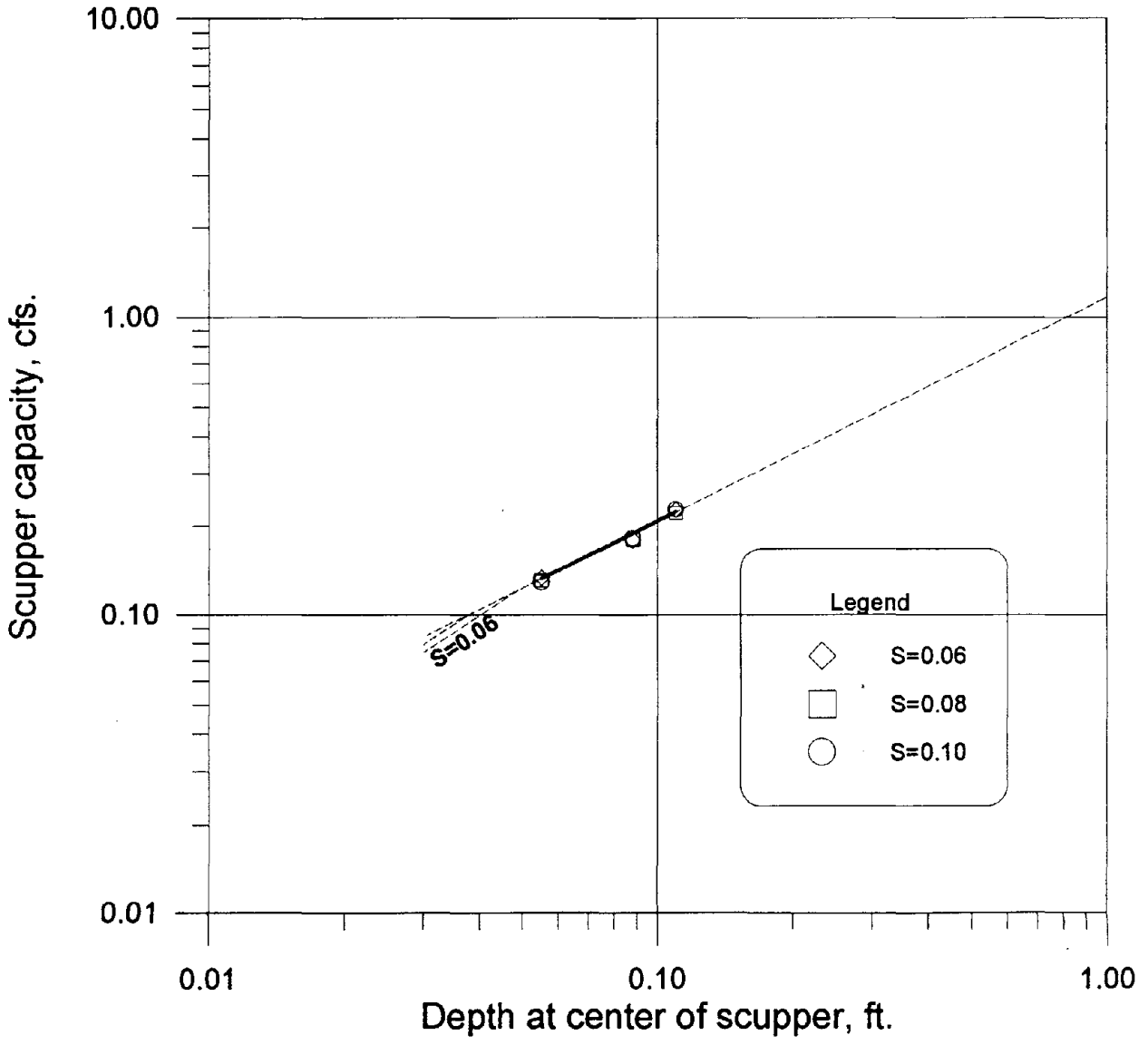


Figure 16

Scupper configuration: rectangular 4" x 8" (recessed 3/4" below deck surface and grouted with orientation of the 8" dimension perpendicular to face of curb) with longitudinal slopes: 0.005 to 0.01 and Manning's n=0.015.

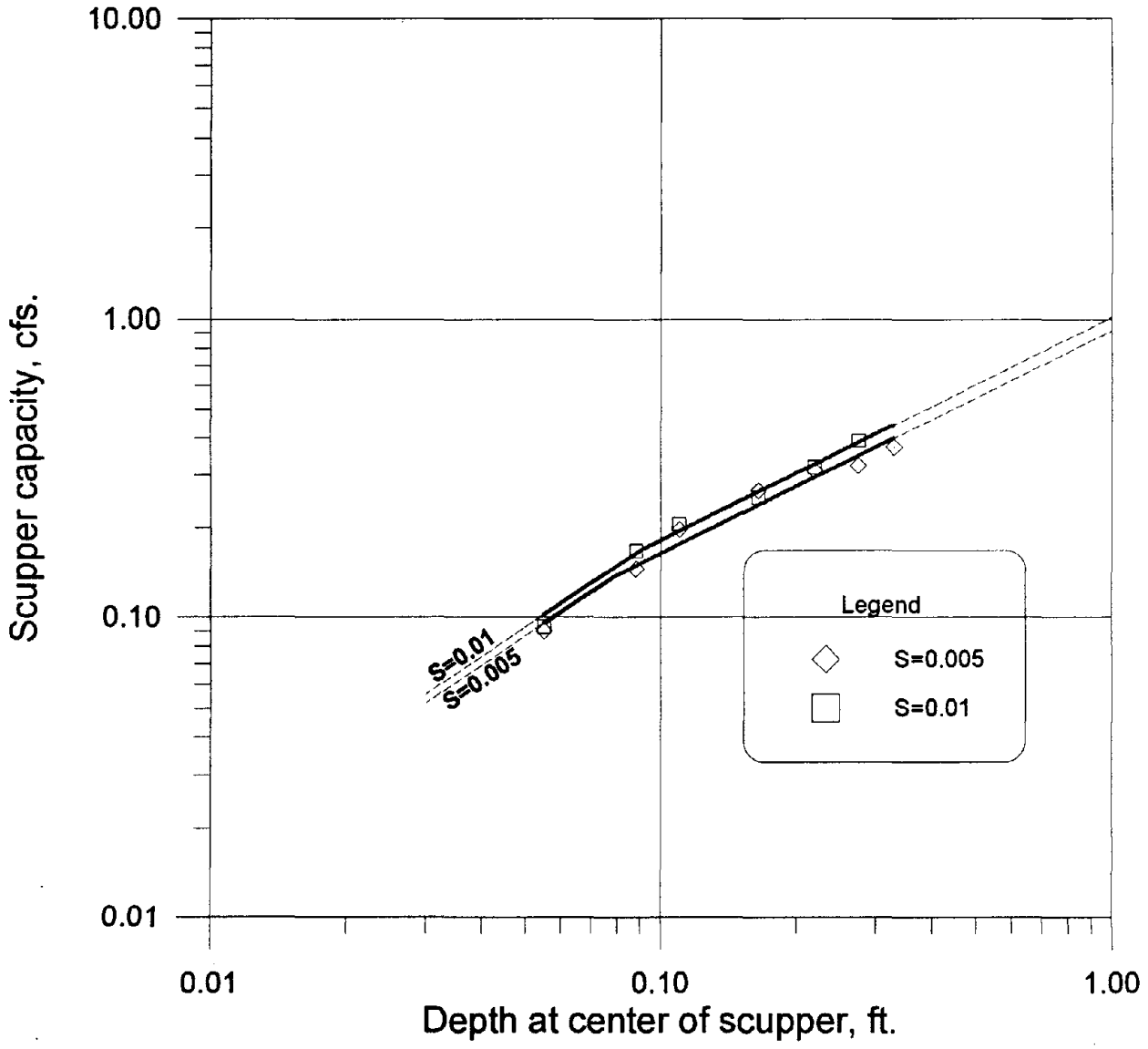


Figure 17

Scupper configuration: rectangular 4" x 8" (recessed 3/4" below deck surface and grouted with orientation of the 8" dimension perpendicular to face of curb) with longitudinal slopes: 0.02 to 0.04 and Manning's n=0.015.

