EFFECTIVENESS OF HIGHWAY EDGEDRAINS

FINAL REPORT--Experimental Project No. 12,
Concrete Pavement Drainage
Rehabilitation

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METRIC CONVERSION FACTORS

Multiply inch-pound unit	by	to obtain SI unit
inch (in)	25.4	millimeter
inch (in)	2.54	centimeter
foot (ft)	.3048	meter
cubic foot (ft^3)	.0283	cubic meter
gallons (gal)	3.785	liters
square feet (ft^2)	0.0929	square meters
inches per hour (in/hr)	2.54	centimeters per hour
feet per day (ft/day)	0.3048	meters per day
pounds per square inch (lbs/in^2)	0.0703	kilograms per square centimeter
pounds per cubic feet (lbs/ft^3)	0.016	grams per cubic centimeter
cubic feet per second (ft^3/s)	0.028	cubic meter per second
gallons per minute (gal/m)	0.000063	cubic meters per second
gallons per day per square feet [$(gal/day)/ft^2$]	0.0408	meters per day
gallons per day per square feet [$(gal/day)/ft^2$]	0.134	feet per day

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ABSTRACT

Highway sites in ten States, where edgedrains had been retrofitted along the pavement edges, were instrumented to measure concurrent rainfall and edgedrain discharges, piezometric water levels, and soil moisture under the pavement and adjacent shoulders. Soil samples were also collected and their physical and hydraulic properties measured; all sites were found to have relatively low permeabilities. Fifty selected rainfall-runoff events were analyzed to assess the amount of infiltration reaching the pavement subgrades, amounts and timing of edgedrain discharges, and the manner of water movement beneath the pavement. The data indicate that retrofitting longitudinal edgedrains to an existing highway provides a sink to collect water draining laterally off the pavement surface as well as water reaching them from the subgrade voids and channels. The tight, low permeability subgrade material found to exist at all ten sites precluded ready, lateral drainage with or without edgedrains. The edgedrain outlets then serve to short-circuit the combined flow through the highway shoulders. The low permeabilities were also confirmed by the transducer data because piezometric water levels beneath the pavements were generally slow to recede.

The data obtained indicate that most of the lateral subgrade movement of water is through voids and channels that develop under the pavements. In a sense, "the horse is already out of the barn" if the deterioration of the highway has reached this state. If highway restoration, as well as new construction, includes providing a permeable subbase as well as edgedrains, the two together should prove the most efficient in restoring the highway.

In addition to providing data as to the effectiveness of edgedrains, this study also developed and tested instrumentation and techniques for studying pavement drainage. A dual tipping bucket gage proved most effective in measuring concurrently rainfall and edgedrain discharges. Pressure transducers were effective in measuring piezometric water levels beneath the pavements. Data loggers proved effective in not only recording all data but could be programmed to operate the sensors only and to the extent needed. The need to acquire core samples of the subgrade material and analyze for physical and hydraulic properties was emphasized.

INTRODUCTION

One of the most important components of a highway pavement system is a well drained foundation. Highway engineers, through the history of road building, have maintained that inadequate subsurface drainage results in rapid deterioration of any road and ultimately leads to its failure. The era of high speed interstate highways has led to increased traffic and load weights requiring increased emphasis on the strength and stability of the base material. These requirements, however, lead to some extent to a compromise of good drainage. The need for strength has led to the typical interstate highway construction consisting of a thick (8 to 9 inches) concrete pavement surface section over a dense, graded aggregate base (DGAB). In most cases, this base is virtually impermeable. To prevent water from entering the base, the concrete-to-concrete transverse and longitudinal joints are sealed with a silicone rubber material and the concrete to asphalt longitudinal shoulder joints are sealed with a rubber asphalt.

The Problem

Highway joint materials deteriorate over time and the joints become avenues for water to enter the pavement structure. The water that enters the pavement section of the roadway seeks to escape through lateral movement at the slab-base interface. Water exits along the pavement shoulder or at down-gradient joints, carrying fine material with it (fig. 1). Any water that becomes trapped is subjected to the pressure created by vehicle traffic and "pumping" occurs (fig. 2). Voids and channels are created in the subbase as this process continues. This development of voids ultimately leads to cracking of the pavement slab (Hansen, 1991) (fig. 3) which results in further infiltration of water into the base. Figure 4 depicts a typical roadway and the routes of possible infiltration.

A Solution

Recognizing that the pavement structural section is the most costly element of the highway system, the Federal Highway Administration (FHWA) recommends the retrofitting of longitudinal edgedrains in their pavement rehabilitation practices (Baumgardner and Mathis 1989). Some State highway departments are also incorporating permeable bases with edgedrain systems into the initial design and construction. The State of the Practice Report by Baumgardner and Mathis (1989) provides considerable detail and information on concrete pavement drainage systems and rehabilitation practices in the ten States involved.

Figure 1.--Subgrade fine material removed by infiltrated water discharging along longitudinal edge joint.

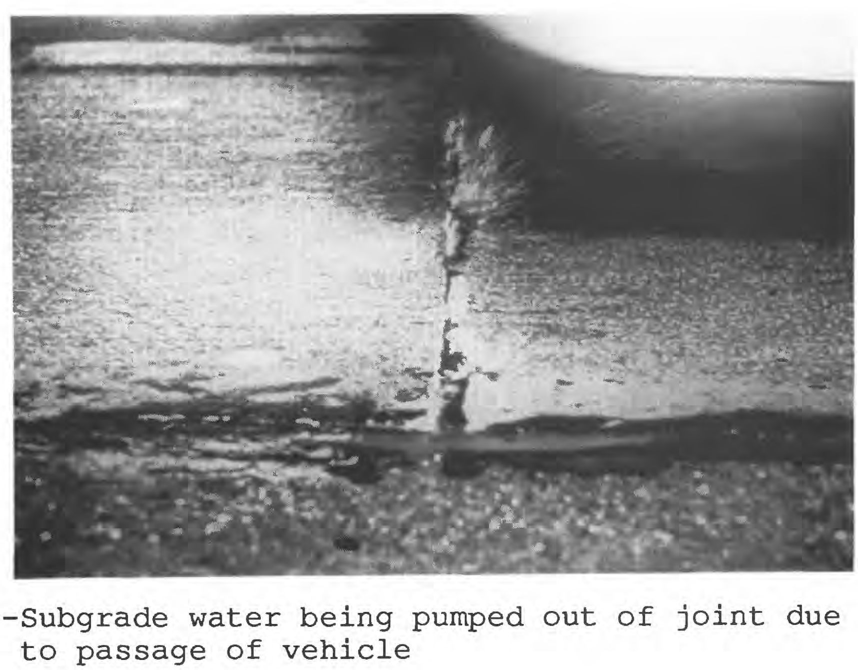

Figure 2.--Subgrade water being pumped out of joint due to passage of vehicle

Figure 3.--Failure of highway pavement due to infiltration of water into subgrade.

Basically, a longitudinal edgedrain is a shallow trench excavated parallel to the immediate edge of the highway pavement and filled with aggregate or other permeable material. Various types of edgedrains are used (fig. 5A); the construction of edgedrains is illustrated in figures 5B through 5E. Some edgedrains have perforated pipe in all or part of the trench length (fig. 5C) with outlets spaced at intervals of 200 to 1000 feet (fig. 5D). Most contain filter cloth either around all or part of the perimeter of the trench or around the pipe. There are also specially designed plastic geocomposite fin drains which are placed vertically to intercept lateral flow from beneath the pavement. These incorporate a filter cloth into their construction.

Purpose and Scope of Report

Edgedrains have been in use throughout the United States for over 15 years. They have been retrofitted primarily to drain concrete pavements and in pavement rehabilitation work such as asphalt overlays. Despite their widespread use, no comprehensive study has been made as to their effectiveness. In 1987, the U.S. Geological Survey (USGS), in cooperation with the FHWA, initiated an experimental project to study pavement drainage and edgedrain

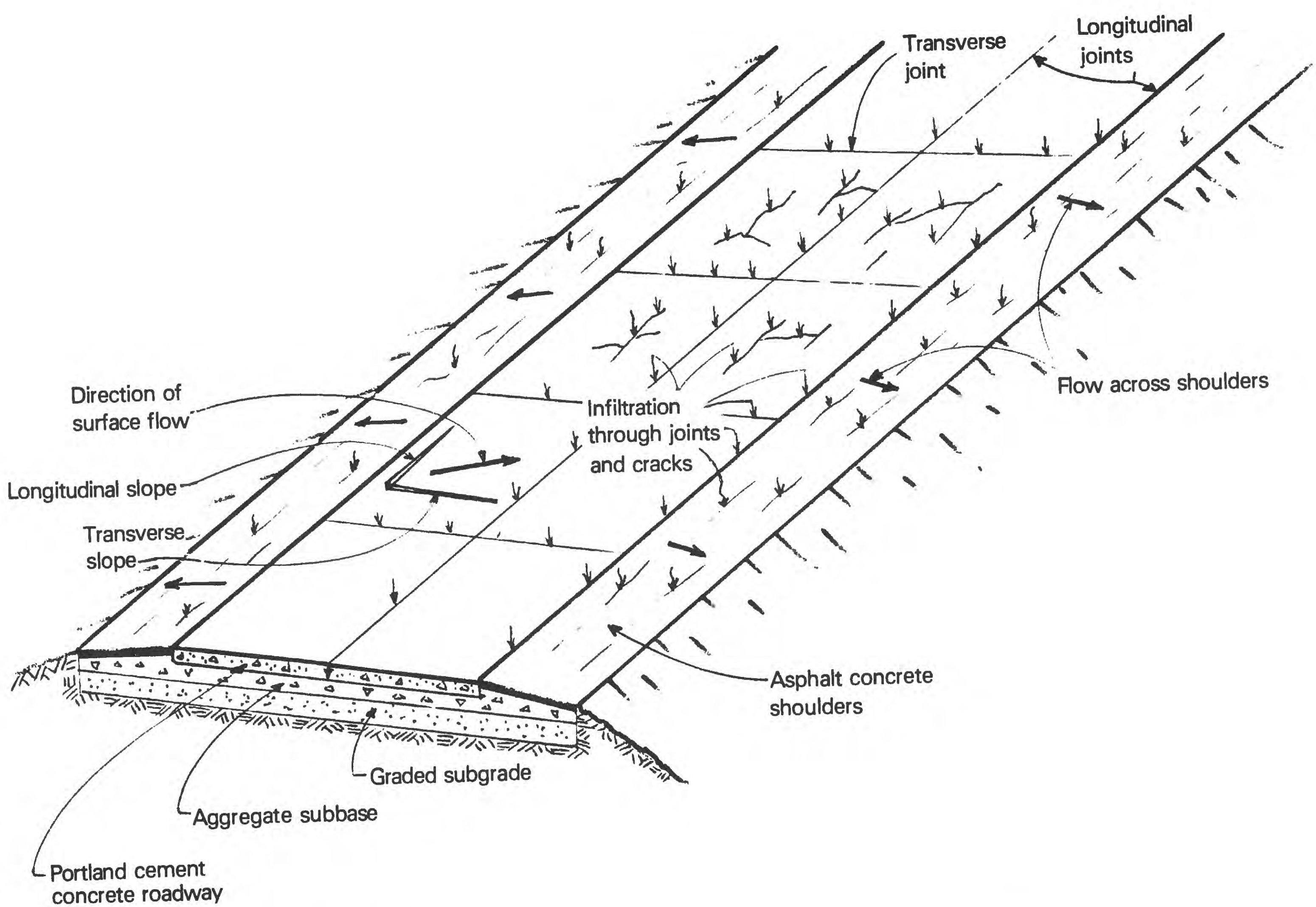

Figure 4.--Typical roadway showing surface flow and routes of infiltration.

effectiveness, and requested assistance from the USGS. This report presents the results of the study of edgedrains at ten highway sites in the United States. The data collected during the study, which include measurements of various aspects of the movement of water into, through, and out of highway subgrades, are analyzed and interpreted in terms of the effectiveness of edgedrains.

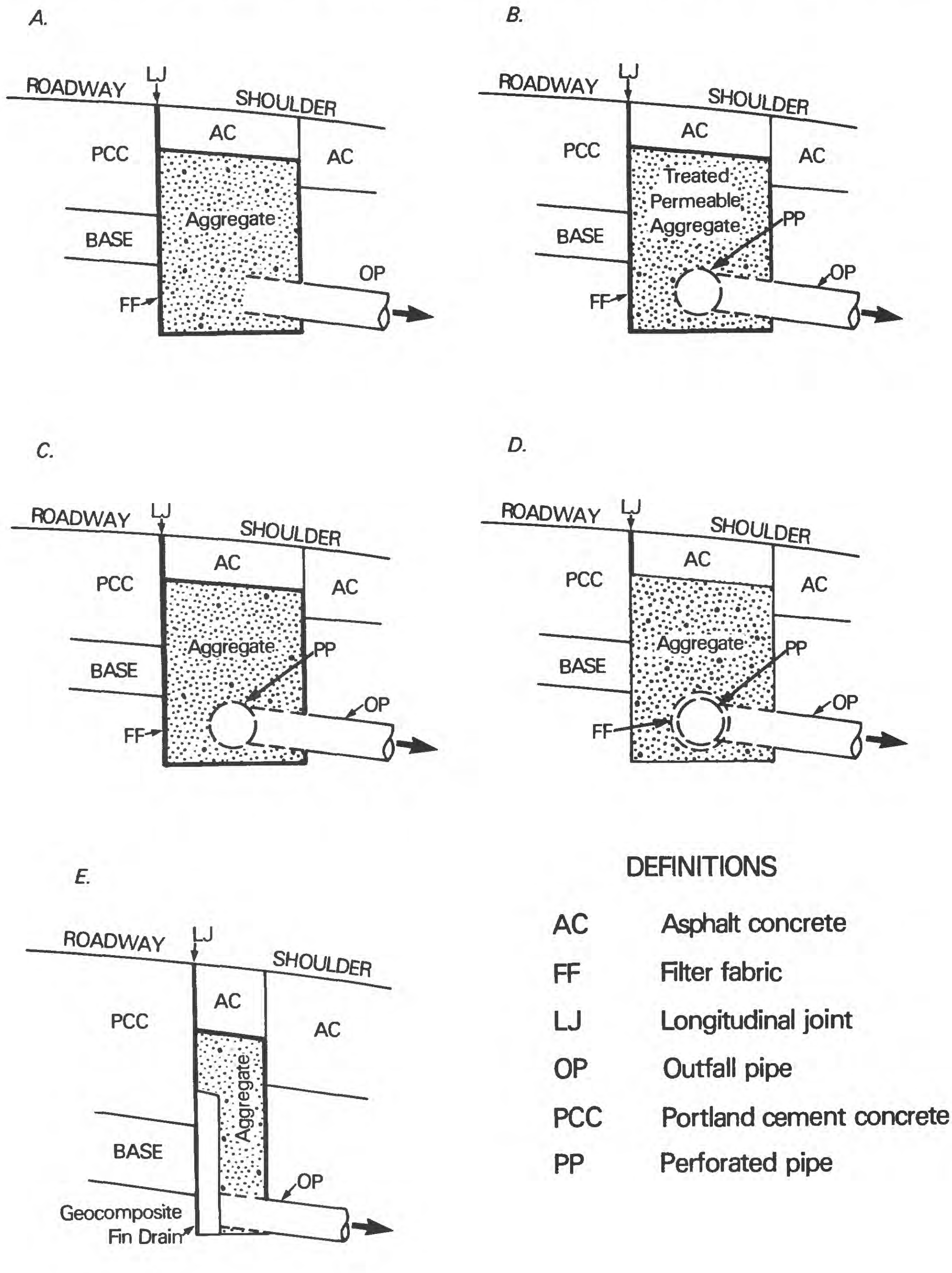

Figure 5A.-- Types of longitudinal edgedrains.

Figure 5B.--Preparing trench for installation of edgedrain.

Figure 5C.--Laying of perforated flexible drain pipe in edgedrain trench.

Figure 5D.--Rigid perforated PVC drain pipe and 4-inch diameter outlet with filter cloth prior to backfilling edgedrain trench.

Figure 5E.--Filling of edgedrain trench with aggregated prior to topping with asphalt.

DESCRIPTION OF STUDY SITES

The ten highway test sites were well distributed geographically and climatically (table 1) and 4 types of edgedrains were included (fig. 5A). At most of the sites, the installation of edgedrains was the only modification to the existing highway. At three sites (North Carolina, Oregon, and West Virginia), the addition of edgedrains was part of an overall rehabilitation program. At two sites (West Virginia and Oregon), rehabilitation consisted of installing edgedrains and adding 6 inches of asphalt overlay on the concrete pavement. This latter practice would be expected to influence infiltration into the roadways and hence discharge from the edgedrains.

Table 1--Description of edgedrain test sites

Location	Type of Edgedrain (a)	Pavement Section	Age, years (b)		Edgedrain Configuration		
			Highway	Edgedrain	Depth inches	Width inches	Outlet Spacing feet
Alabama, I-65 near Greenville	A	9 in. PCC slab 6 in. soil cement subbase 6 in. soil subgrade		3	18	12	600
Arkansas, I-40 near Ozark	C	9 in. in PCC slab 6 in. gravel subbase	17	4	18	12	300
California, I-5 near Willows	B	8 in. PCC 5 in. treated soil subbase	19	10	10	8	200
Illinois, I-57 near Rantoul	D	8 in. PCC slab 5 in. bituminous aggregate subbase	18	4	30	10	500
Minnesota, Trunk Highway 212 in Glencoe	D	9 in. PCC slab 5 in. dense graded aggregate subbase		4	15	10	350
New York, I-88 near Duanesburg	D No filter cloth	9 in. PCC slab 12 in. granular subbase	7	7	30	12	550
North Carolina, I-85 near Kannapolis	A 4 in. pipe in last 200 feet	9 in. PCC slab 3 in. aggregate subbase 6 in. dense graded subgrade	31 (c)	5 (c)	18	12	225
Oregon, I-5 at Coburg	C	6 in. asphalt 8 in. PCC	28 (d)	13 (d)	18	8	550
West Virginia, I-77 near Ripley	A	6 in. asphalt 9 in. crushed PCC slab	(e)	1 (e)	12	10	500
Wyoming, I-25 north of Wheatland	C	8 in. PCC slab 5 in. gravel subbase	14	2	19	6	600

(a) See figure 5A
(b) As of 1989
(c) Highway also rehabilitated for 5 years
(d) Highway also rehabilitated for 13 years
(e) Highway also rehabilitated for 1 year

STUDY APPROACH

In a number of studies of the performances of highway edgedrains, rainfall and runoff have been measured concurrently (Dempsey and Robnett, 1974). This data, although very revealing, does not define the hydraulics of the flow through, under, and away from the highway pavement. For this study, therefore, it was desired that hydraulic conditions under the pavement also be measured if possible. This would involve measuring water levels, or more correctly, piezometric heads and gradients under the pavement as well as soil moisture conditions once saturated flow, if any, ceased to exist. Piezometric head is the pressure, measured in feet of water, that exists whether or not water is actually present. Such measurements would permit an understanding of how and where water entered the subbase and its mode of drainage. The effectiveness of edgedrains would be proven if it was shown they expedite the flow and removal of water from beneath the pavement, minimizing the time and degree of saturation of the subgrade material.

Instrumentation and Data Collection

A variety of instruments were tested and compared at a preliminary site in Alabama before selecting those to be installed at all ten test sites.

Rainfall and Runoff Gages

Large tipping-bucket gages have been used by others to successfully measure very small flows (Wilford, 1984, Helvey and Fowler, 1980, Hollis and Ovenden, 1987, and Edwards and others, 1974). Aside from simplicity and accuracy, the ease of recording tips of a bucket on a data logger made such an instrument a logical choice for this study. A minimum of 10 such units would be needed. Edgedrain discharges might be expected to range from 0-5 gpm (gallons per minute). Some investigators have sized each bucket to measure the specific range of flows expected for their study site. Nevertheless, for this study, it was desirable to mass-produce the units and make one size bucket serve to measure most edgedrain discharges. Very large discharges from edgedrains would be measured by an overflow weir.

Experience indicated that rainfall needed to be measured on site and concurrently with edgedrain discharge, if possible, because in some instances, rainfall amounts from a given storm can vary drastically in just a few hundred feet. Thus, dual tipping buckets were installed side-by-side, one for measuring rainfall and the other to measure edgedrain discharge.

To expedite production and minimize cost, one standard size combination was selected. The two buckets have the same shape but differ in width and hence volume; the bucket receiving the edgedrain discharge being much larger than that receiving the rainfall. Figure 6 shows the design of the dual tipping bucket incorporated into a wooden shelter that also contains the data logger and power supply for all the instruments. Figure 7 is a photograph of the same unit. A PVC pipe secured to the outlet pipe diverts the edgedrain discharge to the house (fig. 8). Referring to figure 6, the edgedrain discharge (6) diverted from its outlet spills into the runoff bucket (7); large flows are allowed to spill over a weir (8). This bucket has an effective volume of approximately 2.15 liters. The bucket receiving the rainfall has an effective volume of approximately 0.190 liters. All tipping buckets were calibrated at the USGS's Gulf Coast Hydroscience Center (GCHC) located at the National Space Technology Laboratories (NSTL), Mississippi. Although the calibrations varied slightly for each unit, a general equation for the runoff buckets is:

$$Q_b \text{ (gpm)} = 0.076 \text{ [tips per minute]}; \qquad (1)$$

The weir flow can be expressed by the general equation:

$$Q_w \text{ (gpm)} = 0.3 \text{ [tips per minute} - 6.2]^{2.25}. \qquad (2)$$

The flow over the weir is added to that measured by the bucket. Note that weir flow was calibrated versus the tips of the bucket, thus no weir head measurements were required. Equations for the weir overflow discharge varied widely, but the relation for each unit was known from individual laboratory calibrations.