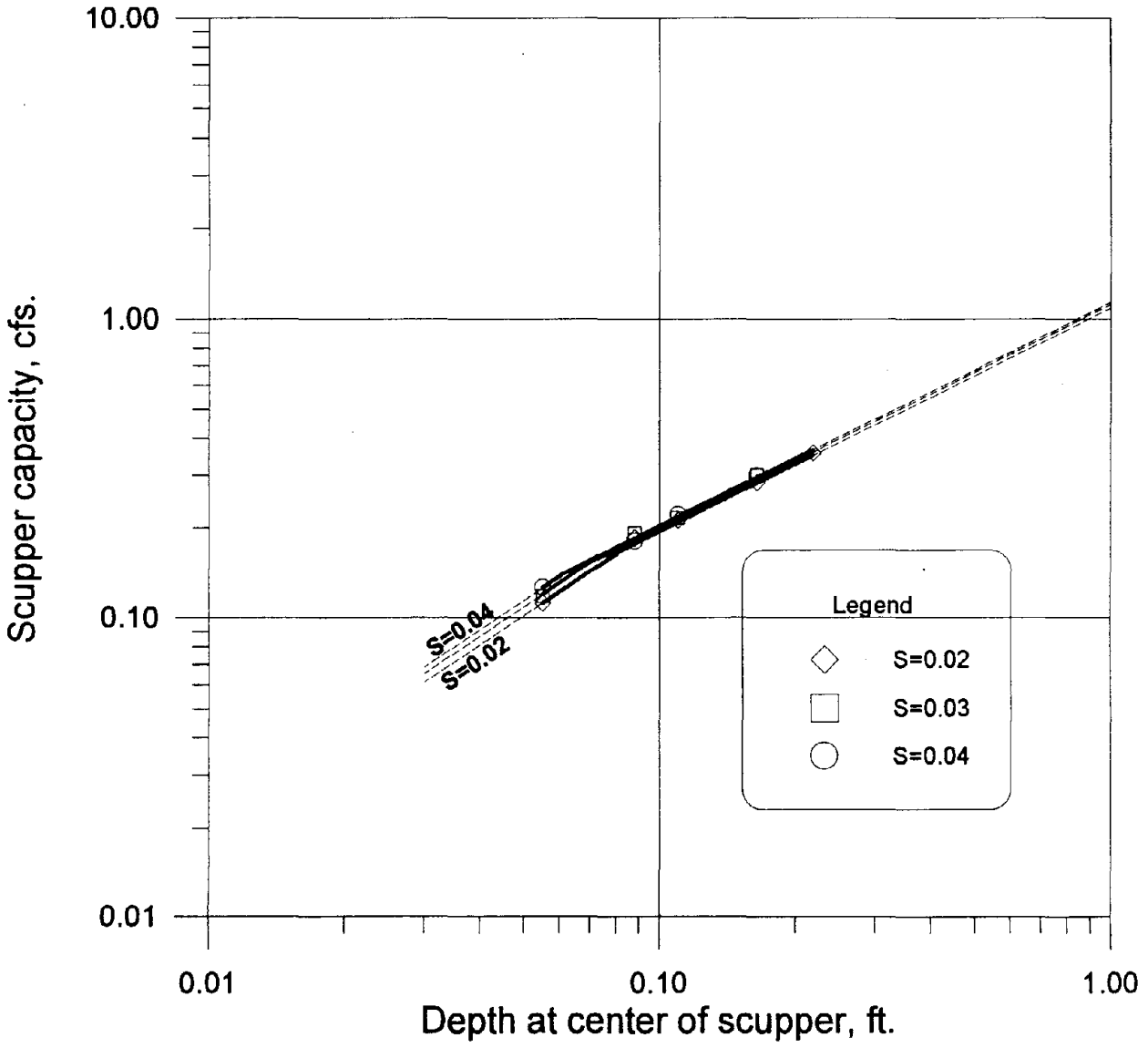


Figure 18

Scupper configuration: rectangular 4" X 8" (recessed 3/4" below deck surface and grouted with orientation of the 8" dimension perpendicular to face of curb) with longitudinal slopes: 0.06 to 0.10 and Manning's n=0.015.

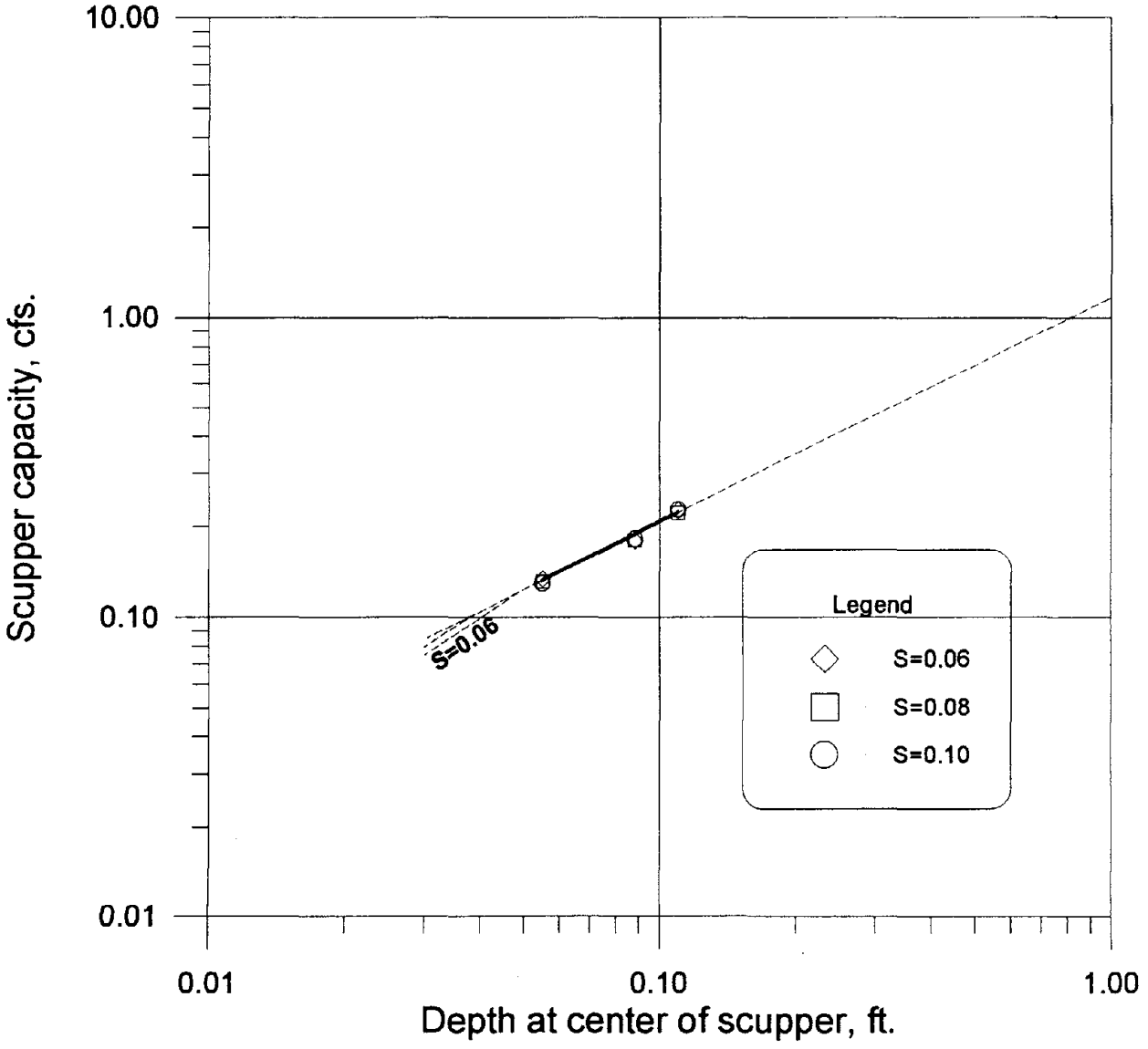


Figure 19

Scupper configuration: rectangular 4" x 8" (recessed 3/4" below deck surface with orientation of the 8" dimension perpendicular to face of curb) with longitudinal slopes: 0.005 to 0.01 and Manning's n=0.015.

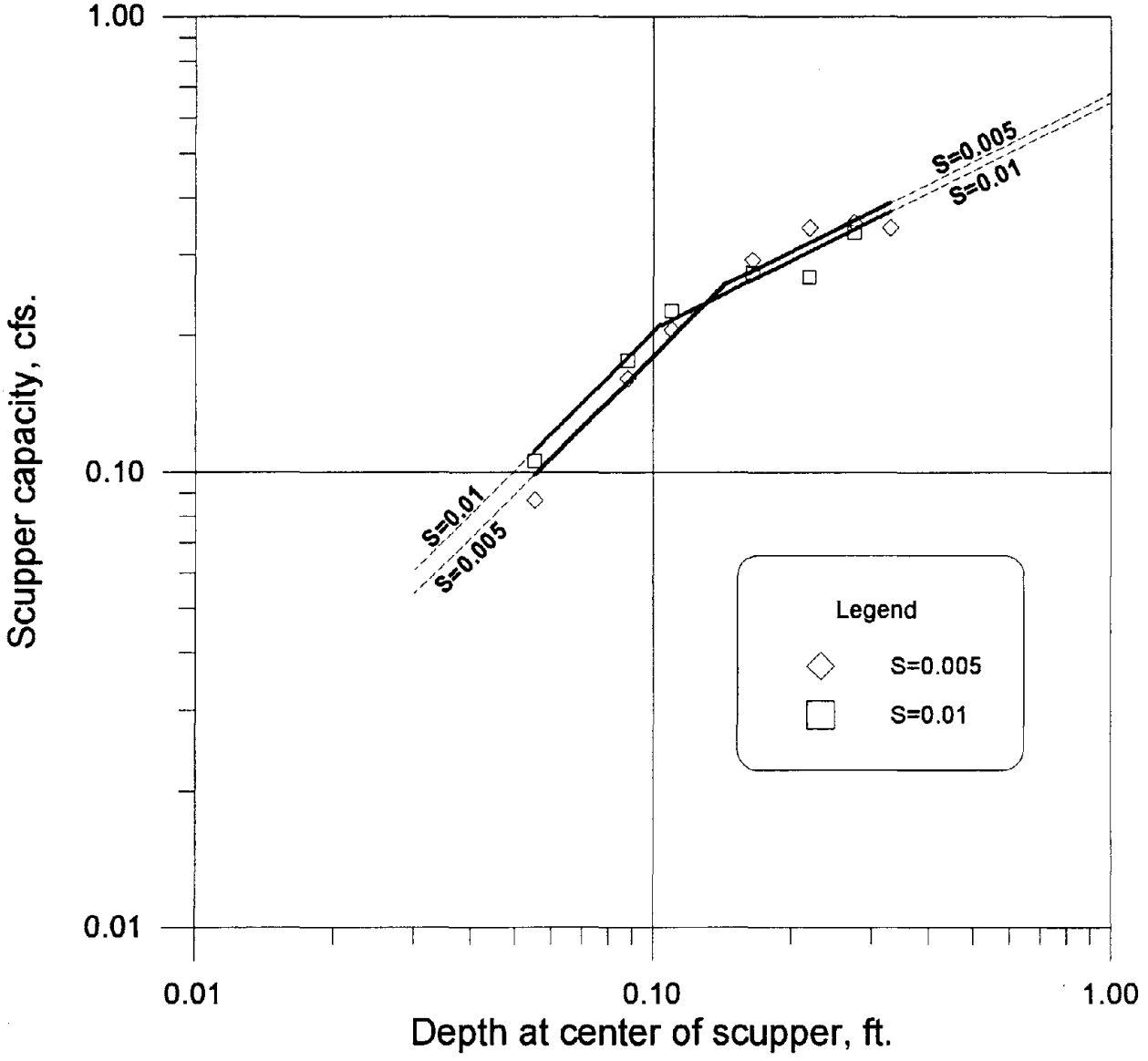


Figure 20

Scupper configuration: rectangular 4" X 8" (recessed 3/4" below deck surface with orientation of the 8" dimension perpendicular to face of curb) with longitudinal slopes: 0.02 to 0.04 and Manning's n=0.015.