A general equation for the rainfall bucket with the 17-inch diameter funnel (fig. 6) is:

$$Q \text{ (inches per hour)} = 3.1 \text{ [tips per minute]}. \qquad (3)$$

This results in one bucket tip being equal to approximately 0.05 inches of rainfall.

The tips from each bucket activated the magnetic switch (11) and were recorded on a Campbell Scientific Model SR10 data logger installed high in the instrument house.

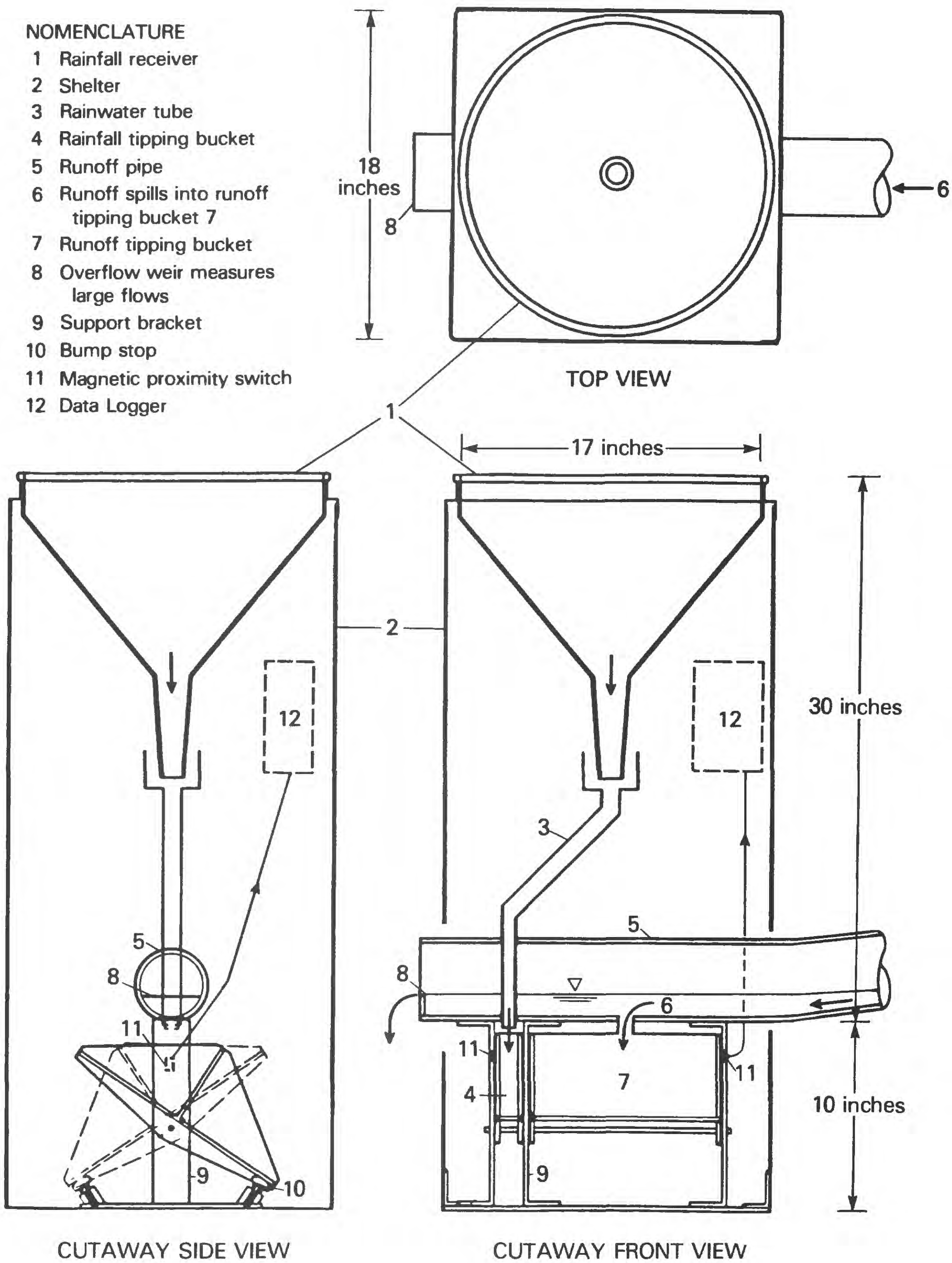

Figure 6.--Design of dual tipping bucket.

Figure 7.--Dual tipping bucket and instrumentation shelter; edgedrain discharge enters from right, spills into large bucket with any excess overflowing weir on left and out 90° elbow.

Figure 8.--Diversion of edgedrain discharge to tipping bucket using PVC pipe secured to outlet pipe.

Water Measurements Beneath Pavement

The test program involved the measurement of water in the saturated and unsaturated state beneath the concrete pavement in one of two traffic lines and under the adjoining shoulder. This test section was to be 1-foot up-gradient from a transverse joint and near an edgedrain outlet. It was expected that voids under the pavement might be more readily intercepted and water levels in them measured if the measuring section was placed parallel and close to a transverse joint. It is along these joints that most initial pavement failures have been observed to occur.

Selecting a transverse joint close to an edgedrain outlet also expedited placing all sensor lines in a shallow slot in the pavement and then routing them to the nearby instrument house. This same house also contained the dual tipping bucket, data logger, and power supply (fig. 7). Figures 9 through 17 illustrate the designation, drilling, placement, and sealing of the various sensors below the pavements.

Piezometric Water Levels.--As a result of the preliminary tests, the Druck PDCR 830 submersible pressure transducer (fig. 13) was chosen for measuring piezometric water levels in the subgrade beneath the pavement. The Druck PDCR 830 is designed for measurement of water depth in small-diameter bore holes. The waterproof assembly has a molded polyurethane-sheathed integral cable to transmit the signal to the data logger. Each transducer was calibrated in a water filled standpipe at the GCHC. The pressure range for the transducer used is 0 to 2.5 pounds per square inch (psi), which is approximately equivalent to 0 to 5.5 feet of water. Figure 18 is a cross-sectional view of the Alabama site showing the transducer hole placement and elevations.

After drilling 20- to 24-inch-deep core holes, each was vacuumed clean, the transducer was placed in the bottom of the hole and the hole backfilled with washed pea gravel (fig. 14). The signal lines for each were then led through a 1-inch wide by 2-inch deep slot cut transversely in the concrete and adjoining shoulder (fig. 17). The depth of each transducer below the concrete pavement surface was recorded. An engineer's level was then used to determine the relative elevations and slope of the highways, edgedrain, and sensors. The bottom of the edgedrain trench was taken as zero datum for a given site and each transducer reading subsequently adjusted during data processing.

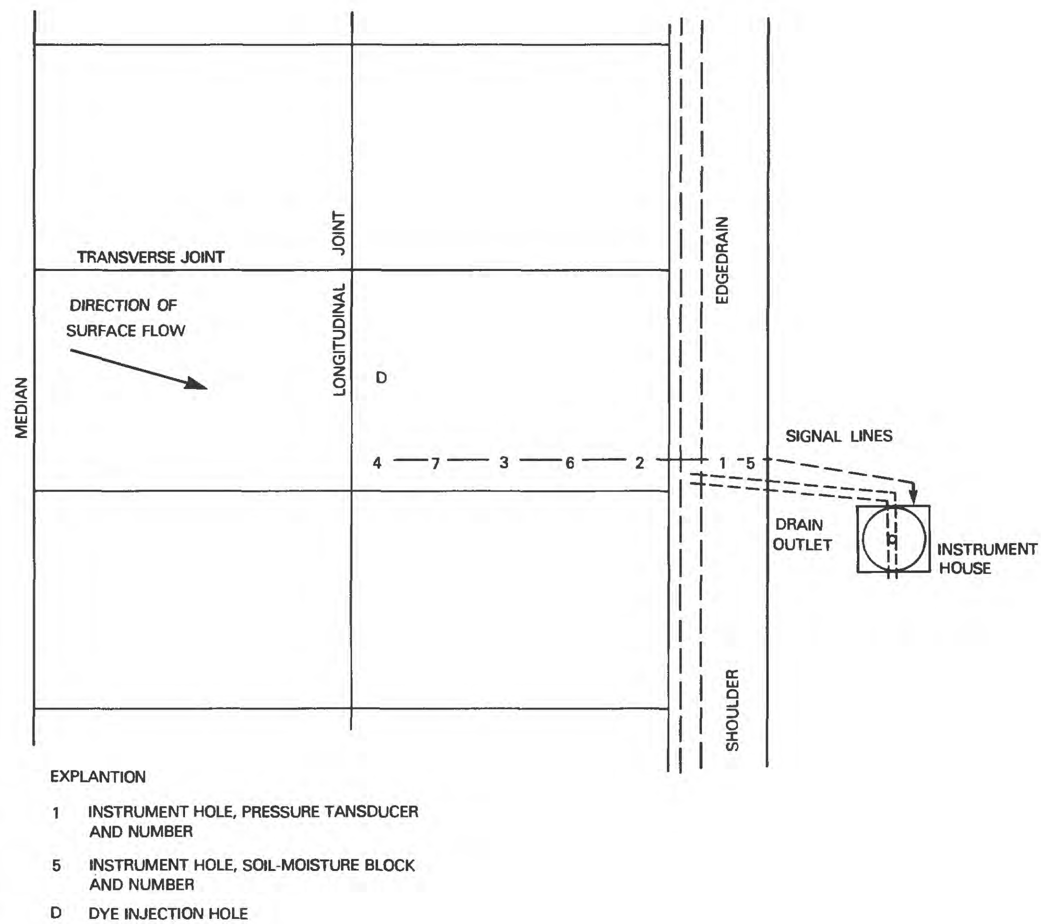

Figure 9.--General instrumentation layout for ten highway test sites.

Figure 10.--Designation and location of drill holes and signal line slot for placement of sensors under pavement

Figure 11.--Drilling of 4-inch diameter sensor holes through concrete pavement.

Figure 12.--Cutting slot in concrete pavement for sensor lines.

Figure 13.--Transducer prior to installation below pavement.

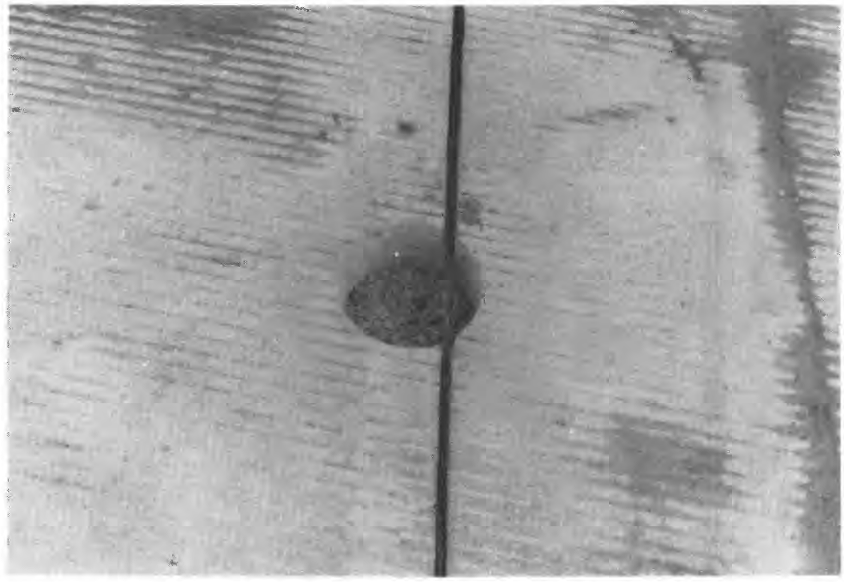

Figure 14.--Sensor installed in core hole and backfilled with pea gravel; signal lines implaced in slot.

Figure 15.--Soil-moisture block prior to placement just below the underside of pavement.

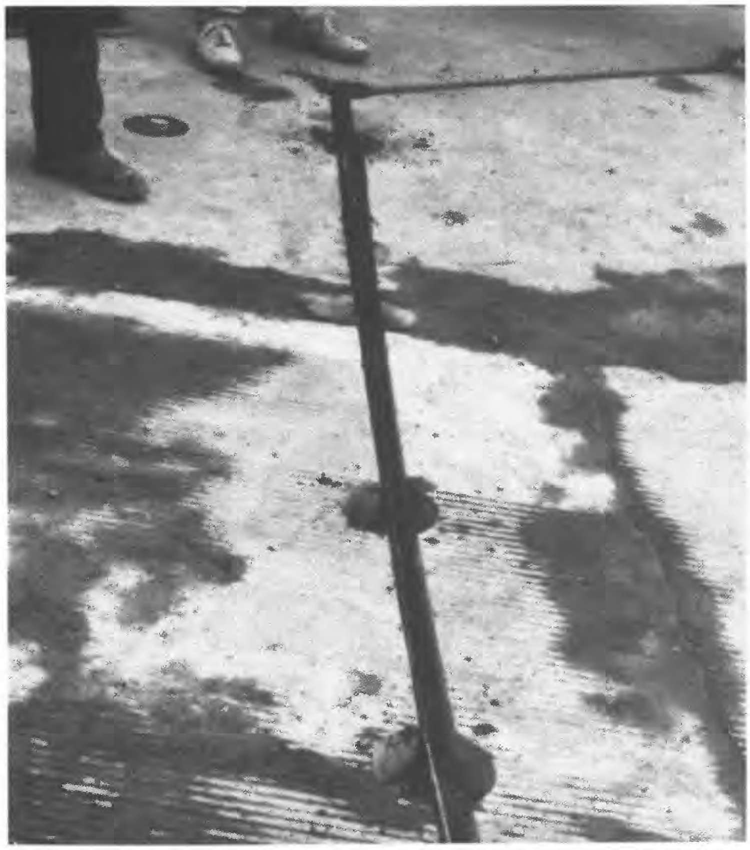

Figure 16.--All sensors in place just prior to filling holes and signal line slot.

Figure 17.--Completed installation with sensor holes and signal line slots sealed with epoxy cement.

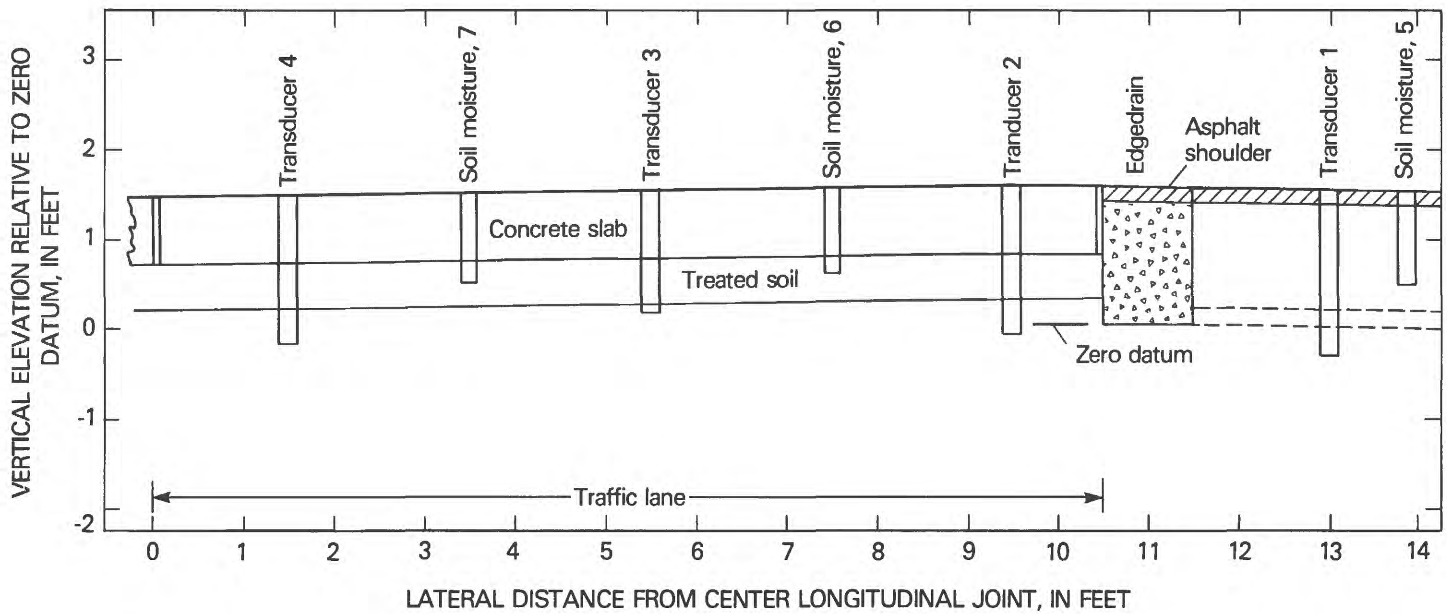

Figure 18.--Cross-sectional view of Alabama site showing locations of sensors under pavement.

Soil Moisture.--Gypsum soil-moisture blocks (fig. 15) provide a method for estimating soil moisture by measuring the negative soil pressures in unsaturated soils. Given sufficient time, the gypsum block assumes the same moisture level as the soil surrounding it. Two electrodes in the block measure resistance, which varies with water content and can be calibrated for each block. Figures 9 and 18 show the location of the soil moisture blocks in each highway test section.

Because soils have different water pressure-soil moisture characteristics, each soil must be tested and calibrated in a laboratory. During the drilling of the holes for the various sensors, core samples were collected at the bottom of selected holes by forcing a standard 3-inch diameter metal core sampler into the subgrade material. The samples were sealed at both ends of the tube and preserved for laboratory analyses. For the Alabama, Arkansas, and North Carolina study sites, two samples were obtained, one each at points 1 and 3 (fig. 9), and for the remaining sites, one sample was taken at point 3. A series of laboratory analyses were made of each sample to determine the relative soil and moisture characteristics at each location. The soil moisture-water pressure calibrations as measured at each location, in conjunction with the calibrated soil moisture blocks, provided a means of monitoring the soil moisture levels under the pavements.

Water-Tracer Tests.--Upon completion of the installation of the sensors, a single 1-inch diameter hole was drilled up-gradient, adjacent to the center longitudinal joint (fig. 9) and into the subgrade. This hole was not selected at random and did not necessarily intercept any voids. Approximately 50 mL of Rhodamine WT (water tracer) dye was poured into each hole and the hole immediately sealed. Laboratory measurements of dye concentrations were made on samples from the edgedrain tipping bucket. Project personnel were instructed to note any sign of the dye at the site with each visit.

Temperature Measurements Beneath Pavement

A single temperature sensor was placed in hole number 6 to record temperatures continuously.

Following the placement of all sensors and signal lines, the holes and slots in the pavement were sealed with epoxy cement (fig. 17).

Data Storage and Retrieval

The Campbell CR 10 data logger receives and stores the input from the other sensors (see fig. 6 and 9). Because continuous operation of the sensors was impractical due to power consumption and the limited storage capacity of the data logger, it's program software was structured to operate in an intensive data collection mode (at 5-minute intervals) during a rain event and gradually reducing in frequency thereafter. The program was initialized by a tip of either the rainfall or discharge tipping bucket.

The CR 10 data logger contains an internal data-storage capacity of 5800 data values, which automatically unloads to a separate storage module with an 88,000 data value capacity. This module is "off-loaded" or retrieved by the observer periodically. The data were then transmitted to the USGS office in Tuscaloosa, Alabama, where it was analyzed and plotted.

DATA ANALYSIS AND INTERPRETATION

The following sections will address the various aspects of water movement into, through, and out of the highway pavements, subgrades, and edgedrains.

Rainfall and Edgedrain Discharge

Figures 19 through 28 are plots of the rainfall and edgedrain discharge and piezometric water level hydrographs for the ten highway sites as collected in 1989 and 1990. Not all data acquired at these sites are presented; in some instances, rainfall was erratic and the resulting hydrographs difficult to interpret or repetitious. Fifty rainfall events have been identified ranging from 0.05 inches to a maximum of 2.95 inches total volume. The 50 events are tabulated in table 2 and labeled in figures 19 through 28. The maximum discharge measured was 5.5 gpm from the edgedrain at the New York site, event 31, and was produced by a relatively small total rainfall volume of 0.50 inches.

The most striking feature of these data is the almost instantaneous discharge from the edgedrains with the occurrence of rainfall. Table 2, column 5 shows in almost all instances that discharge starts in less than 1 hour from the occurrence of rainfall.

Not only does this show that the edgedrains begin to immediately drain water off from under the pavement but also, unfortunately, the joints leak badly and extensive voids and channels exist to transmit water to the edgedrains very quickly. It must be concluded that edgedrains can drain off only the water reaching them be it via interior longitudinal and transverse joints or the longitudinal joint between the pavement edge and the shoulder. The immediacy of runoff to rainfall would not be expected if flow were through the dense aggregate base; it must be via voids and channels that have developed.

Examination of these discharge hydrographs reveals wide differences in the length of time water discharges from beneath the pavements. Column 6 of table 2 shows similar recession characteristics for certain sites for all rainfall events regardless of rainfall volumes or intensities. For example, the Arkansas and Minnesota sites required 1 to 1 1/2 days to drain, whereas the California, Illinois, and North Carolina sites drain rather quickly, usually in 6 to 12 hours.