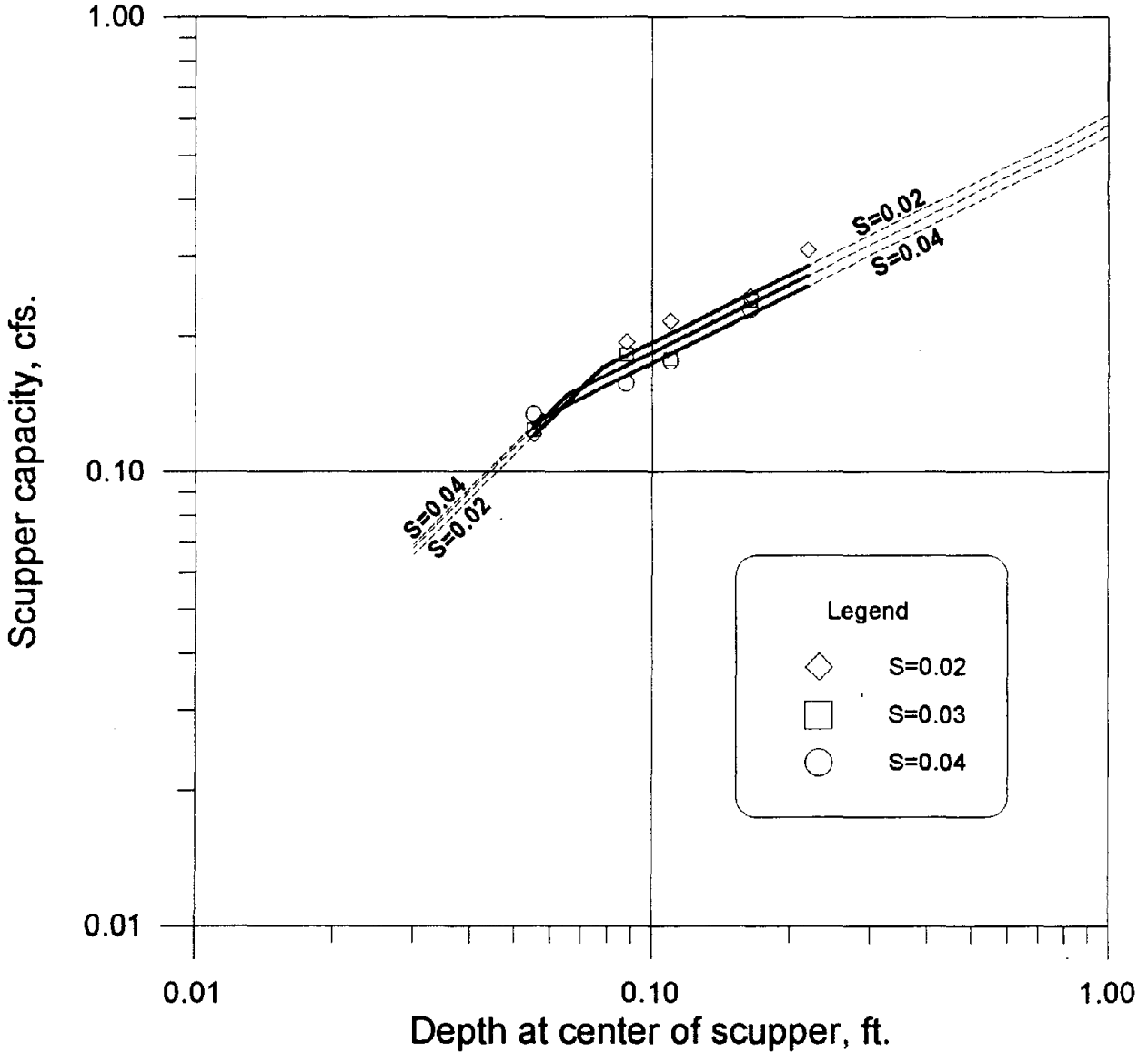


Figure 21

Scupper configuration: rectangular 4" x 8" (recessed 3/4" below deck surface with orientation of the 8" dimension perpendicular to face of curb) with longitudinal slopes: 0.06 to 0.10 and Manning's n=0.015.

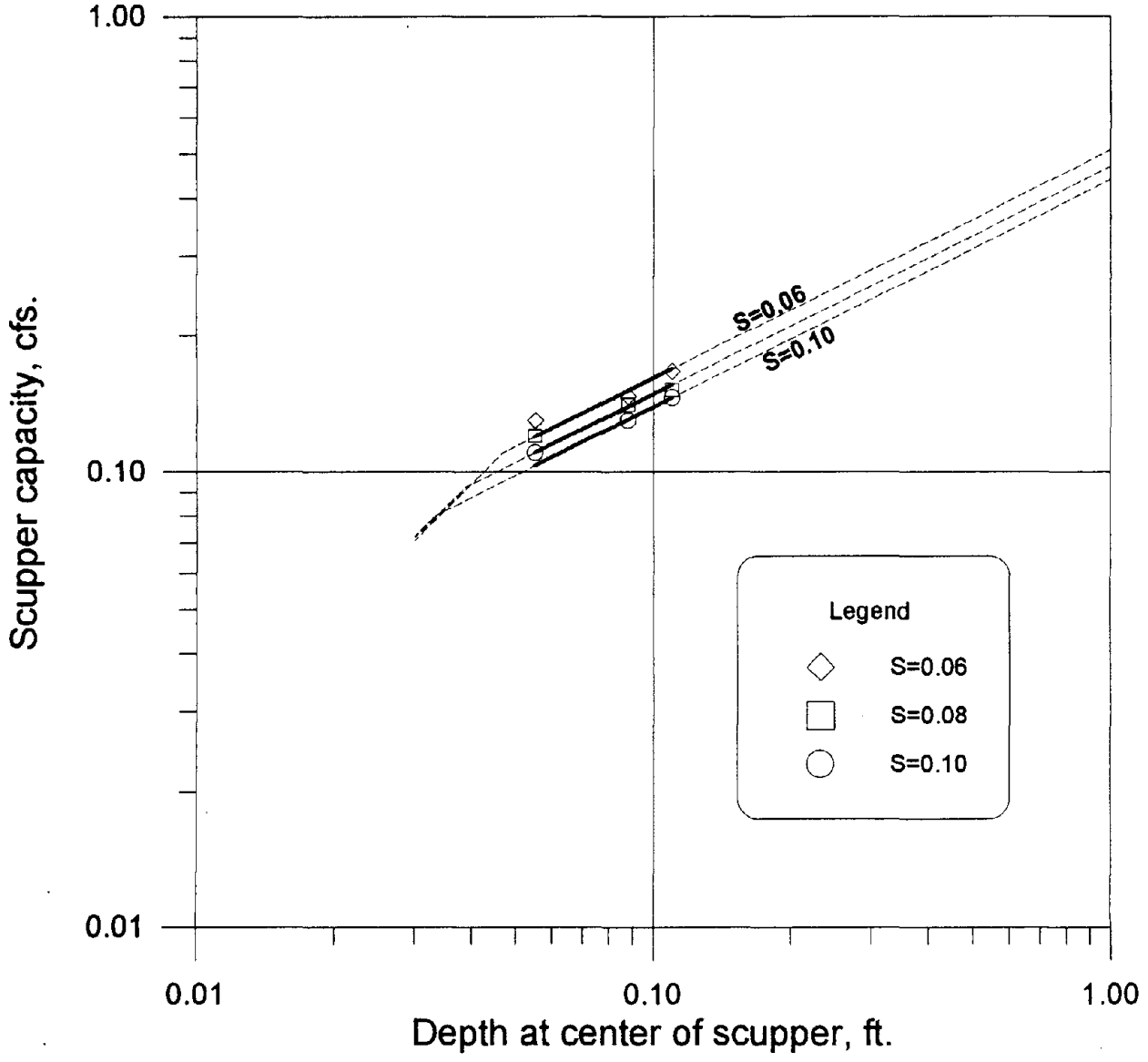


Figure 22

Scupper configuration: circular 6" diameter (flush with deck surface) with longitudinal slopes: 0.005 to 0.01 and Manning's n=0.015.