Table 2-- Response time, duration of potential pavement saturation, and duration of drainage from edgedrains for selected rainfall events

Event	Location	Date Year Mo. Day	Rainfall Volume, inches	Elapsed Time From Start of Rainfall to Start of Edgedrain Discharge, hours	Elapsed Time From End of Rainfall to Cessation of Edgedrain Discharge, hours	Period of Time Piezometric Water Levels are (a): Above Pavement Surface hours	Above Bottom of Pavement hours	Remarks (b)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	AL	1989 Feb 21	2.20	1	8	0	7	D
2		1989 Feb 21-22	0.05	0	11	0	3	W
3		1989 Mar 5- 6	0.75	0	19	0	4	D, Est. (6)
4		1989 Mar 29-30	2.25	1	23	0	7.5	D
5		1989 May 20-21	1.20	1.75	11	0	7.5	D
6	AR	1989 May 22-23	0.90	0	38	0	4	D
7		1989 July 15-16	1.10	0	34	0	1.5	D
8		1989 Sept 2- 3	1.40	0	26	0	1.5	D
9	CA	1989 Sept 28-29	0.75	1	6	0	15	D, Est. (6)
10		1989 Sept 29	0.20	0	14	0	14	W
11		1989 Nov 25-26	0.90	0.25	12	5	48	D, Est. (8)
12		1990 Jan 16	0.05	0	8	0	0	D
13		1990 Jan 16-17	0.35	0	16	8	(c)	W, Est. (6)
14		1990 Feb 3- 4	0.55	0	13	12	(c)	D
15		1990 Feb 6	0.05	0	20	(c)	(c)	W
16	IL	1989 Apr 28	0.90	0	7	0	11	D
17		1989 May 8- 9	0.45	0	13	0	6	D
18		1989 May 25	0.20	0	6	0	8	D
19		1989 May 25-26	1.25	0	14	0	13	W
20		1989 July 18-19	1.45	0	5	0	1.5	D
21		1989 July 19	0.05	0	4	0	0	W
22		1989 July 19-20	0.55	0	6	0	1.5	W
23	MN	1989 July 29-30	1.57	0	25	16	(c)	D, Est. (6)
24		1989 Aug 19-20	0.89	0	34	15	(c)	D, Est. (7)
25		1989 Aug 19-20	1.32	0	20	13	(c)	W, Est. (6)
26		1989 Aug 26-27	0.79	0.50	38	4	(c)	D, Est. (6)
27		1989 Sept 7- 8	1.13	0	27	2	(c)	W
28	NY	1989 July 4- 6	2.80	0.25	35	6	53	D, Est. (8)
29		1989 July 20-21	1.20	0	44	0	35	W
30		1989 Sept 14-15	0.70	0	30	0	4	D, Est. (6)
31		1989 Oct 2- 3	0.50	0	30	(c)	(c)	D

Table 2--Response time, duration of potential pavement saturation, and duration of drainage from edgedrains for selected rainfall events (Continued)

Event (1)	Location (2)	Date Year Mo. Day (3)	Rainfall Volume, inches (4)	Elapsed Time From Start of Rainfall to Start of Edgedrain Discharge, hours (5)	Elapsed Time From End of Rainfall to Cessation of Edgedrain Discharge, hours (6)	Period of Time Piezometric Water Levels are (a): Above Pavement Surface hours (7)	Period of Time Piezometric Water Levels are (a): Above Bottom of Pavement hours (8)	Remarks (b) (9)
32	NC	1989 May 15	0.40	0.25	10	12	(c)	D
33		1989 May 15	0.05	0	2	0	0	W
34		1989 July 19-20	0.10	0.50	6	8	(c)	D
35		1989 July 30-31	0.05	0	8	1	(c)	D
36		1989 Sept 22	2.95	0.25	9	20	82	D
37		1989 Sept 25-26	1.85	0	3	23	(c)	W
38	OR	1989 Aug 22	0.30	0	0.25	0	0	D, A
39		1990 Apr 26	0.15	--	--	(d)	(d)	D,A, No flow
40		1990 Apr 27	0.60	0	0.25	(d)	(d)	W, A
41		1990 June 6- 7	0.10	1	12	(d)	(d)	D, A
42	WV	1989 Aug 23-25	1.45	0	--	0	15	D, A
43		1989 Aug 24-26	0.25	0	>48	0	10	W,A, Est.(6)
44		1989 Oct 16-20	1.60	0	>48	0	35	D, A
45		1989 Nov 15-19	1.15	0	>48	0	21	W, A
46	WY	1990 May 4	1.40	--	--	13.5	(c)	D, No Flow
47		1990 May 20	1.90	0	2	1	19	D
48		1990 May 29	1.00	1	2	6	15	W
49		1990 May 30-31	0.10	0	5	3	13	D
50		1990 June 1- 2	0.70	0	5	4	18	D

(a) Longest duration based on 1 of 3 transducers beneath pavement.

(b) A - Asphalt topped
W - Wet antecedent condition
D - Dry antecedent condition
Est. (6) - refers to data in column estimated

(c) Indeterminate, piezometric water levels above pavement surface or bottom for several days.

(d) Transducers malfunctioned.

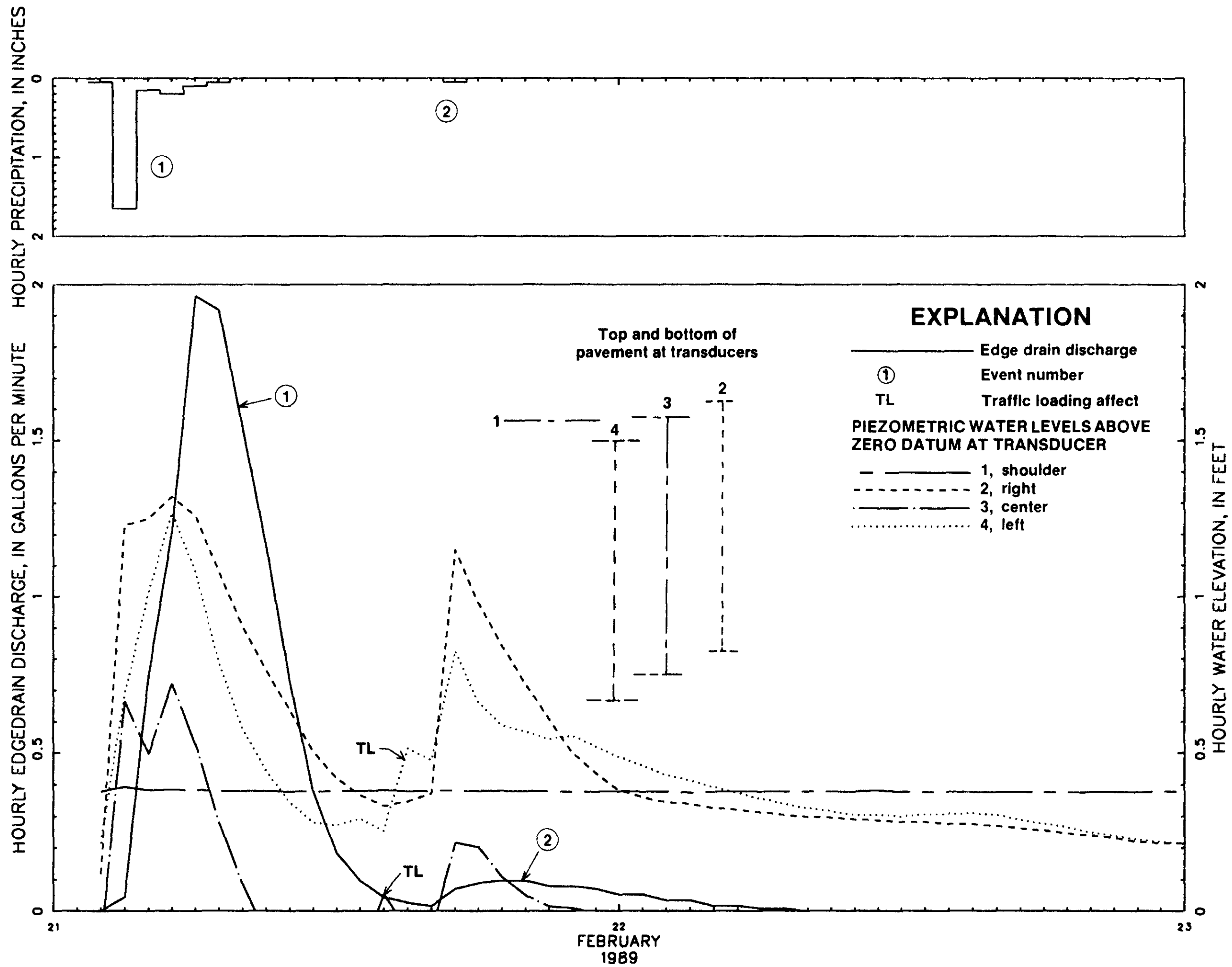

Figure 19A.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Alabama.

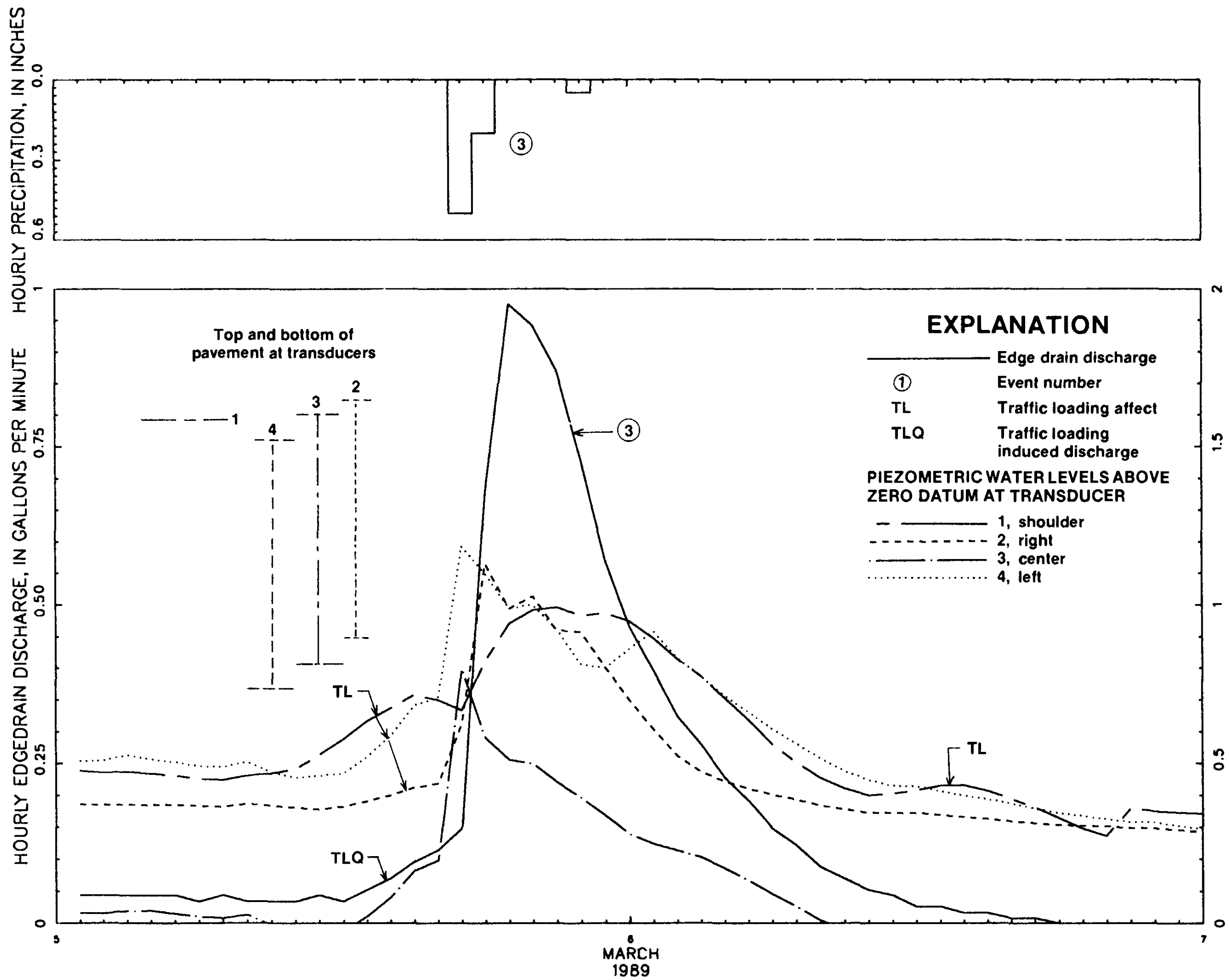

Figure 19B.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Alabama.

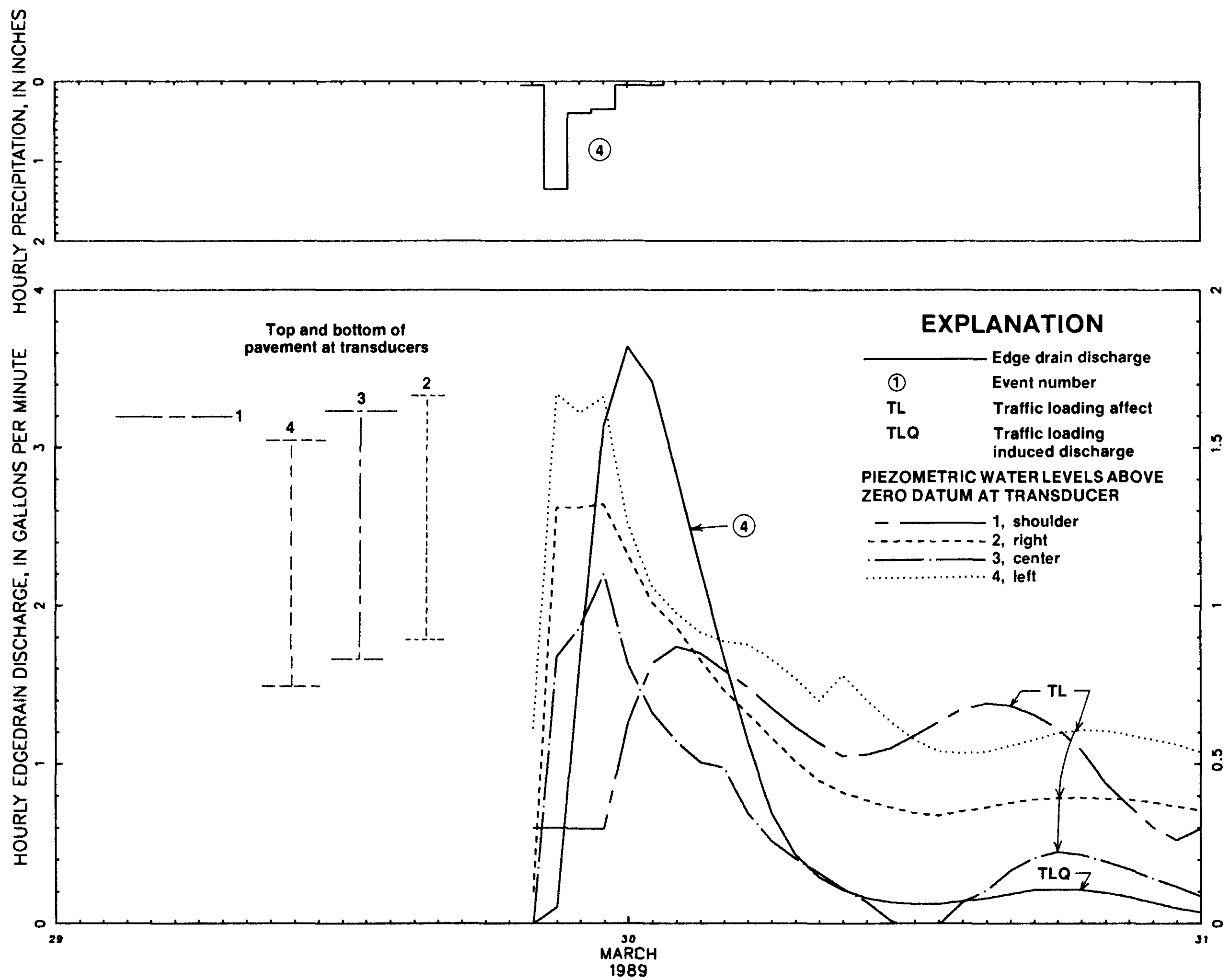

Figure 19C.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Alabama.

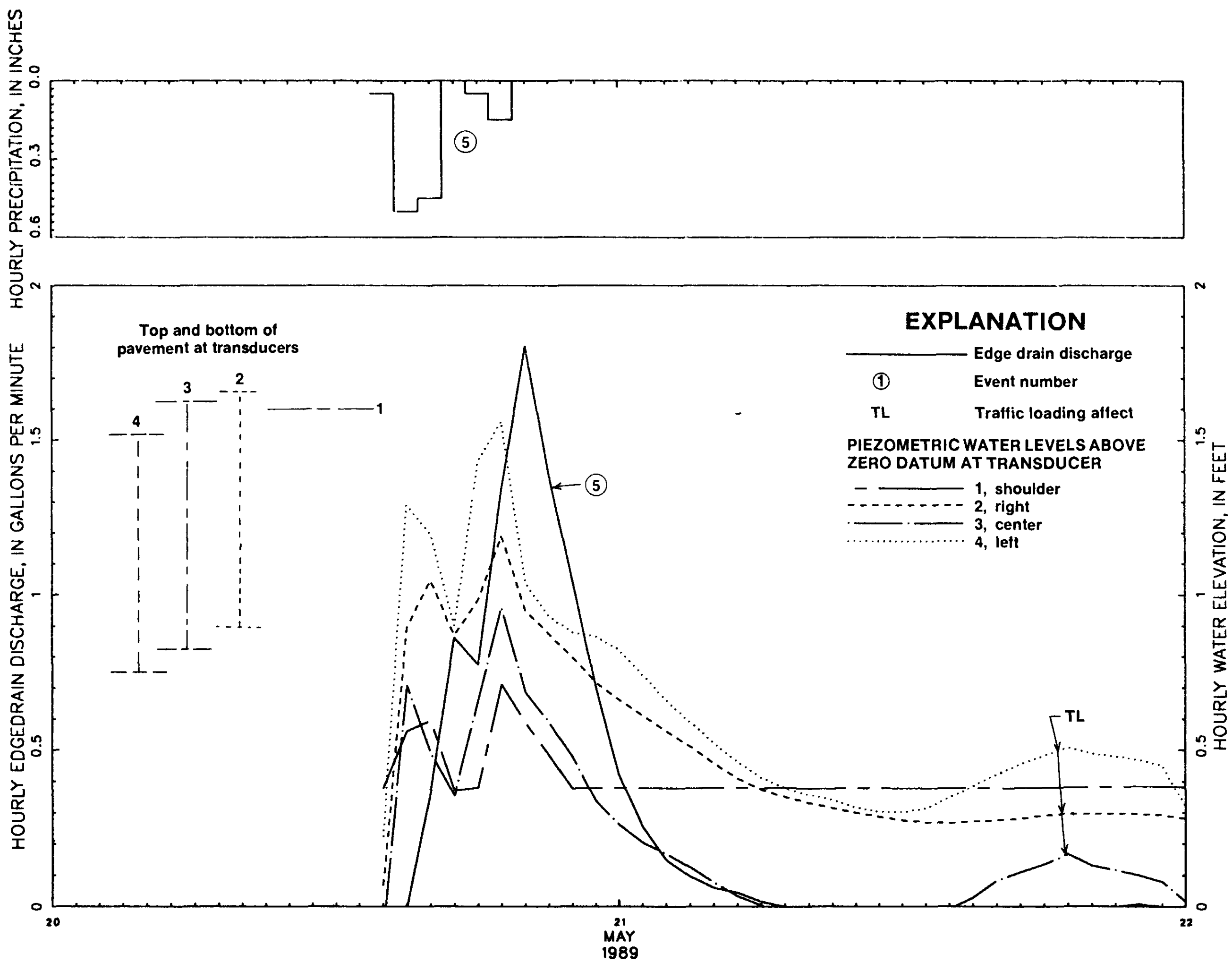

Figure 19D.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Alabama.