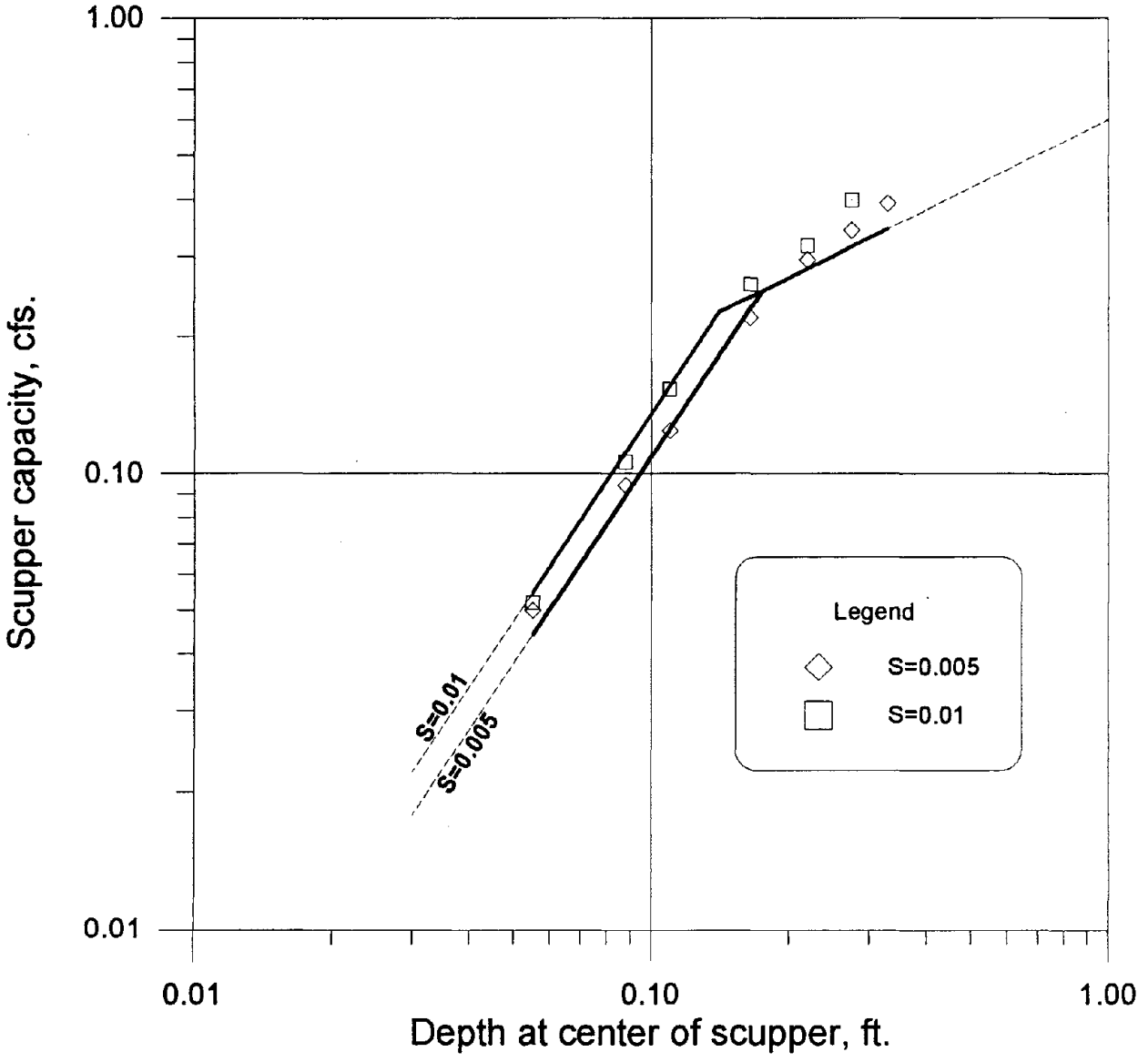


Figure 23

Scupper configuration: circular 6" diameter (flush with deck surface) with longitudinal slopes: 0.02 to 0.04 and Manning's n=0.015.

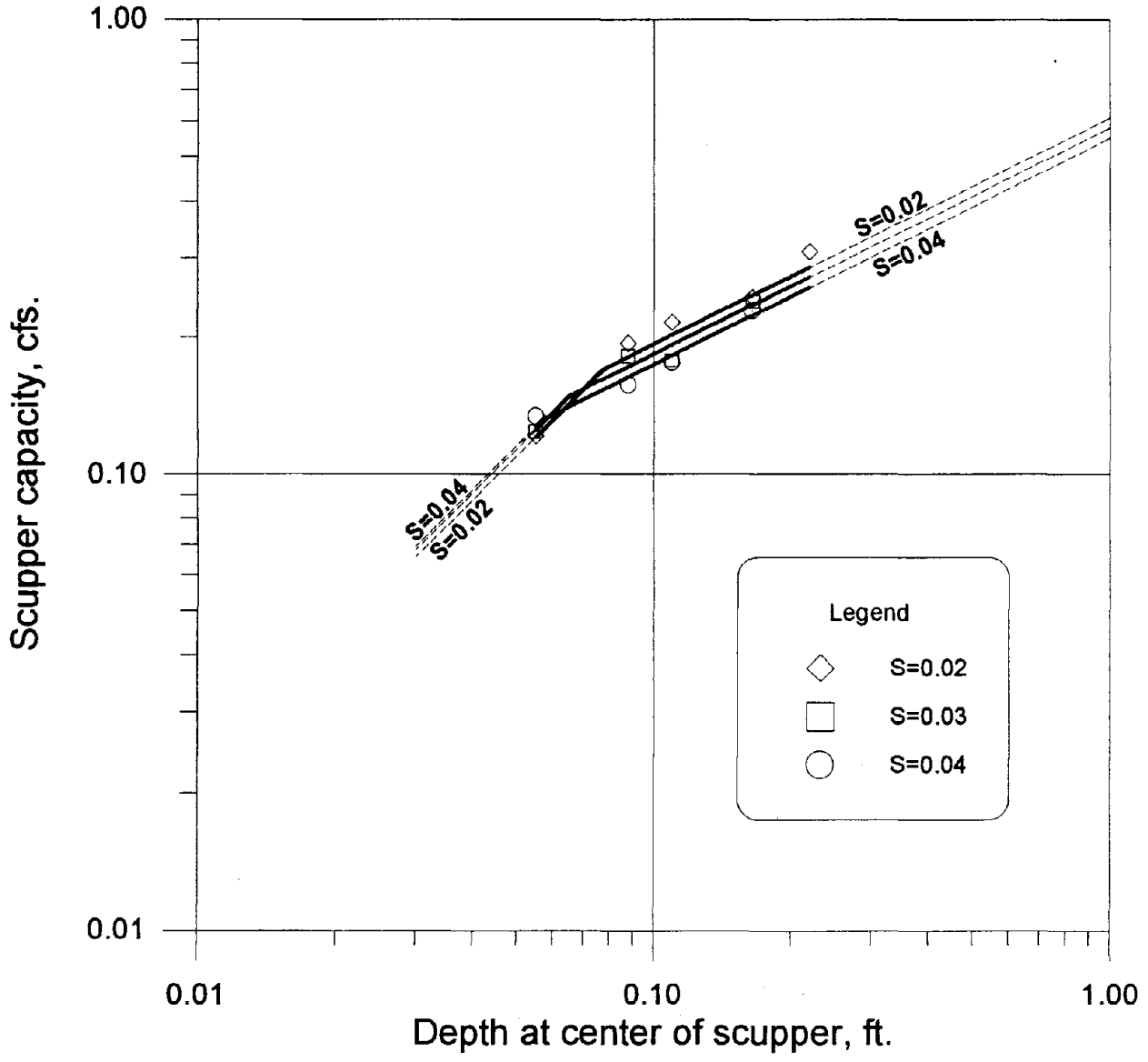


Figure 24

Scupper configuration: Circular 6" diameter (flush with deck surface) with longitudinal slopes: 0.06 to 0.10 and Manning's n=0.015.