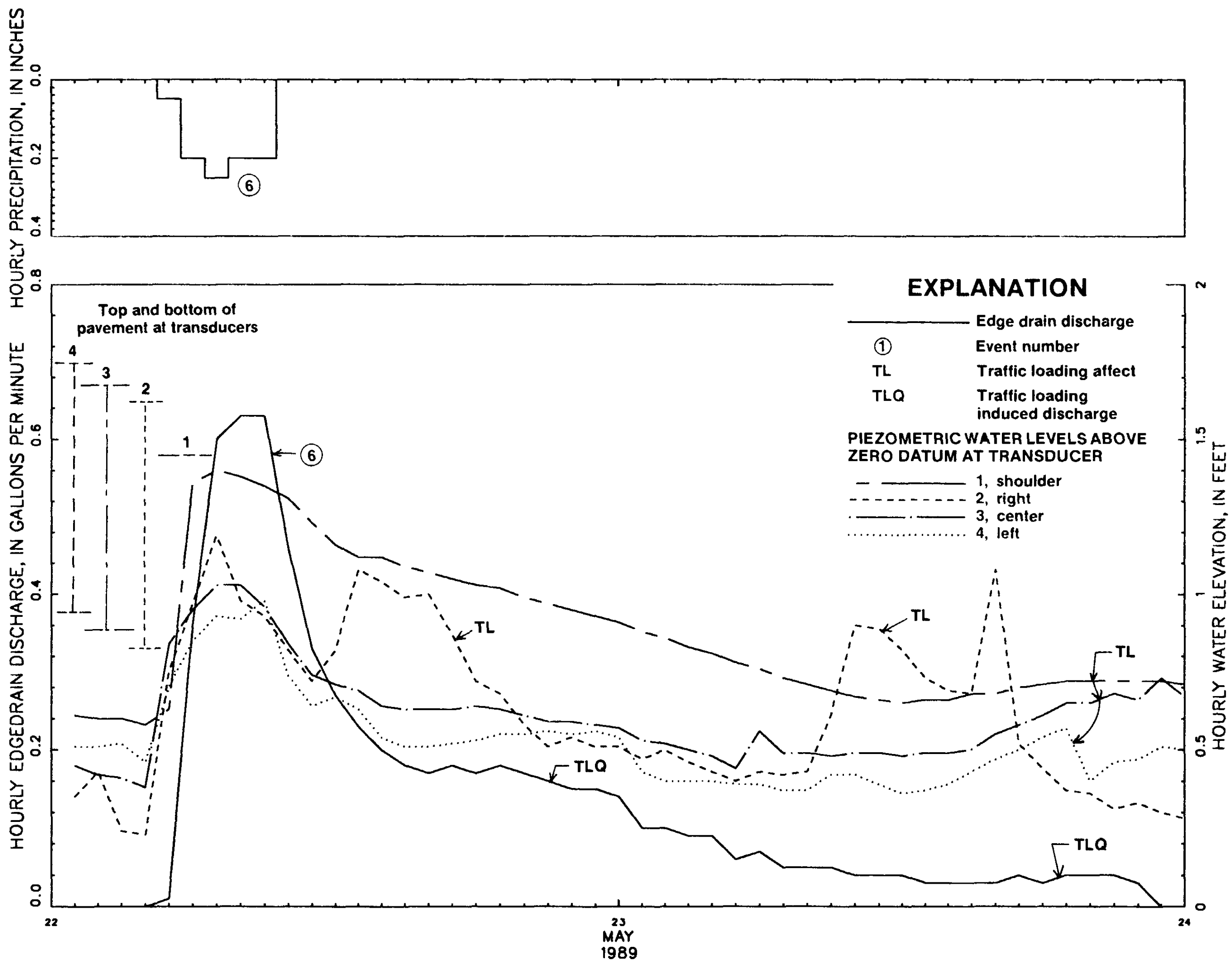

Figure 20A.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Arkansas.

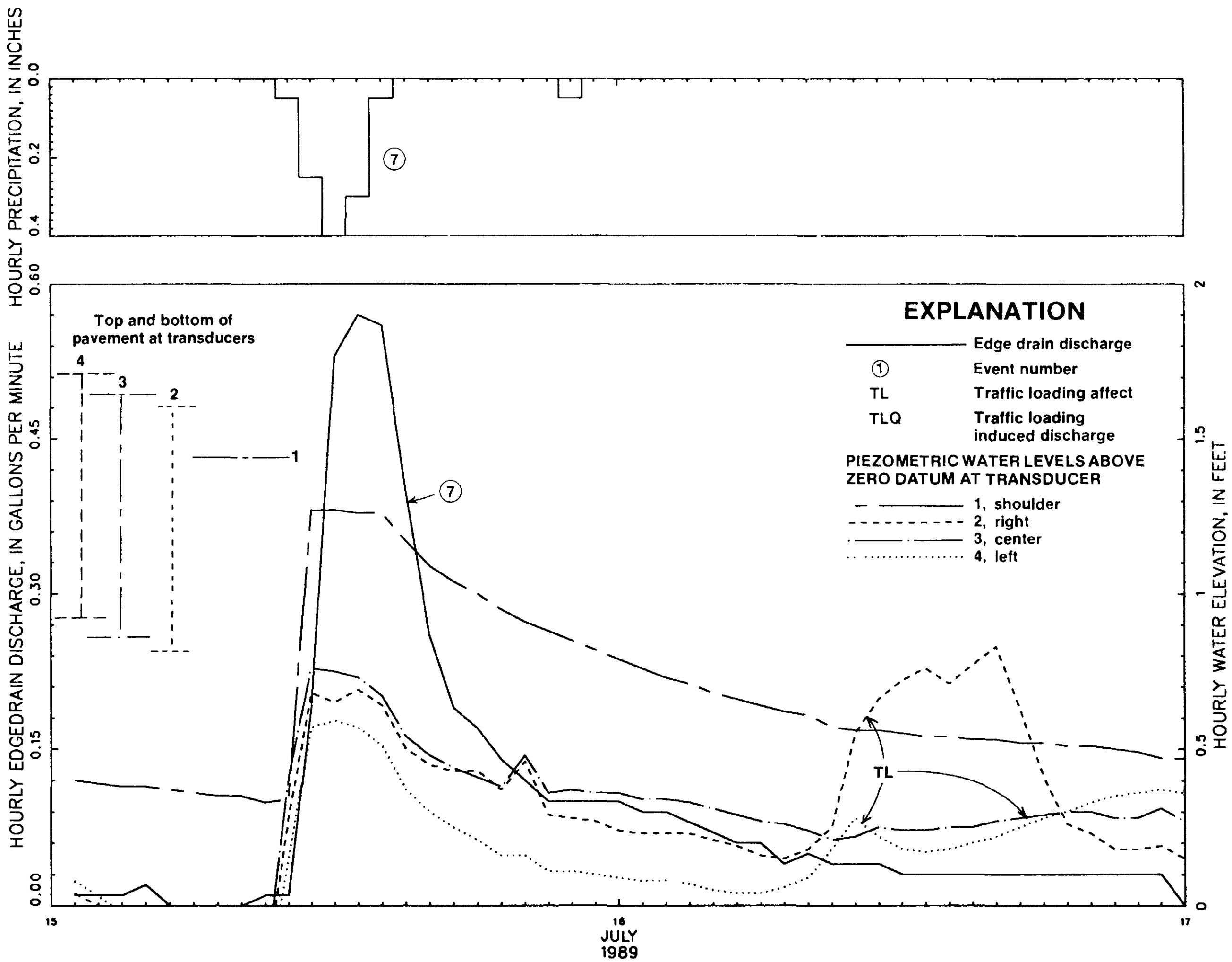

Figure 20B.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Arkansas.

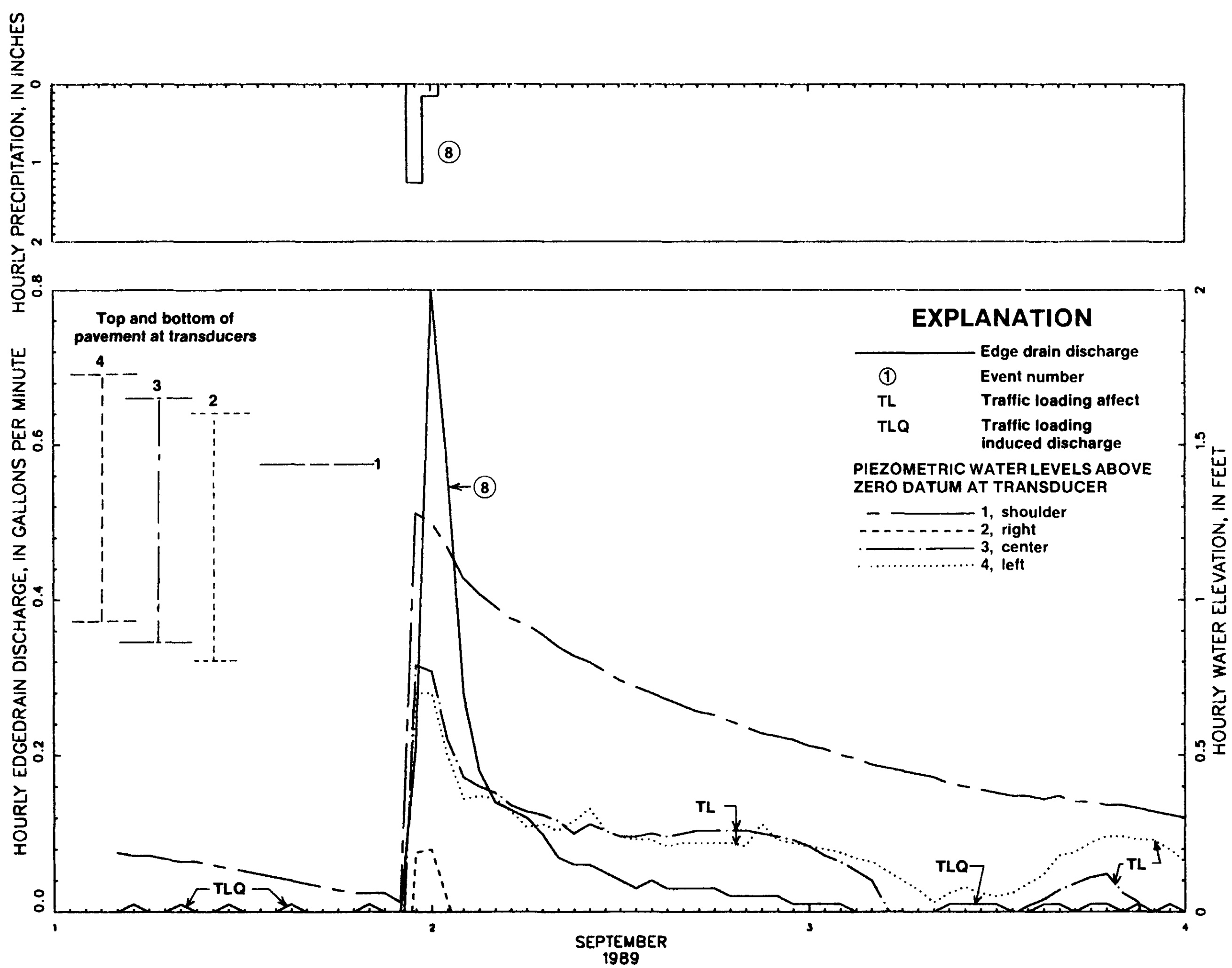

Figure 20C.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Arkansas.

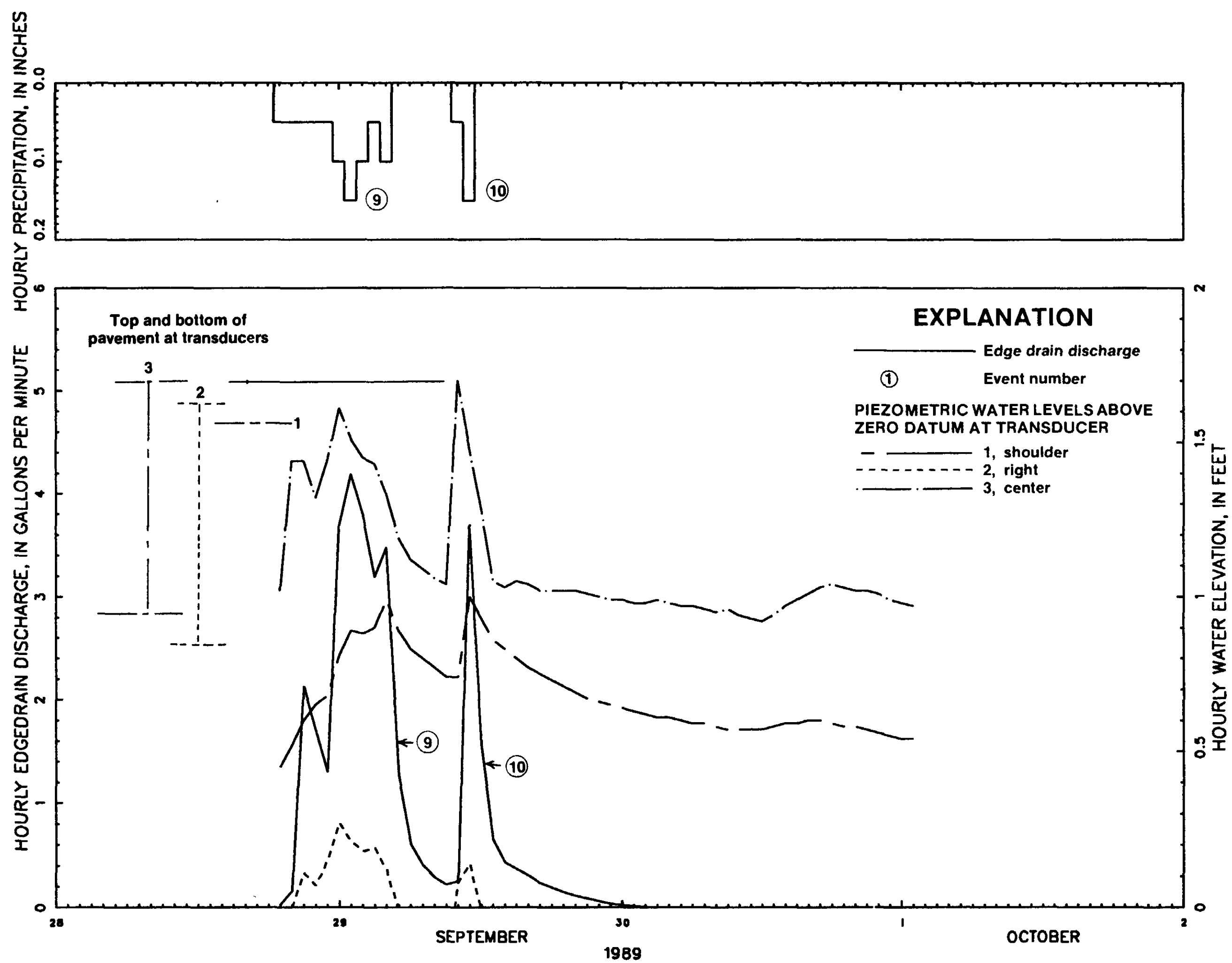

Figure 21A.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in California.

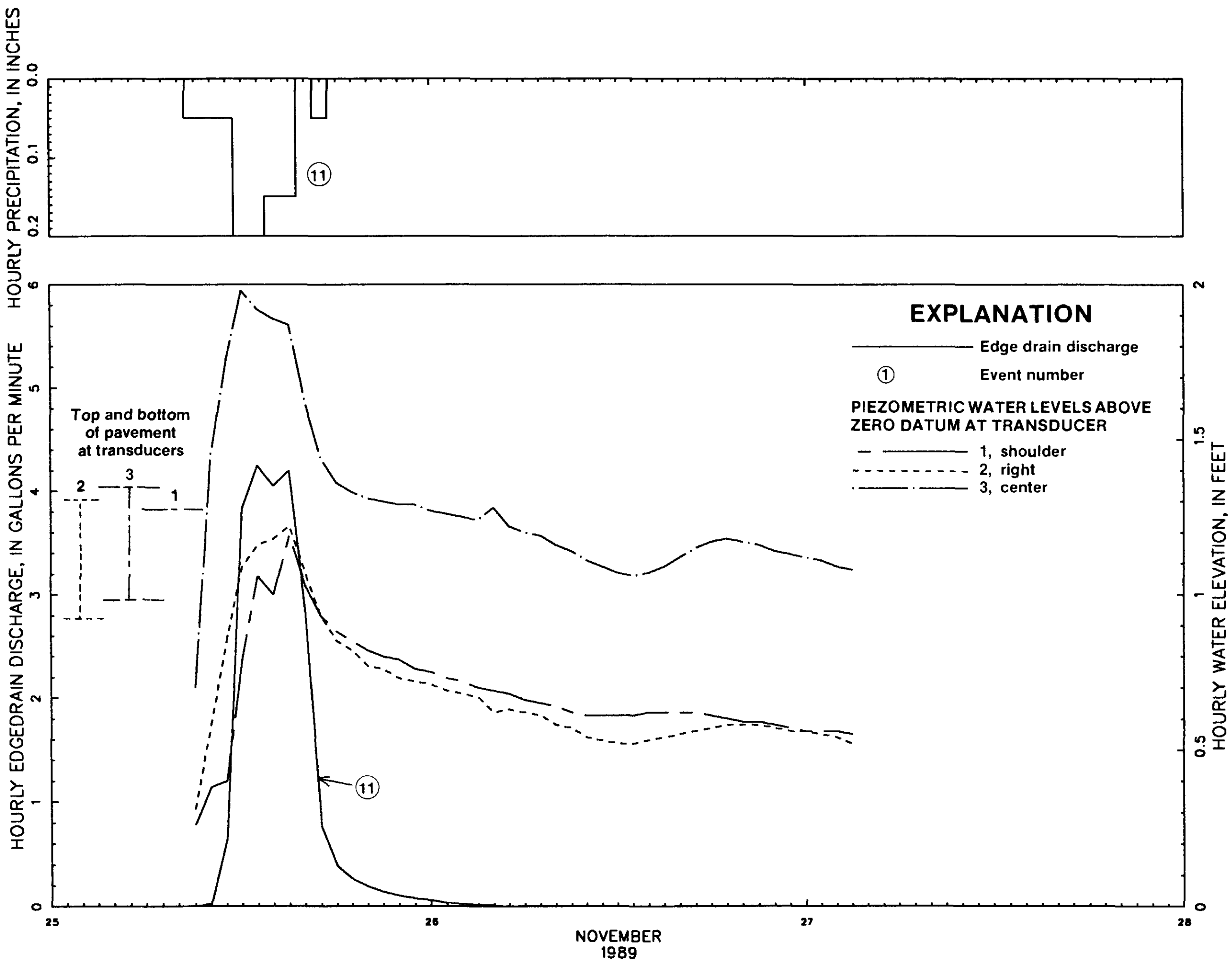

Figure 21B.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in California.

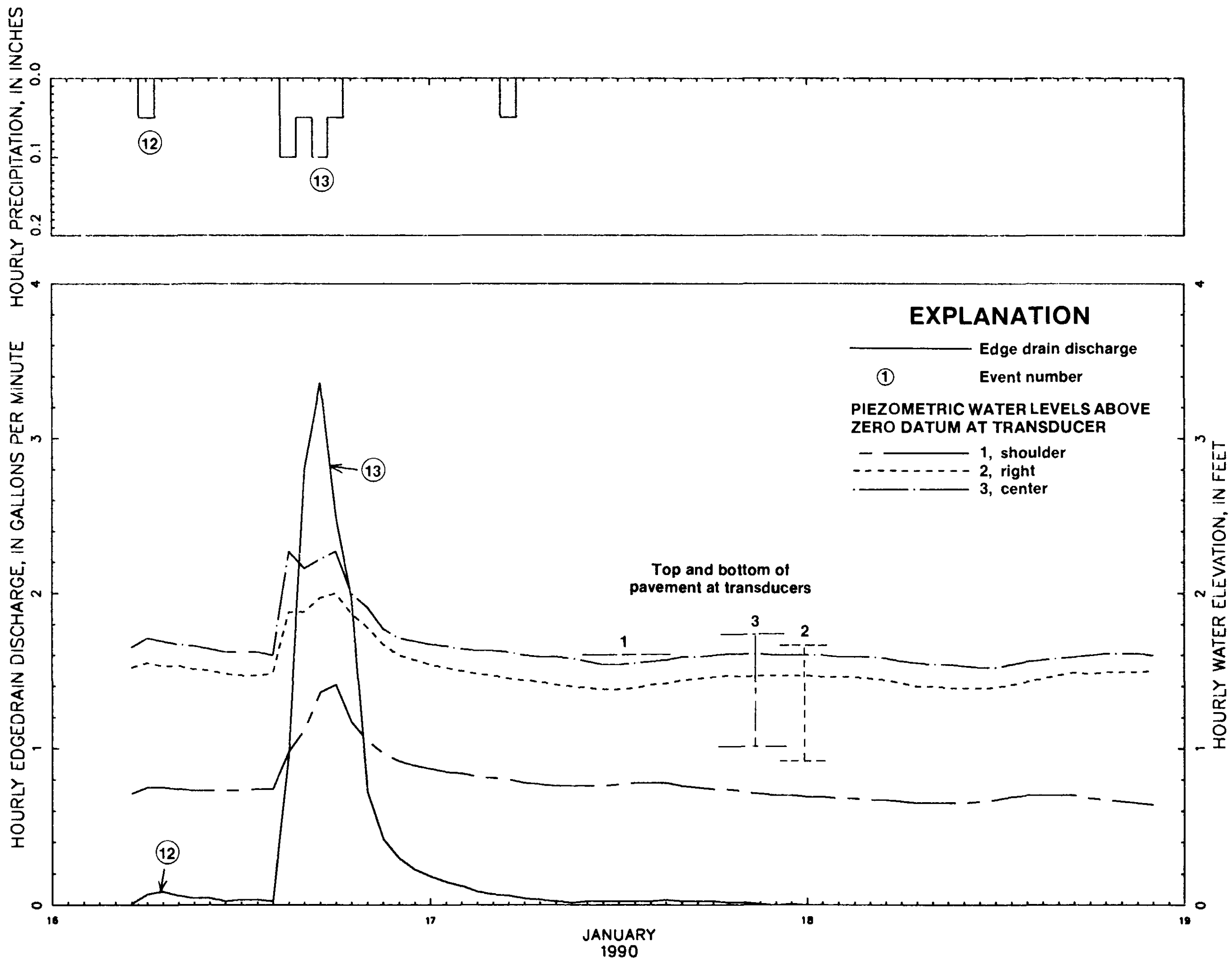

Figure 21C.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in California.