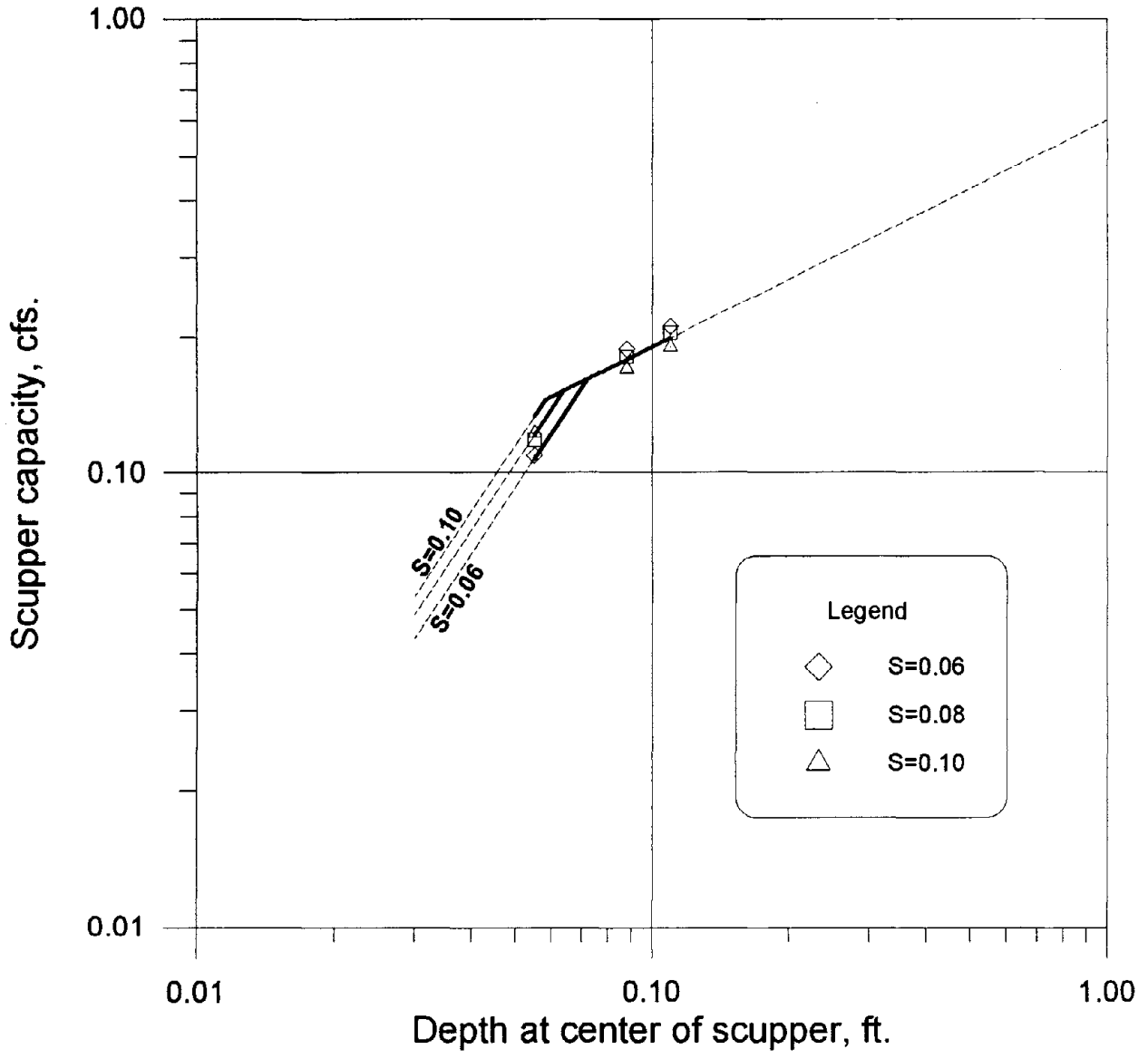


Figure 25

Scupper configuration: rectangular 4"X 8" (flush with deck surface with orientation of the 8" dimension parallel to face of curb) with longitudinal slopes: 0.005 to 0.01 and Manning's n=0.015.

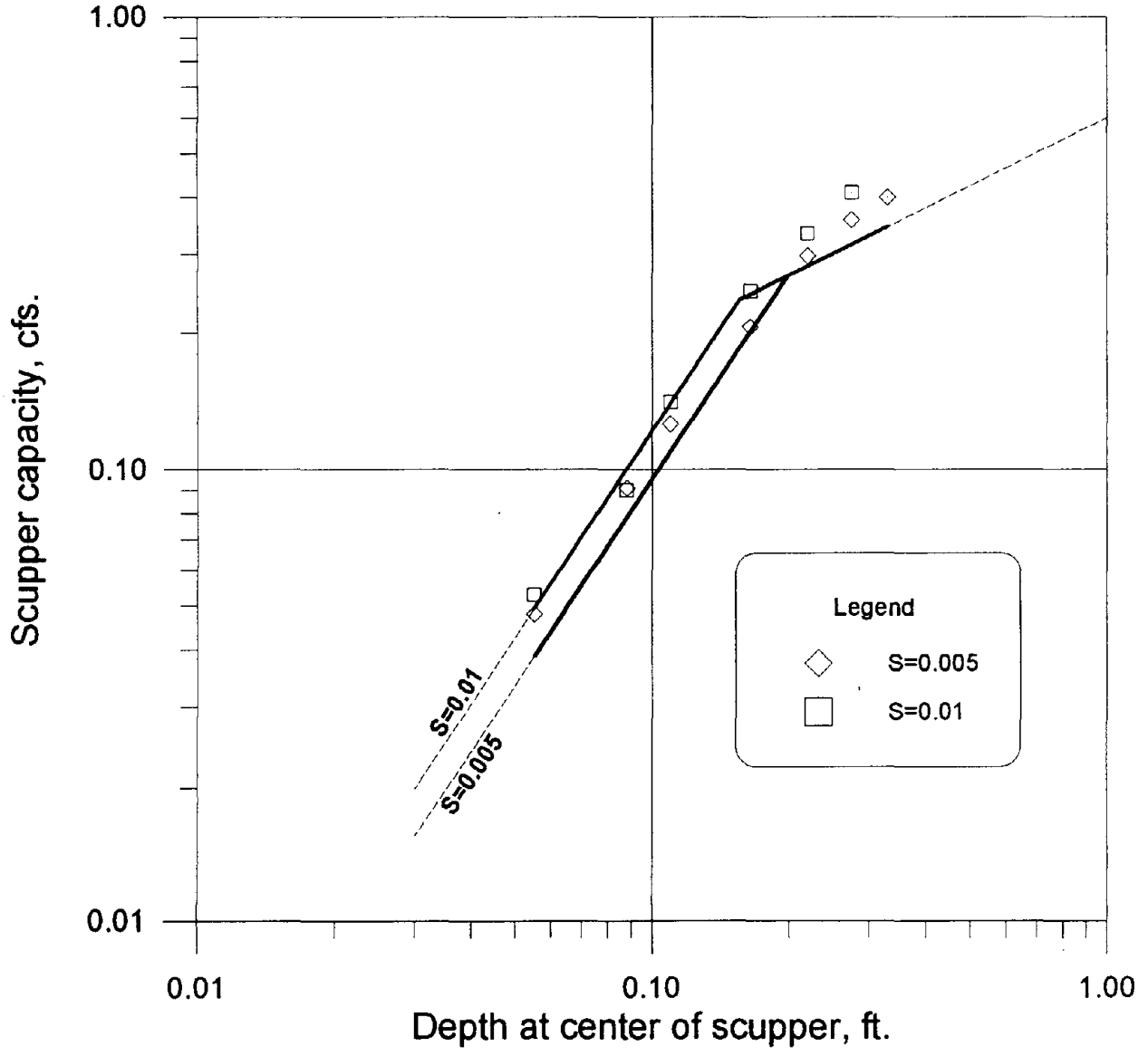


Figure 26

Scupper configuration: rectangular 4"x 8" (flush with deck surface with orientation of the 8" dimension parallel to face of the curb) with longitudinal slopes: 0.02 to 0.04 and Manning's n=0.015.

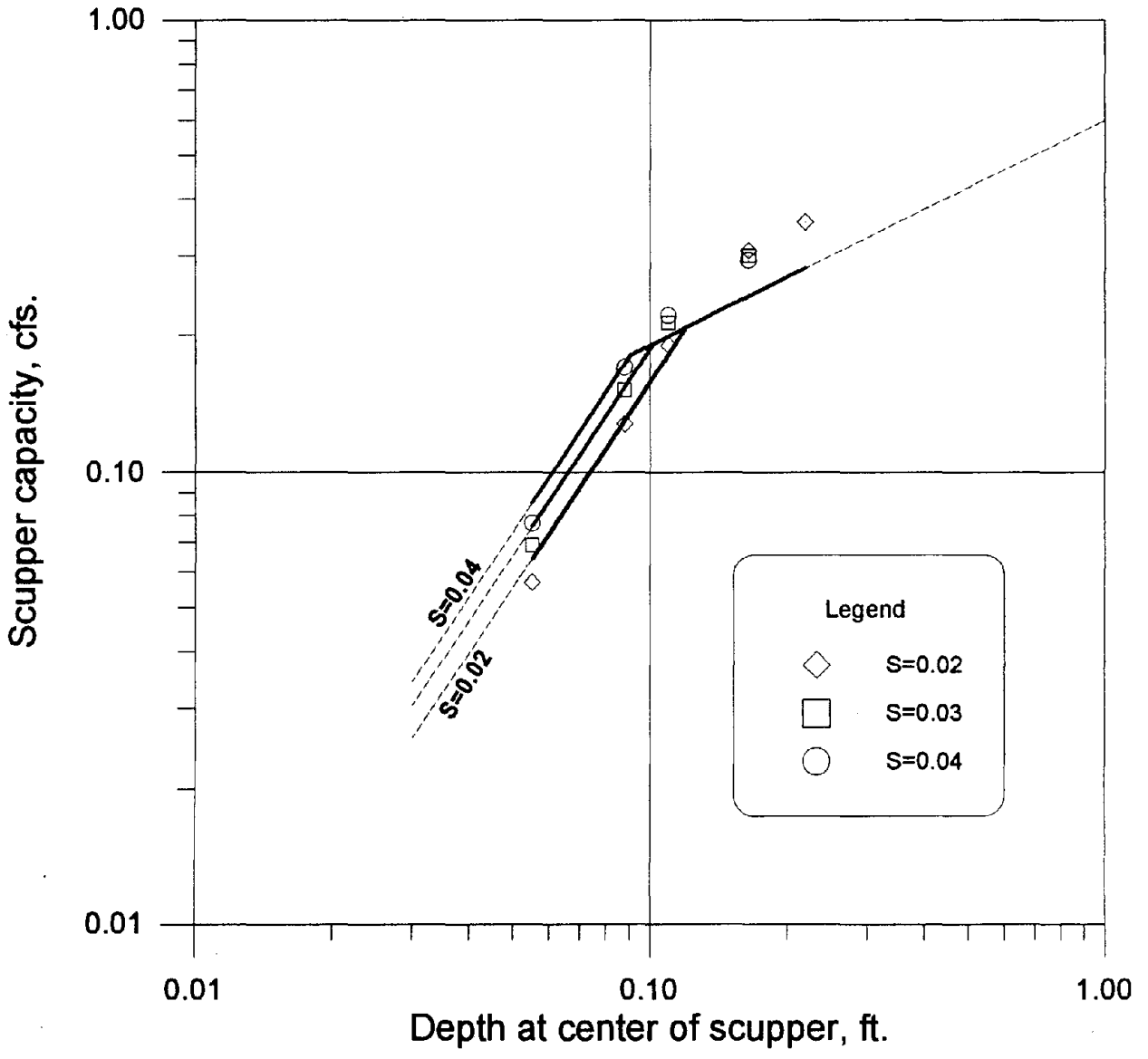


Figure 27

Scupper configuration: rectangular 4"X 8" (flush with deck surface with orientation of the 8" dimension parallel to face of the curb) with longitudinal slopes: 0.06 to 0.10 and Manning's n=0.015.

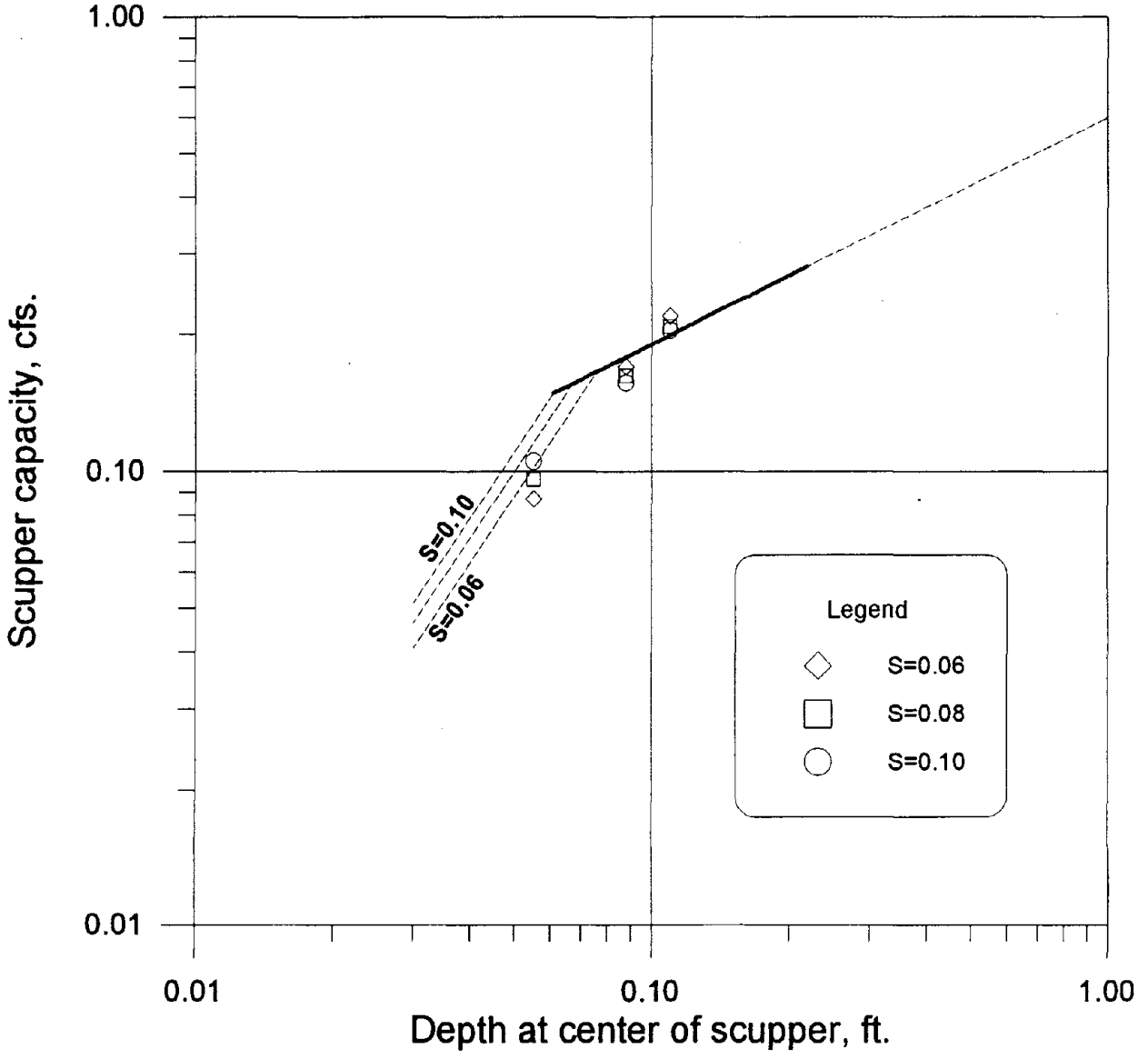


Figure 28

SCUPPER SPACING

A contractual agreement with MHTD called for UMR to create interception equations for new scupper configurations. In the process, the spacing method suggested by the FHWA report was reviewed. Several assumptions in this method were noted, and the effect of the assumptions on scupper spacing was determined. To make the determination, computer program (**BSSP-B**ridge Scupper Spacing Program) was developed which utilizes a mathematical, hydraulic model and varying intensities of rainfall to find the optimal scupper spacing for the most critical design storm. A comparison was made between this model and the FHWA method to determine the affect of the latter's assumptions and the limits of the applicability. Results indicated that UMR's model can also be applied to the placement of drainage elements on streets with curb and gutter, once the interceptions of the drainage inlets are known.

FHWA SCUPPER SPACING PROCEDURE

FHWA uses the rational method for determining the amount of runoff, (Q) contributed to gutter flow by a specified length of bridge deck. The amount of flow in the gutter filled to the design spread ,(Q_T), is approximated by means of Mannings equation and assuming uniform flow. Length of the bridge required for the bridge deck to supply (Q_T) is calculated. An outline of the approach is discussed in *Bridge Deck Drainage Guidelines*.

Variables:

W = width of drainage area

S = grade of deck parallel to flow (may be constant or variable in the case of vertical curve)

Sx = cross-slope of the bridge deck (usually constant, assumed constant in the mathematical model)

I = average design rainfall intensity

T = design spread (width of encroaching water in the gutter)

n = Manning's roughness coefficient (0.016 used as default)

C = runoff coefficient for rational method (0.9 typically used, found in FHWA monographs)

Cross-Deck Runoff- Rational Method

To determine runoff from the bridge deck, the rational method uses

$$Q = \frac{CIWL}{43,560} \tag{7}$$

where

Q = peak runoff in cfs
C = runoff coefficient (0.7-0.9 recommended for concrete and pavement)
I = average rainfall intensity in inches per hour
W = width of drainage in feet
L = length of drainage element in feet

Rainfall Intensity

Three methods for determining intensity values "I" are discussed in *Bridge Deck Drainage Guidelines*. They include the (1) rational method, (2) avoidance of hydroplaning method, and (3) avoidance of driver vision impairment method.

1) *Rational Method*

This rational method makes the assumption that peak flow in the watershed occurs when rainfall intensity lasts as long as time of concentration (Tc). Time of concentration is calculated by summing the time of overland flow and the time of travel in the channel. Empirical relationships are available for estimating both of these times of flow. The kinematic wave equation is suggested for calculating overland flow time and is given as

$$T_o = \frac{56 L_o^{0.6} n^{0.6}}{I^{0.4} S_o^{0.3}} \tag{8}$$

where

L_O = length of overland flow plane

S_o = slope of the overland flow plane (cross slope of roadway)

I= rainfall intensity for selected design frequency (for a duration equal to T_o)

Time of flow in the channel is estimated by dividing the length of the gutter by the average velocity in that length of gutter. Average velocity is estimated as velocity at the point where spread equals

65 percent of design spread. Velocity at this point is calculated using a modified version of Manning's equation recommended by National Hydraulic Laboratory staff at the Bureau of Standards for flow in a channel of uniformly varying depth (triangular)

$$V_{avg} = \frac{0.856}{n} S^{0.5} S_x^{0.67} T^{0.67} \tag{9}$$

This equation appears to over-predict flow in the channel because the effect of the curb on the wetted perimeter is neglected. However, this is only a small percentage of the wetted perimeter in most cases. For example, with a 2% cross-slope and a depth of 0.2, actual wetted perimeter is 10.2 compared with that accounted for in the above equation of 10, about a 2% difference. Use of this equation results in a discharge about 20% greater than the direct application of Manning's equation for equal values of n and depth.

FHWA's publication does not discuss what values of S and S_x to use when these values change with bridge length as found in vertical curves. Using values of S and S_x which produce the largest velocity will produce the smallest time of concentration. This results in the largest average rainfall intensity and yields the most conservative scupper spacing. Once the average velocity and overland flow time are calculated, the time of concentration is calculated as

$$T_c = T_o + \frac{L}{V_{avg}} \tag{10}$$

After calculating the time of concentration, the average intensity of rainfall is determined from an intensity-duration-frequency (I-D-F) curve. After selecting rainfall frequency, the designer determines the Intensity from the I-D-F curve, given the frequency and duration. This selection is an iterative process since overland flow time depends on intensity. FHWA recommends the following procedure in its publication.

- Select frequency
- Assume intensity
- Calculate T_c
- Determine intensity using T_c and an I-D-F curve
- Use new value of I to calculate new T_c
- Continue procedure until assumed I or previous I equals value from I-D-F curve

As proposed by FHWA, this method uses time of concentration of the entire bridge length. This approach may result in an inadequate spacing of scuppers at the beginning of a vertical curve or at the beginning of a bridge. Time of concentration for the entire bridge may yield a smaller rainfall intensity for scuppers near the top of a vertical curve or at the beginning of a bridge than time of

concentration as calculated for each individual scupper. Such a case would typically be found on long bridges with flat slopes which yield greater times of concentration. Thus time of concentration should be computed for each scupper when spacing scuppers on a bridge to assure that an adequate rainfall intensity is used for all scuppers.