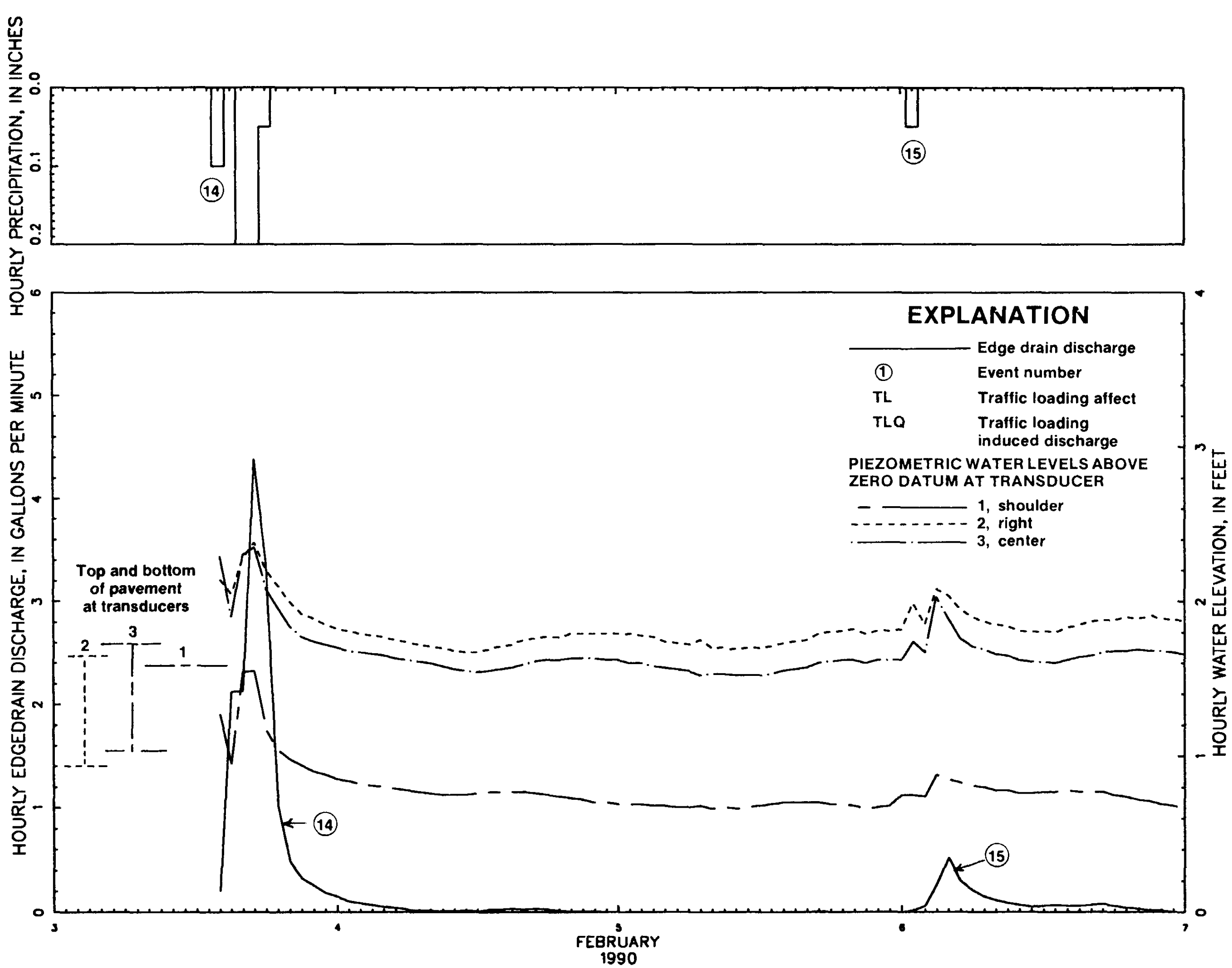

Figure 21D.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in California.

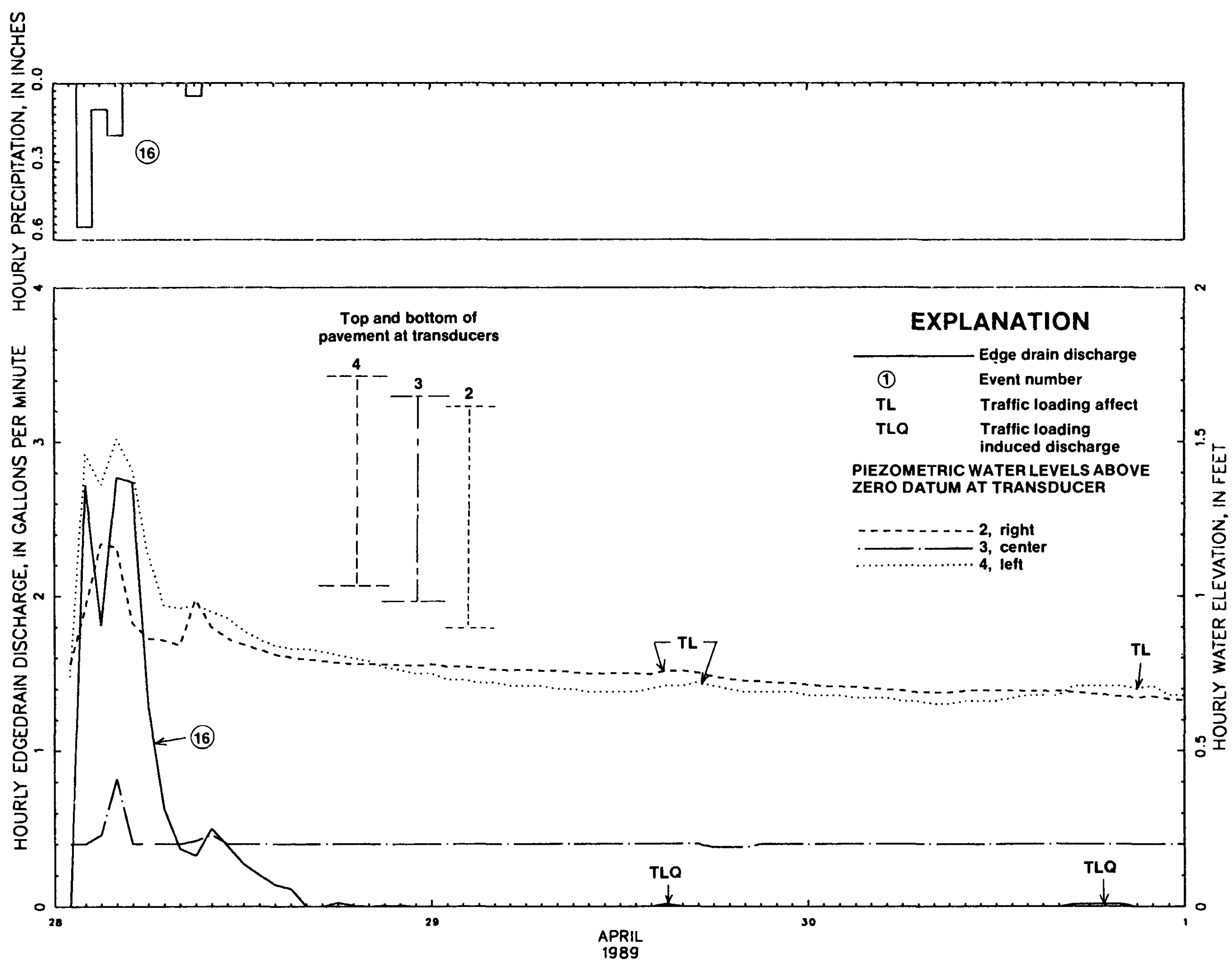

Figure 22A.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Illinois.

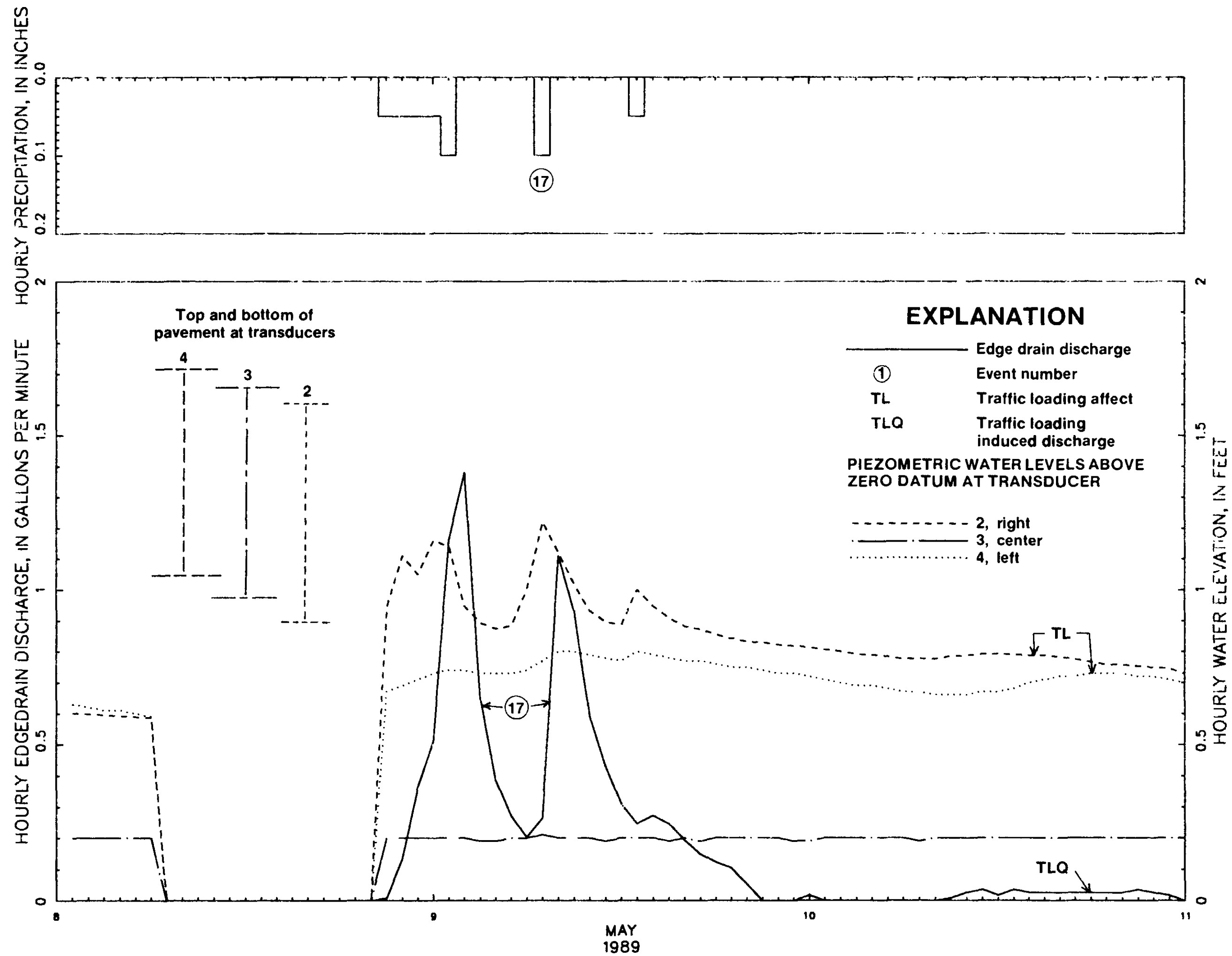

Figure 22B.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Illinois.

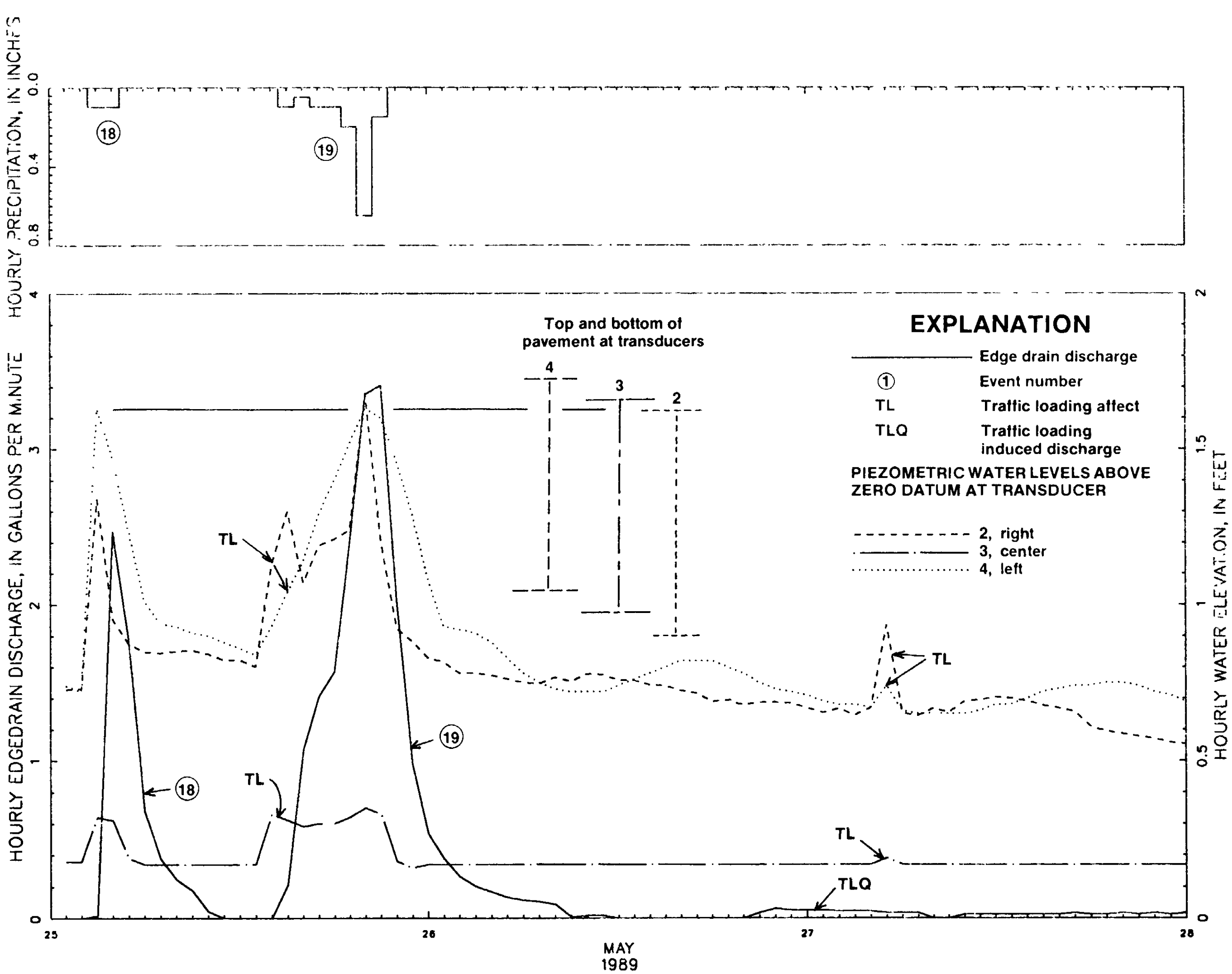

Figure 22C.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Illinois.

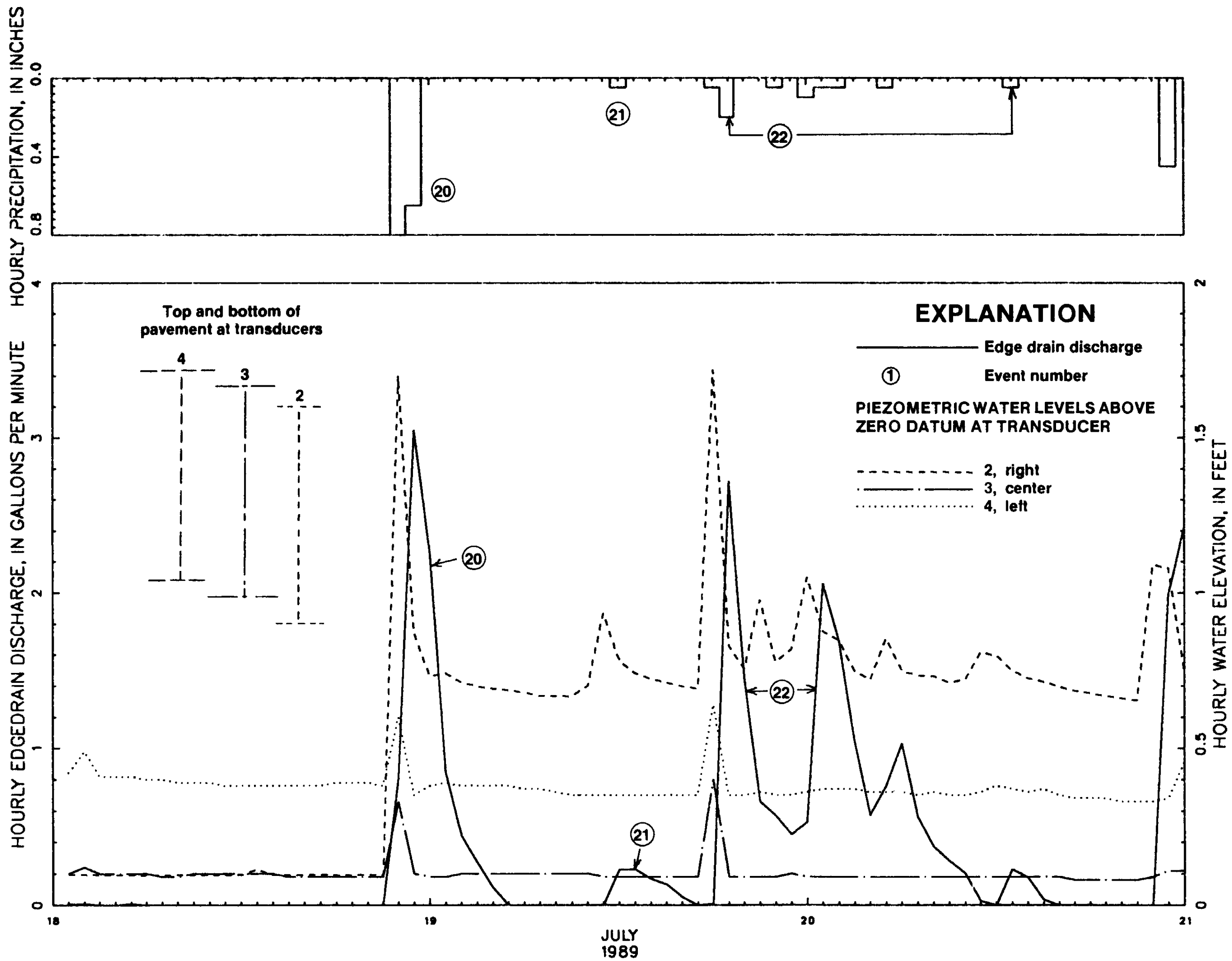

Figure 22D.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Illinois.

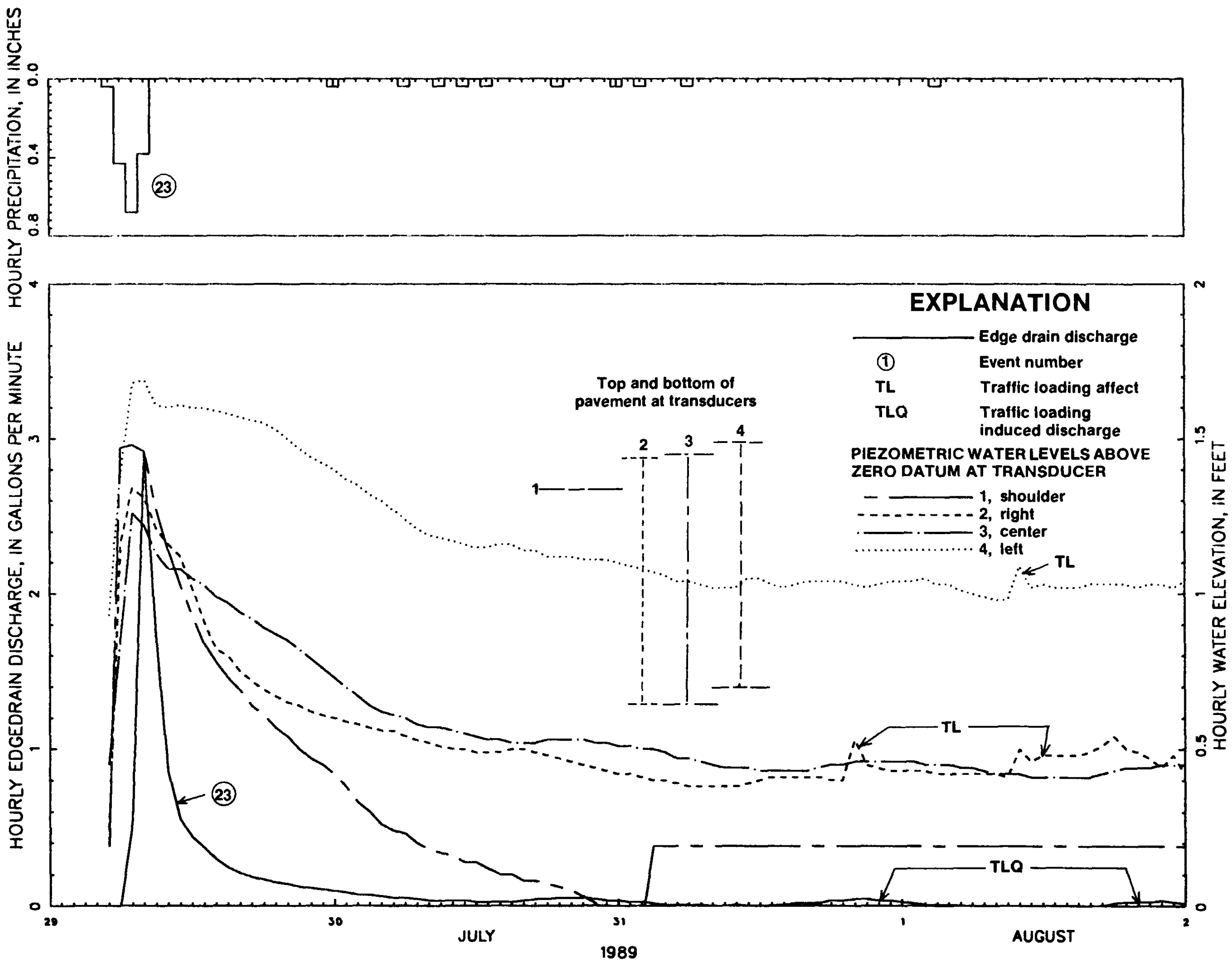

Figure 23A.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Minnesota.

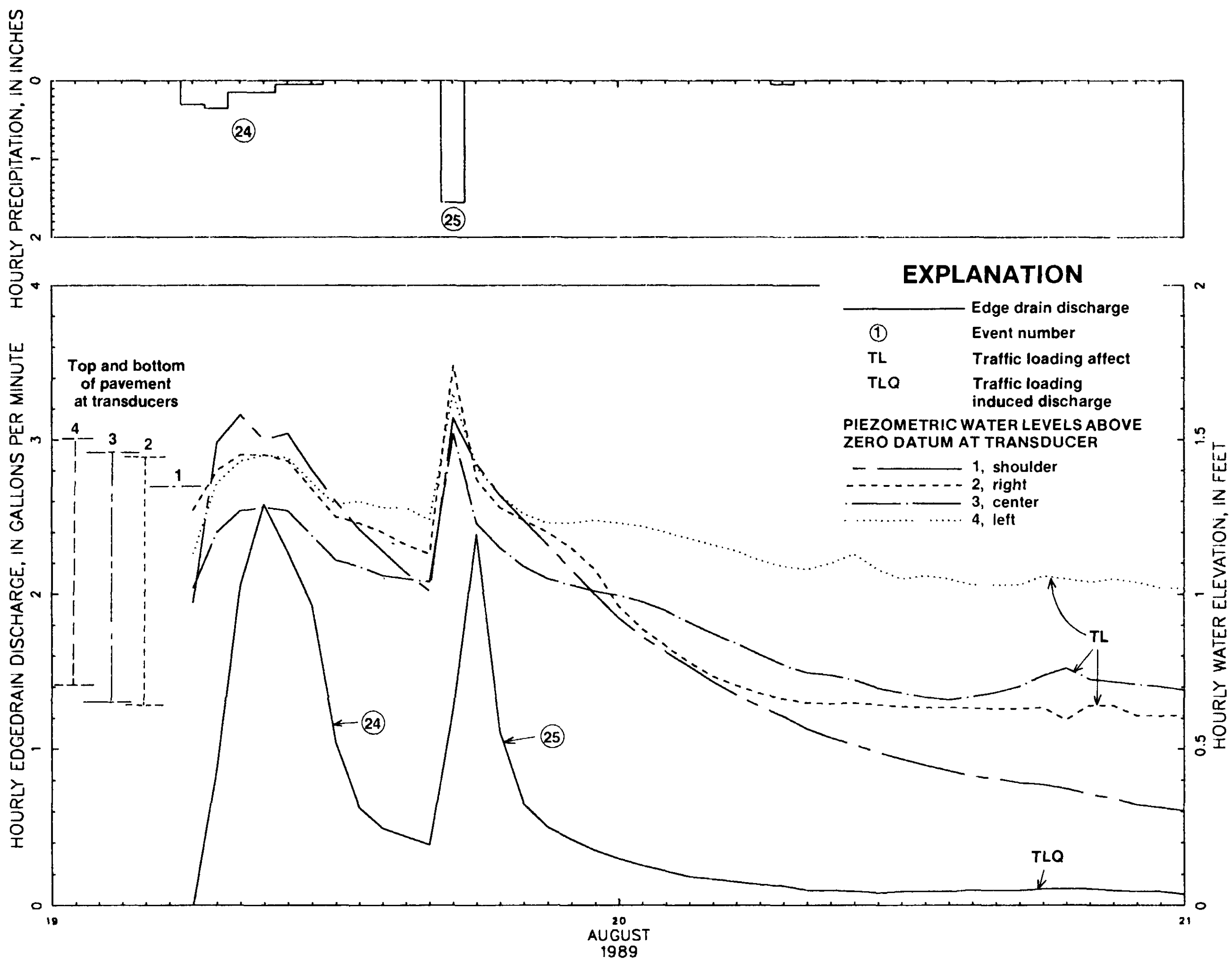

Figure 23B.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Minnesota.

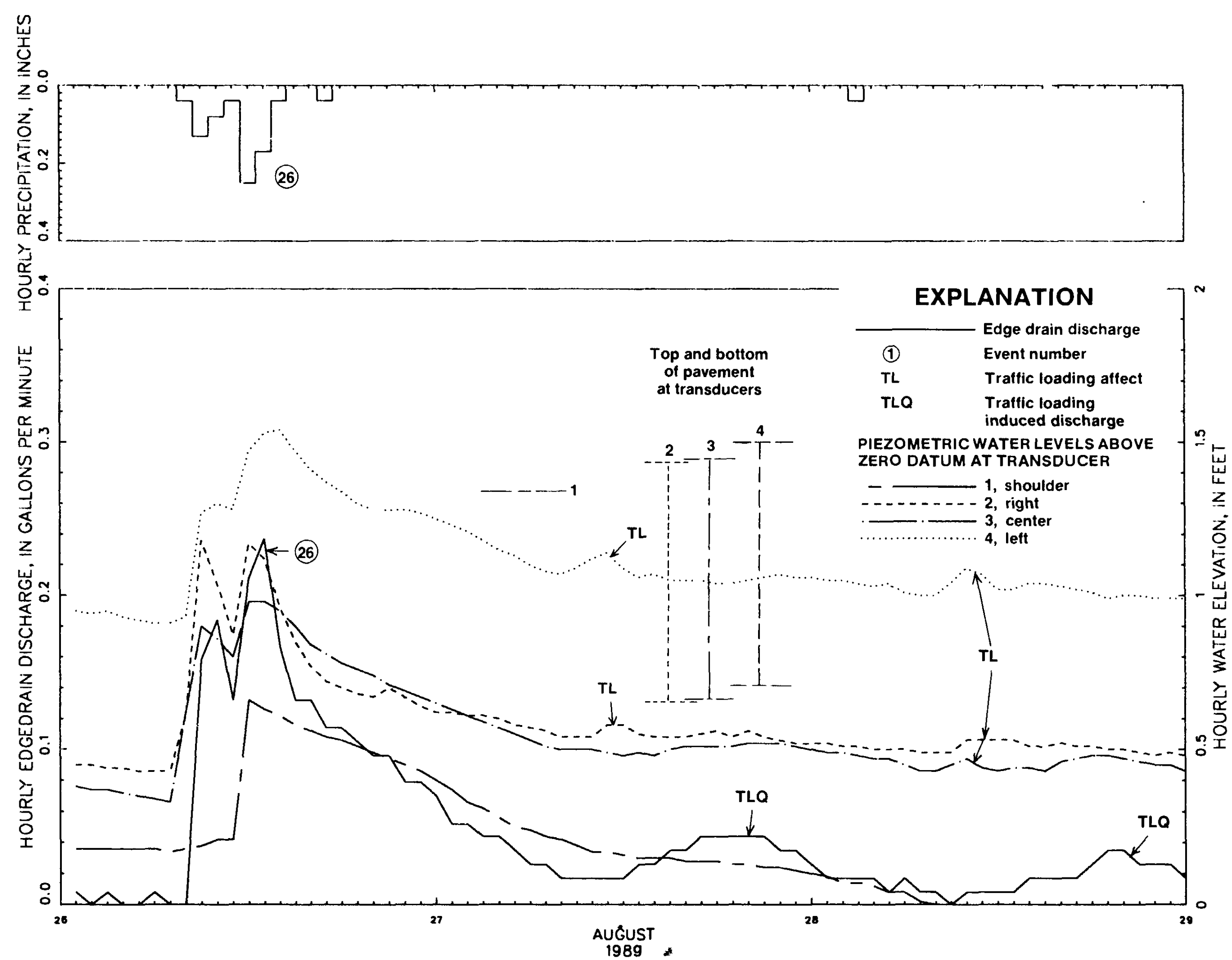

Figure 23C.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Minnesota.

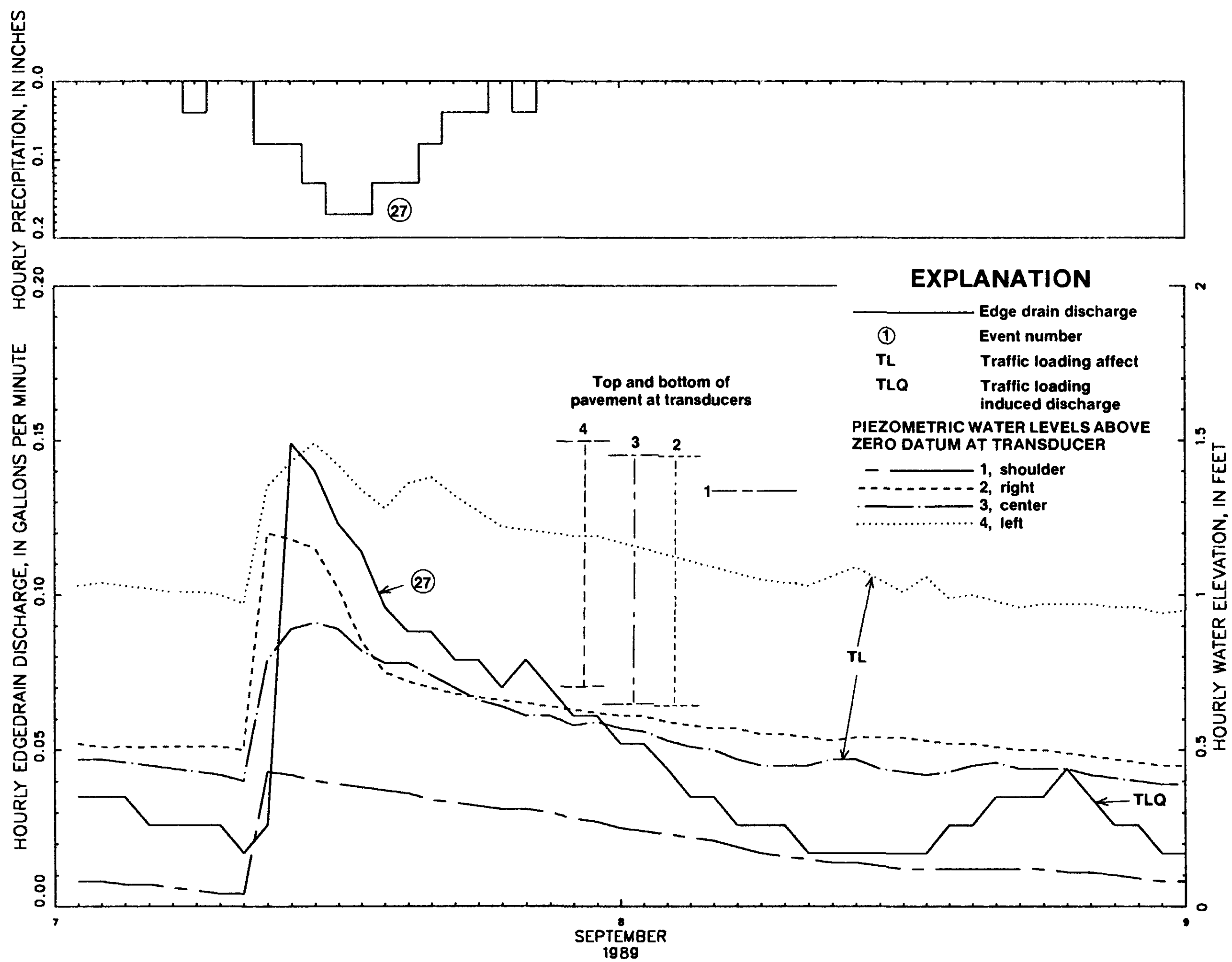

Figure 23D.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Minnesota.

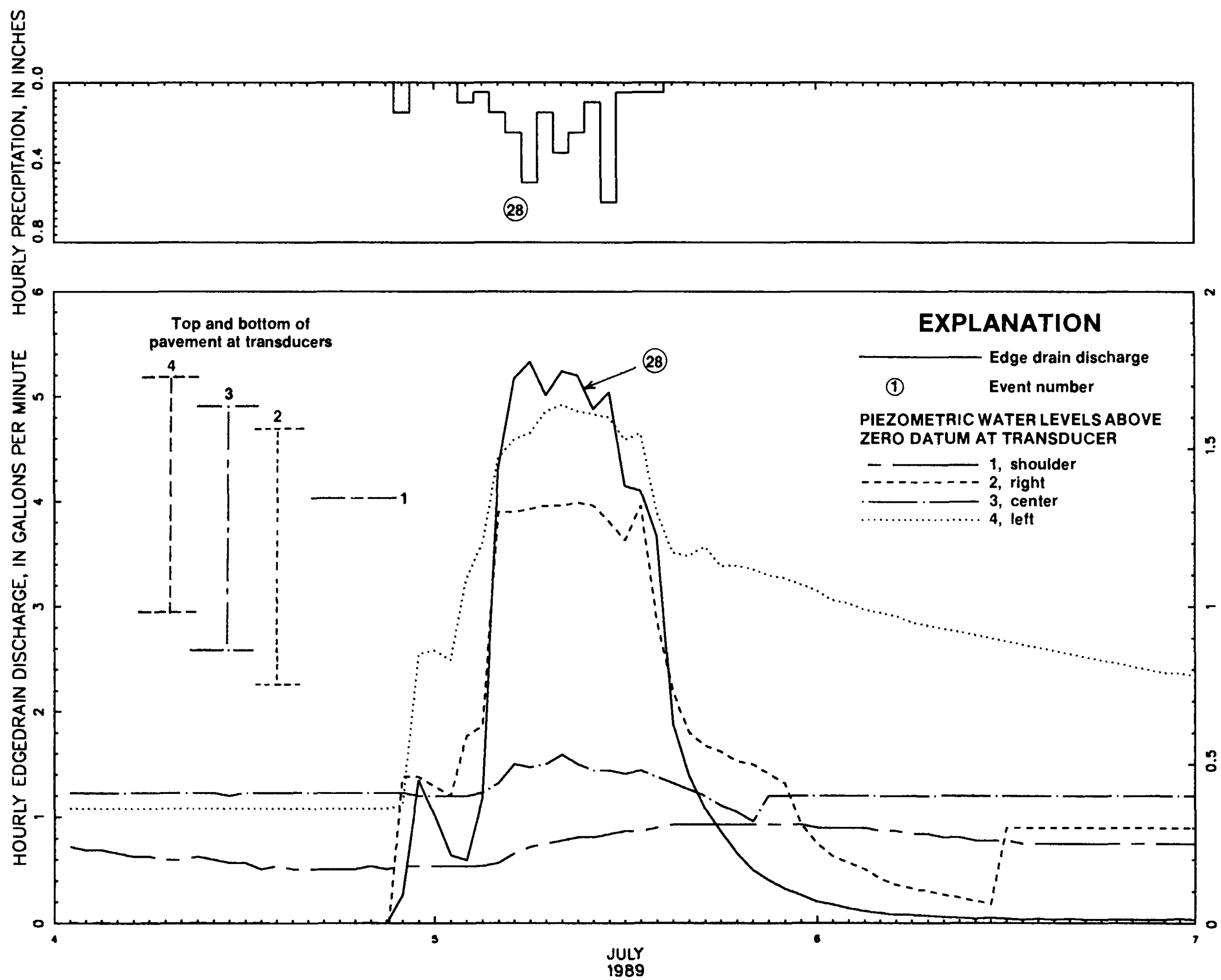

Figure 24A.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in New York.

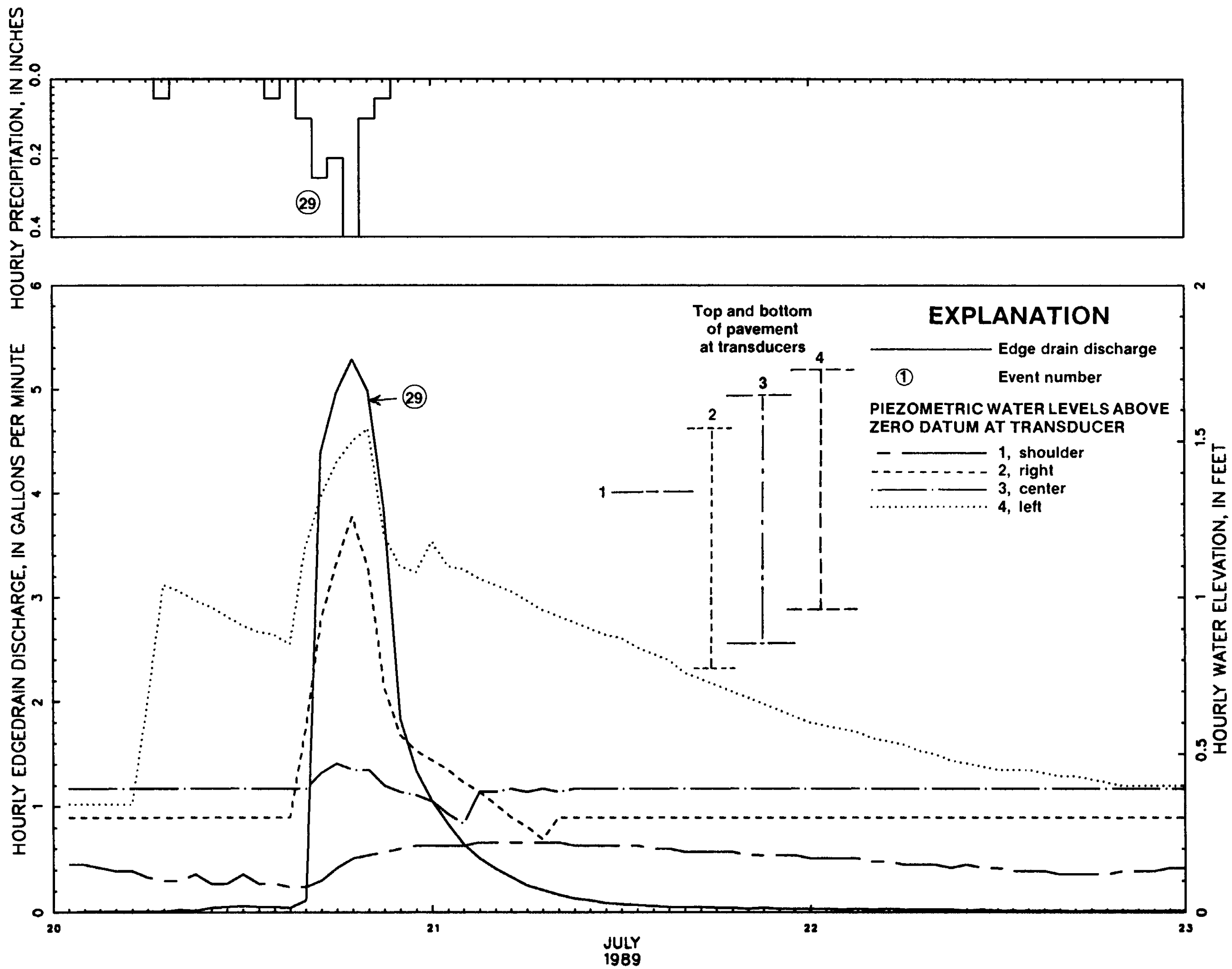

Figure 24B.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in New York.

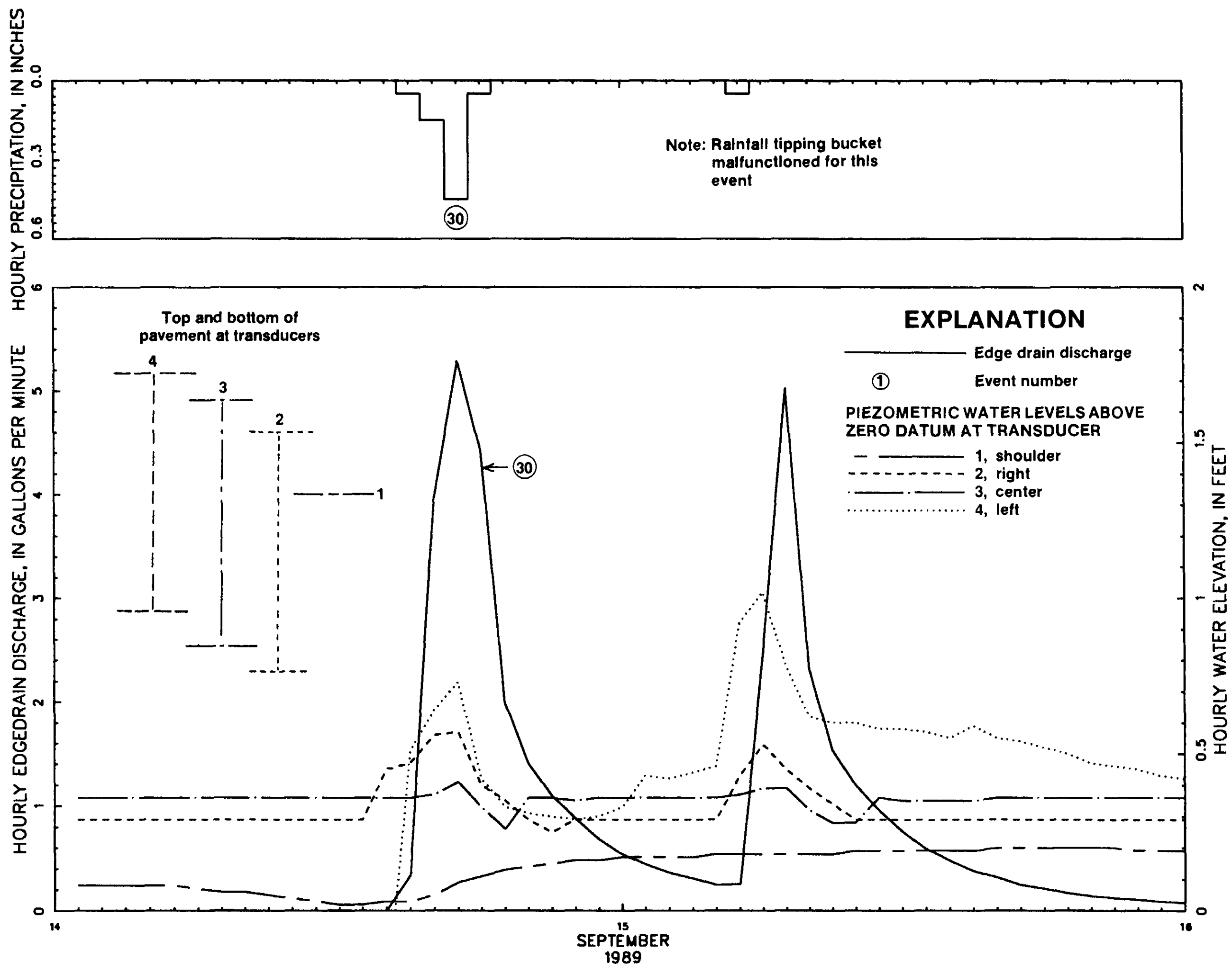

Figure 24C.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in New York.