2) Avoidance of Hydroplaning

To determine rainfall intensity, the method uses an empirical equation for the speed at which hydroplaning begins. This speed is dependent on: tire tread depth, vehicle mph, pavement texture, water film depth, tire pressure, and spindown. Spindown occurs when tires spin. An empirical equation can be solved for the water film depth given conditions set by the designer. Manning's equation can be used to determine the flow for the calculated water film depth. The rational equation is then used to determine rainfall intensity required to produce the flow corresponding to that water film depth.

If rainfall is more intense than this method recommends, then the water draining from the roadway exceeds the capacity of the scuppers. Ponding results which may in turn cause hydroplaning upon a decrease in rainfall intensity below what this method recommends. Even though rainfall intensity is less than that leading to hydroplaning, the possibility of hydroplaning may exist. Another liability accompanies this method. Some of the parameters (texture of the pavement, tire technology, and design speed) in the empirical equation used to determine rainfall intensity may change during the life of the bridge. Thus it is recommend that the I-D-F method be used to determine rainfall intensity.

3) *Avoidance of Driver Vision Impairment*

This method is based upon the assumption that if rainfall intensity is greater than that at which a motorist's vision is impaired, the motorist slows down and hydroplaning does not occur. An empirical relationship has been established between minimum stopping sight distance, intensity, and velocity. Rainfall intensity is determined from this relationship, given the velocity and minimum stopping sight distance. According to research, this yields an intensity of approximately 5.6 in/hr at 55 mph with no traffic. Intensity values in traffic are lower due to the effect of splashing by passing cars on the driver's vision.

Use of this method to determine rainfall intensity produces problems similar to those found with changing parameters in the avoidance of hydroplaning method. A further liability of this method is the possibility that a storm may be intense but short. A driver may thus slow down initially, but accelerate before the bridge is adequately drained. This delay in drainage is due to scupper spacing based on a less intense storm. Thus it is recommended that the I-D-F Method be used to determine rainfall intensity.

"C" Coefficient

Currently the FHWA computes runoff for scupper spacing by using the rational method which utilizes a "C", also called the runoff coefficient. "C" values normally used for pavement can be found in reference tables with a range of 0.7 to 0.9. Runoff coefficient is dependent on soil type, antecedent moisture, recurrence interval, land use, slope, amount of urban development, rainfall intensity, surface and channel roughness. The rational method is a simple peak flow formula, used to model a complex process.

On a bridge deck, most rainfall runs off, except at the beginning of a storm when rain wets the bridge deck and fills small depression areas. Runoff used to design scupper spacing occurs after the bridge deck is wetted and depression areas are filled. Therefore a "C" value of 1.0 is more appropriate for a bridge deck and is recommended to determine scupper spacing.

Gutter Routing - Uniform Flow

Quantity of flow in the gutter is estimated with a modified version of Manning's equation for flow in a triangular channel

$$(Q_T)_1 = \frac{0.56}{n} S_x^{1.67} S_1^{0.5} T^{2.67} \qquad (11)$$

where $(Q_T)_1$ is full flow in the gutter at design spread T. $(Q_T)_1$ is an estimate of the amount of flow for the bridge slope, shape, and roughness at scupper #1 (first scupper), assuming uniform conditions as predicted by Manning's equation.

Effects of lateral inflow on depth of gutter flow are not addressed in the current design procedure. Note that channel flow accelerates the inflow component. Depth thus increases to compensate for additional energy or momentum needed to accelerate the inflow. This process decreases actual velocity of gutter flow from the value approximated by Manning's equation. Decreased velocity of gutter flow increases the frequency of scupper spacing. Thus, excluding the effect of lateral inflow is not conservative when applied to scupper spacing. Also note that Manning's equation is for uniform flow. But the changing slope of bridges on vertical curves creates additional effects on gutter flow making it nonuniform or varied.

CALCULATION OF SCUPPER SPACING

In previous sections components of the FHWA method were discussed. This section examines how these components methods are used to determine the distance to the first and subsequent scuppers. Estimated gutter flow is set equal to the bridge deck runoff, calculated by using

$$(Q_T)_1 = \frac{0.56}{n} S_x^{1.67} S_1^{0.5} T^{2.67} = \frac{CIWL}{43,560} \tag{12}$$

Solving for length yields

$$(L)_1 = \frac{24,393 .6 S_x^{1.67} (S)_1^{0.5} T^{2.67}}{CnIW} \tag{13}$$

If the value of L_1 is greater than bridge length, scuppers are not required. If the bridge contains a vertical curve, the procedure is iterative. Because L_1 is dependent upon the slope which changes with respect to L_1, the procedure is iterative if the bridge contains a vertical curve.

Calculation of Additional Scupper Spacings

Once the first scupper has been located, its efficiency is required to determine the spacing of additional scuppers. Given the efficiency of this scupper, flow intercepted by the first scupper $(q)_i$ is determined by

$$(q)_i = [E_s(\ Scupper\ Efficiency\ )](Q_T)_i \tag{14}$$

where $(Q_T)_1$ is the flow at which the gutter is filled to the design spread (T) at scupper #1. Intercepted flow decreases the flow in the gutter by q (intercepted) the latter must be replaced before another scupper is added. Runoff in the gutter at the second scupper is given by

$$(Q_T)_{i+1} = \frac{CIW\ \ (TL)_{i+1}}{43,560} - \sum_{j=1}^{i} (q)_j \tag{15}$$

Another scupper is located when runoff minus intercepted flow equals flow in the gutter filled to the design spread (T) at length $(TL)_{i+1}$, where $(TL)_{i+1}$ is the total length of bridge to $(\text{scupper})_{i+1}$. $(Q_T)_i$ equals $(Q_T)_{i+1}$ if the bridge is on a tangent slope, but $(Q_T)_i$ does not equal $(Q_T)_{i+1}$ if the bridge is on a vertical curve. When estimated gutter flow is calculated as equal to bridge deck runoff, the result is

$$(Q_T)_{i+1} = \frac{0.56}{n} S_x^{1.67} S_{i+1}^{0.5} T^{2.67} = \frac{CIW\,(TL)_{i+1}}{43,560} - \sum_{j=1}^{i} (q)_j \tag{16}$$

This equation is solved by means of an iterative process for different $(TL)_{i+1}$ values until successive trials yield identical $(TL)_{i+1}$ values. This process is continued for the 3rd, 4th,....., nth scuppers until the end of the bridge is reached.

HYDRODYNAMIC SCUPPER SPACING PROCEDURE

A computer program (BSSP) a mathematical, hydraulic model to analyze the flow on a bridge deck, is used to study the effects of the FHWA design assumptions. The program uses a kinematic wave approximation to determine the overland flow hydrograph used in the unsteady flow equations. Components of the model are briefly described below.

Kinematic Wave Approximation for Cross-Deck Flow

BSSP uses a kinematic wave approximation to produce a hydrograph of overland flow (cross-deck) in units of cubic feet per second per foot of bridge length (cfs/ft.). This approximation was chosen because it quickly and efficiently produces a hydrograph. An overland flow hydrograph provides lateral flow input for the channel flow. Adequate results are obtained when backwater is not a concern, as in most overland flow cases. With the kinematic wave approximation the continuity and momentum equations are solved, producing a complete and realistic solution for routing. Use of the kinematic wave approximation allows overland flow routing to be considered in the design. This design factor in turn allows the determination of critical rainfall intensity for scupper spacing. Continuity and momentum equations are

$$\frac{\partial y}{\partial t} + y\frac{\partial V}{\partial x} + V\frac{\partial V}{\partial x} = i \qquad (17)$$

$$V = \frac{8\,S\,g\,y^2}{c\,\nu} \qquad (18)$$

where c is a constant.

Kinematic wave approximation is a routing routine and makes no provision for abstractions, such as the "C" value in the rational equation. Textured or cracked pavement may have a small depression storage value which is not considered in kinematic wave approximation. In most cases, a program utilizing this approximation yields realistic scupper spacing because abstractions on bridges are small

and can be neglected. In fact, the most critical design case occurs when initial wetting is followed by heavy rainfall. In this event, most of the abstractions are fulfilled during initial wetting and most, if not all, the heavy rainfall contributes to runoff. If abstractions need to be considered, they can be subtracted from rainfall before input to BSSP.

Critical rainfall intensity is determined using BSSP by routing different I-D (rainfall, intensity-duration) pairs for a given frequency to each individual scupper. This procedure allows each scupper to be designed for a specific intensity and the associated duration. The program's overland flow subroutine was verified to assure continuity in the hydrograph calculation procedure.

Gutter Flow Routing Using Dynamic Wave Equations

After the overland flow hydrograph is calculated, it serves as input (lateral inflow) into the gutter flow routing equations. Lateral inflow value at a particular time is calculated by interpolation between two bounding discharge values (larger and smaller time values) produced in the overland flow subroutine. Gutter flow intercepted by scuppers is taken into account with a negative lateral inflow (q). Quantity of flow intercepted so is calculated in a subroutine containing interception equations for the scupper configurations studied (parallel, perpendicular, grouted perpendicular, and round). At the points where scuppers are located, lateral inflow and intercepted flow are summed, and added to or subtracted from gutter flow.