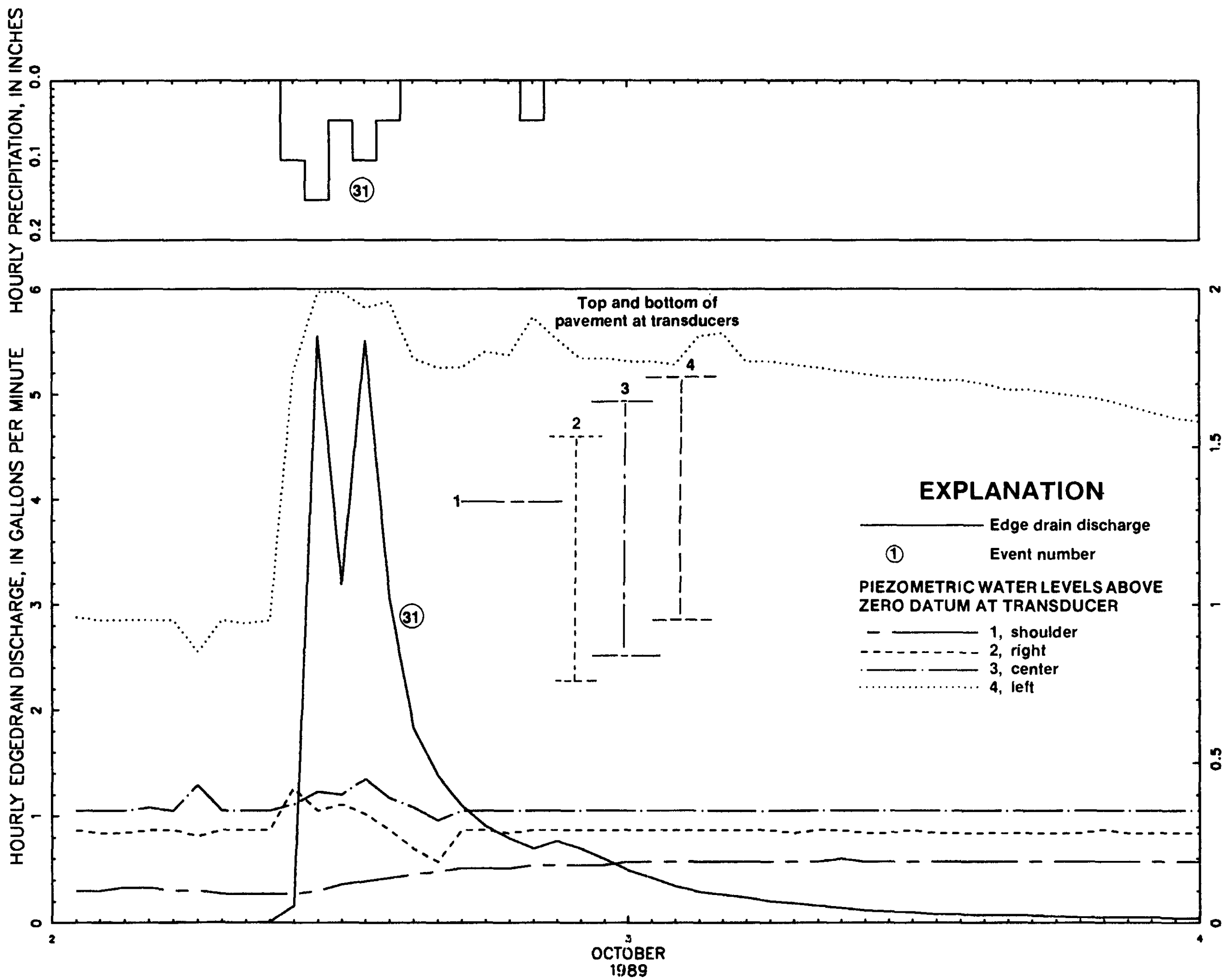

Figure 24D.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in New York.

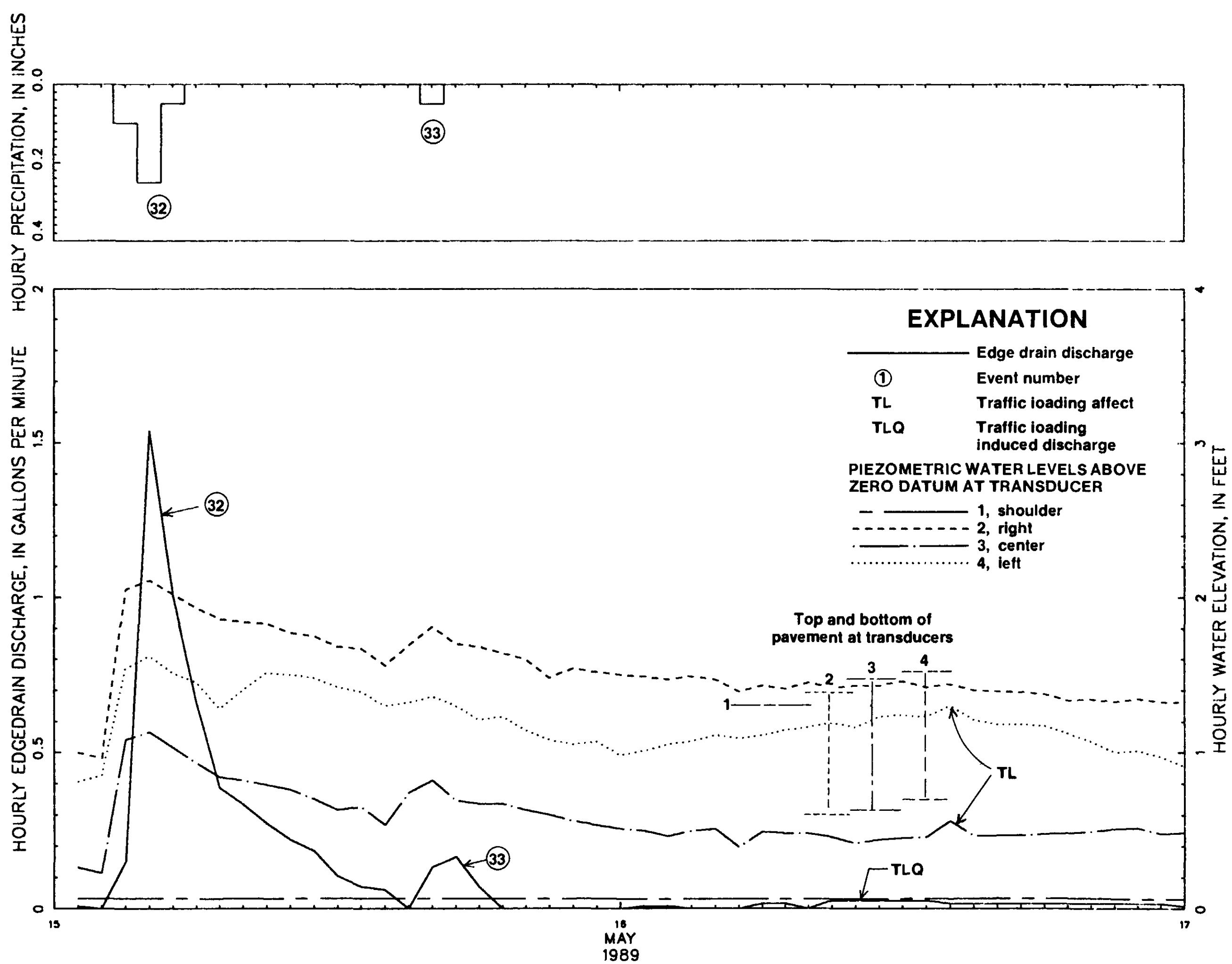

Figure 25A.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in North Carolina.

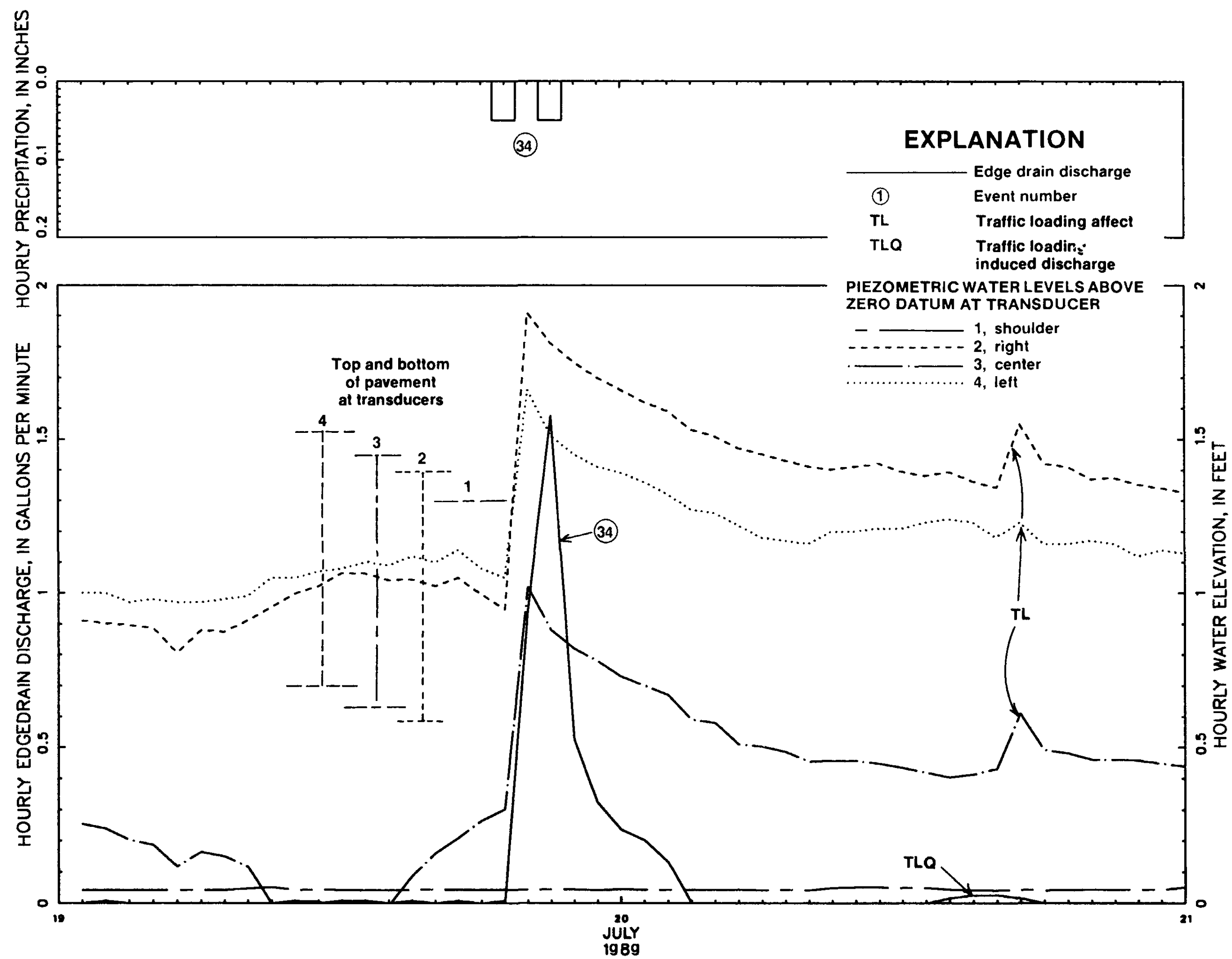

Figure 25B.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in North Carolina.

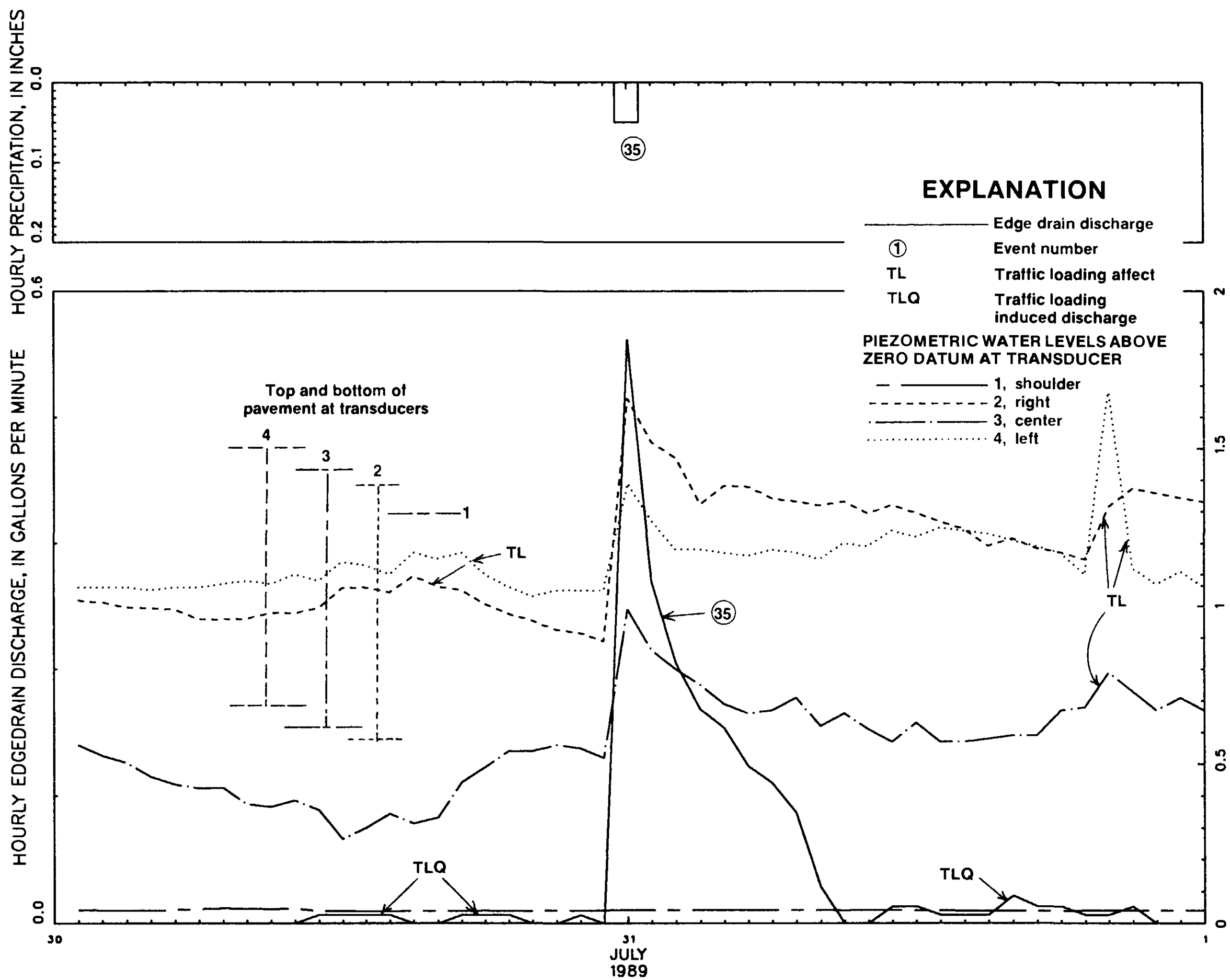

Figure 25C.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in North Carolina.

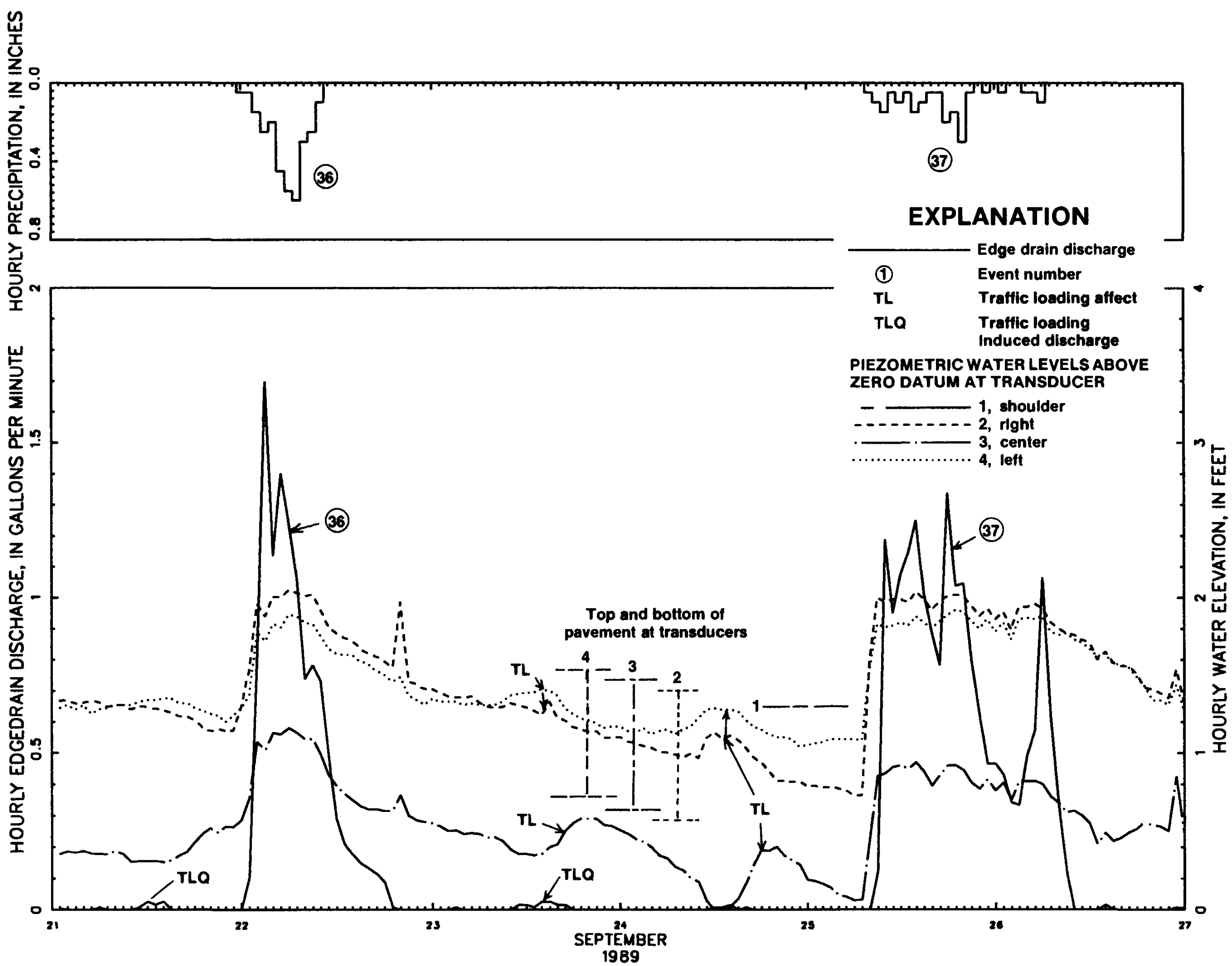

Figure 25D.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in North Carolina.