Gutter flow is routed by means of dynamic wave equations. These equations consist of continuity and momentum equations

$$\frac{\partial h}{\partial t} + D\frac{\partial V}{\partial x} + V\frac{\partial h}{\partial x} = \frac{q}{T} \tag{19}$$

$$g(S_o - S_f) - \frac{Vq}{A} = g\frac{\partial y}{\partial x} + V\frac{\partial V}{\partial x} + \frac{\partial y}{\partial t} \tag{20}$$

and are solved with an explicit finite difference approach. Derivations of the momentum and continuity equations have been presented (Hesterberg, 1994). BSSP uses the explicit finite difference solution to the St. Venant equations, while the continuity and momentum equations are expressed explicitly in terms of known quantities. To evaluate the partial derivatives the lax diffusive scheme (Roberson, Cassidy, Chaudry, 1988) is used.

Boundary conditions (BC) for the time equal zero line are calculated by introducing a small flow into the system (QMIN). The small flow is continually input as lateral inflow. Depth and flow in the system are approximated by using Manning's equation. These approximations are used as boundary values. Unsteady flow equations are solved and iterated until the difference in depth calculated at a

node by successive time steps is within desired limits for the entire bridge length. Depth is used as the steady state flow conditions for the channel with QMIN as the lateral inflow. Steady state conditions serve as boundary conditions along the time equal zero grid line. When an actual rainfall event is entered into the system, lateral inflow begins at QMIN to assure continuity.

Boundary conditions at the beginning of the bridge along the upstream BC, (x=0), are not known and must be approximated. To calculate the BC, (x=0) at grid line, it was assumed the slope at node (x=0) is zero, and the depth at (x= -1) is equals the depth at node (x=1). This configuration is symmetric about node (x=0). Depth at node (x=0) can be calculated for the next time step once depth at node (x=1) is known. If the slope at node (x=0) is not zero, the program makes it zero and gradually increases it to the actual slope. This procedure takes advantage of a symmetrical condition to calculate boundary values.

Boundary conditions along the downstream BC, (x=L), grid line are calculated by altering the approximations used to solve the unsteady flow equation. This alteration assigns a depth value to the node (x=L,t) given the depth at nodes (x=L,t=t-1) and (x=L-1,t). Conditions at node x=L are dependent on upstream conditions because in most cases, the flow is supercritical , and does not rely on the downstream conditions (no backwater).

Stability is an important consideration in hydrodynamic applications. A stable solution scheme is one in which introduced error does not grow as computations progress in time. To achieve this the Courant condition is widely used to find what the limiting size of Δx and Δt should be to maintain stability. An explicit scheme is predicted to be stable if the Courant condition is satisfied, but there is no guarantee of stability. In BSSP, the value of Δx is selected; the value of Δt is determined by its minimum value according to the Courant condition for depth values of the previous time step. As values for a time step are calculated down the x-axis, the minimum Δt is stored. Seventy percent of the minimum Δt value is then used for the next time step.

<u>Rainfall intensity values are selected according to time of concentration of each scupper, not time of concentration for the entire bridge.</u>

Rainfall intensities used in the mathematical, hydraulic model are based on user-defined I-D-F (Intensity-Duration-Frequency) values. BSSP begins with the shortest, most intense Intensity-Duration (I-D) pair for the desired frequency, and calculates length required to fill the gutter to the design spread during the rainfall event. Spacing to the first scupper is calculated by finding the shortest distance in which the gutter is filled to the design spread for a particular I-D pair. This pair spacing is then stored, and the next I-D pair is used to calculate a length to fill the gutter to the design spread. If the first value is smaller, the first scupper is placed at that spacing. If the second value is the smaller, another intensity is chosen and the process is repeated until the smallest value is obtained.

Spacing after the first scupper is determined in a similar manner, with intercepted flow removed at previously determined scupper locations. Using this procedure, the intensity which creates the minimum spacing is used to locate each scupper. The method is analogous to calculating a time of concentration for each scupper.

BSSP's hydrodynamic subroutine was examined to verify that the hydrodynamic equations were solved correctly. Flow was checked for continuity and depth (uniform flow conditions) under steady flow conditions.

Steady flow conditions were checked on a constant slope, using a constant overland flow (lateral flow) from the deck for 100 ft. followed by no lateral inflow for 400 ft. After the flow stabilized, the values of flow and depth compared with those predicted by the continuity and Manning equations (Hesterberg, 1994).

COMPARISON OF METHODS FOR DETERMINING SCUPPER SPACING

Comparisons of the scupper spacings obtained from BSSP and FHWA methods are now discussed. FHWA was used with a constant intensity value for the entire bridge and with a varying rainfall intensity, t_c (time of concentration) calculated numerically for individual scuppers with a 0.1% tangent slope.

FHWA yields scupper spacings between 10.0% and 20.0% greater than those calculated by BSSP for bridges with tangent curves greater than 0.5% (using a "C" value of 0.9 in FHWA). FHWA yields scupper spacings within 7.0% of those calculated by BSSP for bridges with tangent curves greater than 0.5% (using a "C" value of 1.0 in the FHWA method). Differences between BSSP and FHWA with C = 1.0 come from the difference in intensity values used to calculate the respective spacings. Intensities for BSSP were calculated using I-D-F values input by the user while the values for FHWA were interpolated between BSSP's intensity values. Therefore, slightly different intensities may have been present in the methods. If more I-D-F values had been used in BSSP, the spacings would have been closer. Bridge results (3.0% and 4.0% slope), which used rainfall intensities of 8 in/hr in both BSSP and FHWA , show scupper spacing comparisons within 4.0%.

FHWA yields scupper spacings much greater than BSSP's for the 0.1% tangent bridge. Spacing differences were decreased with variable time of concentration and a C value of 1.0. Yet values continue to be more than 50% larger than those calculated by BSSP (Hesterberg, 1994).

CONCLUSIONS AND RECOMMENDATIONS

The hydraulic interception efficiency and flow capacity of bridge deck drains are determined for design. Scupper configurations are 6" circular and 4" x 8" rectangular sections. Parallel and perpendicular orientation to the flow direction and recessing (with and without grout) below the pavement were investigated. The results of this investigation indicate that the 4" x 8" rectangular section orientationed with the 8" dimension perpendicular to the flow provides the greatest interception efficiency for small longitudinal slopes (maximum of approximately 40%). For larger longitudinal slopes the flow tends to over shoot the scupper, thus both orientations and the circular configuration provide approximately the same interception efficiency (maximum of approximately 30%). The circular scupper interception capacity lies between the parallel and perpendicular orientations.

Underestimation by the FHWA method for spacing scuppers on tangent slopes less than 0.5%, is due to the effect of overland flow on gutter flow. As the former contributes to the latter, momentum from the gutter must be used to accelerate overland flow. Water depth thus increases but is not accounted for in the FHWA method. The result is shorter scupper spacings. Effects of overland flow on gutter flow decrease as the slope increases, or as the bridge width decreases. At 0.1% the difference is fairly large (spacing for the first scupper is 82 feet as compared to 37 feet computed with BSSP) while the difference at 0.5% is negligible.

For tangent slopes less than 0.5%, the effects of unsteady flow should be taken into account when spacing scuppers. For tangent slopes greater than 0.5%, the current scupper spacing method may be used with error less than 7.0%. Error is less than 7.0% because the intensities used to compare the FHWA and BSSP spacings were not exactly the same. A "C" value of unity should be used with the rational method to reduce scupper spacings by approximately 10.0% and more accurately model the runoff. I-D-F method, with modifications, should be used to calculate rainfall intensity. This intensity should be verified at subsequent scuppers to determine if a representative intensity is being used.

Use of the FHWA method for vertical slopes has the same limitations it does for tangent slopes (C = 1.0, variable time of concentration calculated). If the first scupper is located on a slope of less than 0.5%, then unsteady, non-uniform flow effects should be taken into account when spacing scuppers (Hesterberg, 1994).

Neglecting the economics, maintaince, and other engineering aspects of design (considering only the hydraulic capacity) associated with the various shapes studied, it is recommended that the 4" x 8" rectangular section oriented with the 8" dimension perpendicular or circular section be used for small longitudinal slopes and cross slopes. For steeper longitudinal slopes, the rectangular section orientation with the 8" dimension parallel to flow or circular section should be used. In either case the circular section gives less interception capacity than the rectangular section if the appropriate rectangular orientation for a given slope is used. When using the circular section the slopes are not a consideration.

BIBLIOGRAPHY

1.) Bridge Deck Drainage Guidelines. U.S. Department of Transportation. Federal Highway Administration Report No. FHWA/RD-87/014, December 1986.

2.) Johnson, Frank L., Chang, Fred F.M., "Drainage of Highway Pavements," Hydraulic Engineering Circular No. 12, FHWA-TS-84-202, Federal Highway Administration, Washington, D.C., March 1984.

3.) Chow, Ven T., Open Channel Hydraulics. McGraw , Inc., 1988.

4.) Izzard, C.F., "Hydraulics of Runoff from Developed Surfaces." Proc. Highway Research Board, Volume 26, pp. 129-150, Highway Research Board, Washington, D.C., 1946

5.) Mahmood, K., and V. Yevjevich, Unsteady Flow in Open Channels. Volume I, Water Resources Publications, Fort Collins, Colorado, 1975.

6.) Roberson, John A., Cassidy, John J., and Chaudry, Hanif, Hydraulic Engineering. Houghton Mifflin Company, 1988.

7.) Daugherty, Robert L., Franzini, Joseph B., and Finnemore, E. John, Fluid Mechanics with Engineering Applications. 8th Ed. McGraw - Hill, Inc., 1985.

8.) Modern Sewer Design. 2nd Ed., American Iron and Steel Institute, 1990.

9.) Viessman, Warren Jr., Lewis, Gary L., and Knapp, John W., Introduction to Hydrology. 3rd Ed., Harper & Row, Publishers, Inc., 1989.