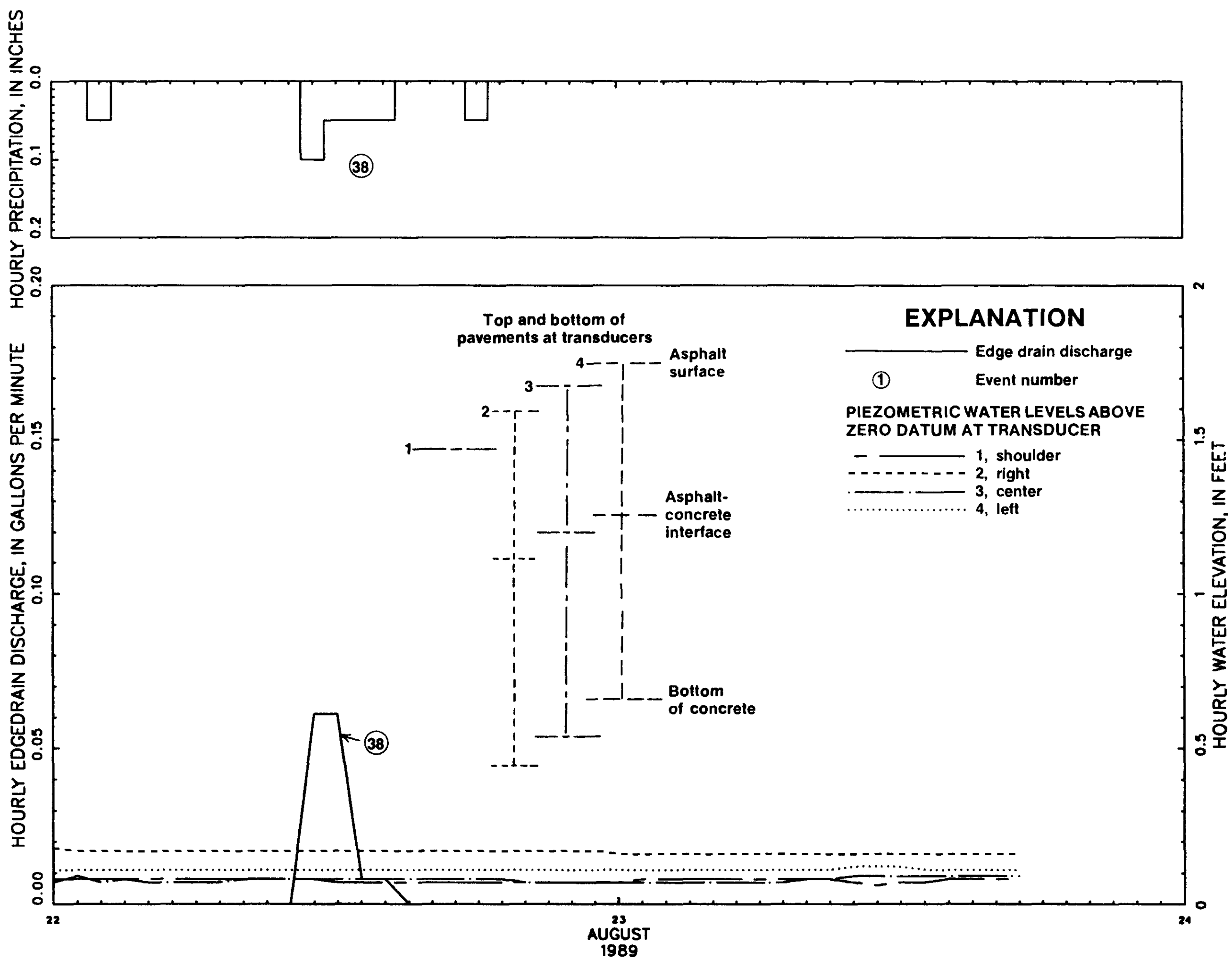

Figure 26A.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Oregon.

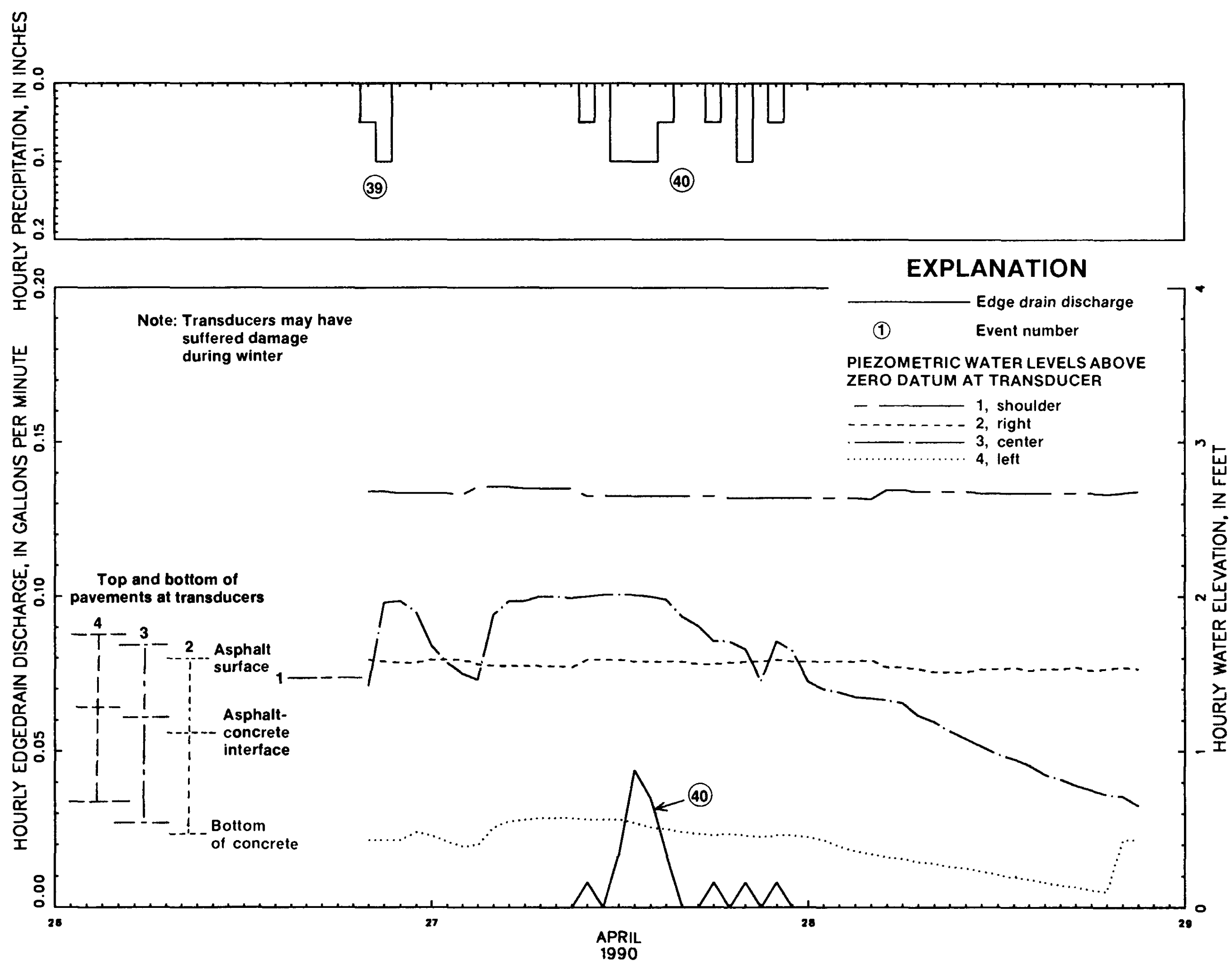

Figure 26B.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Oregon.

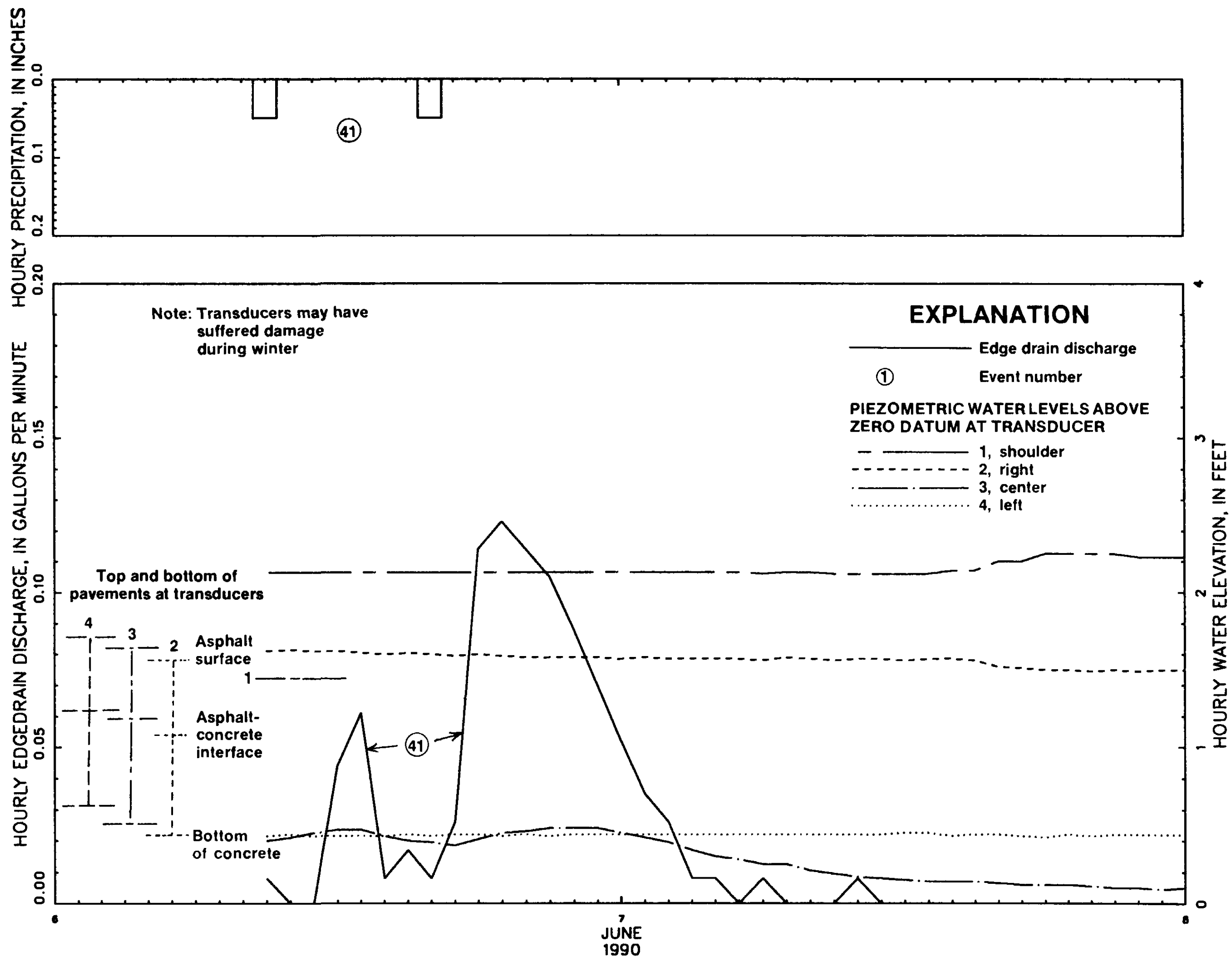

Figure 26C.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Oregon.

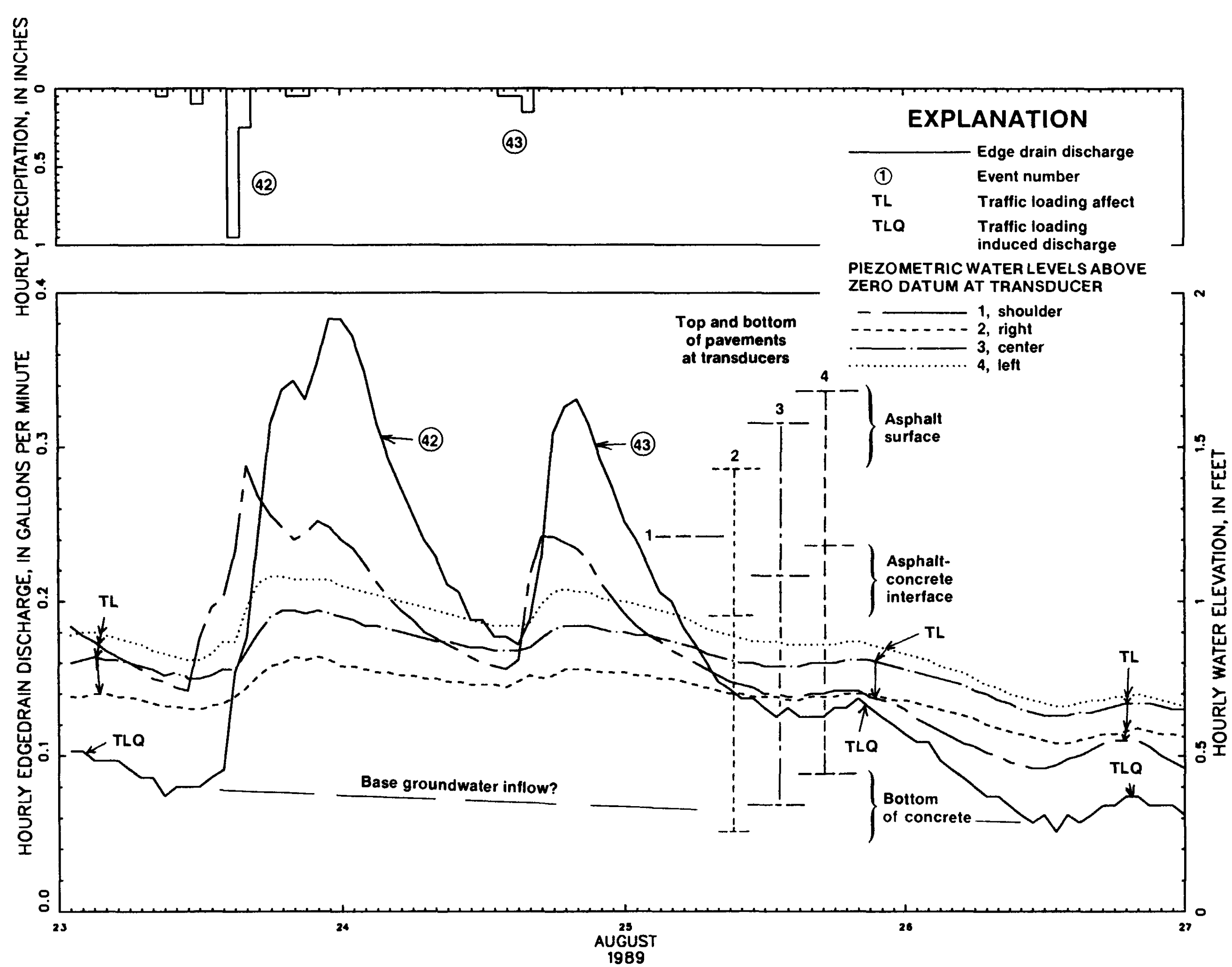

Figure 27A.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in West Virginia.

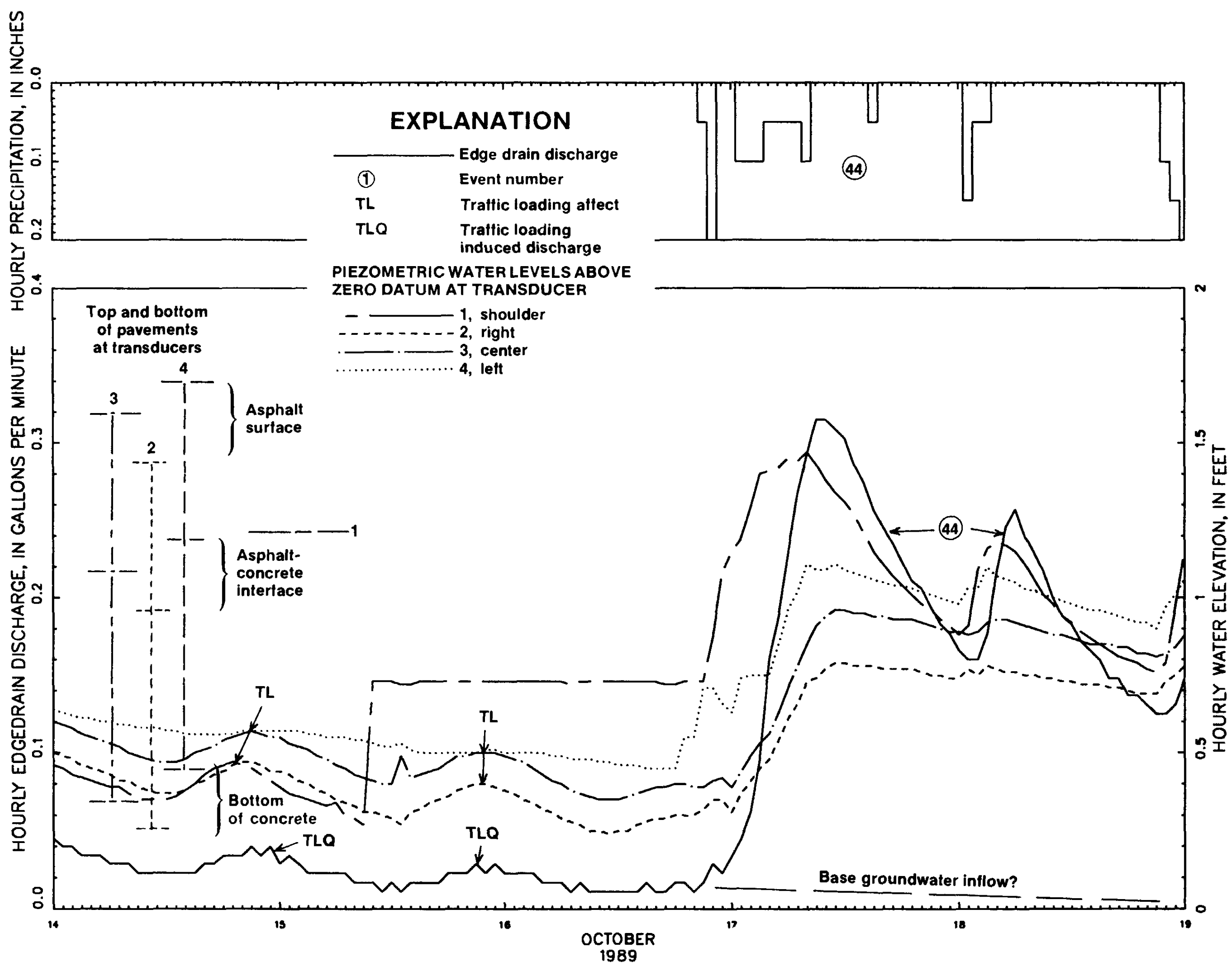

Figure 27B.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in West Virginia.

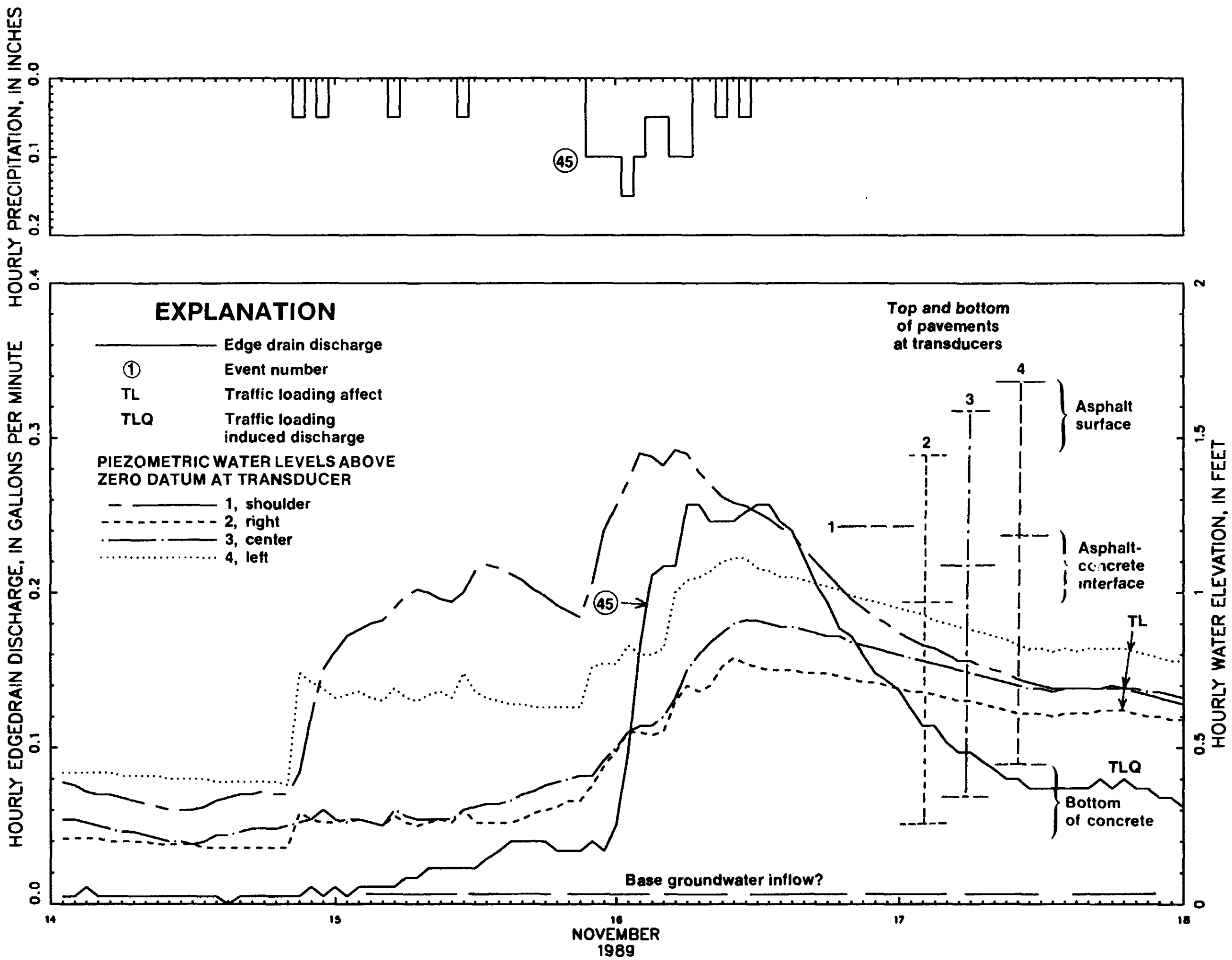

Figure 27C.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in West Virginia.

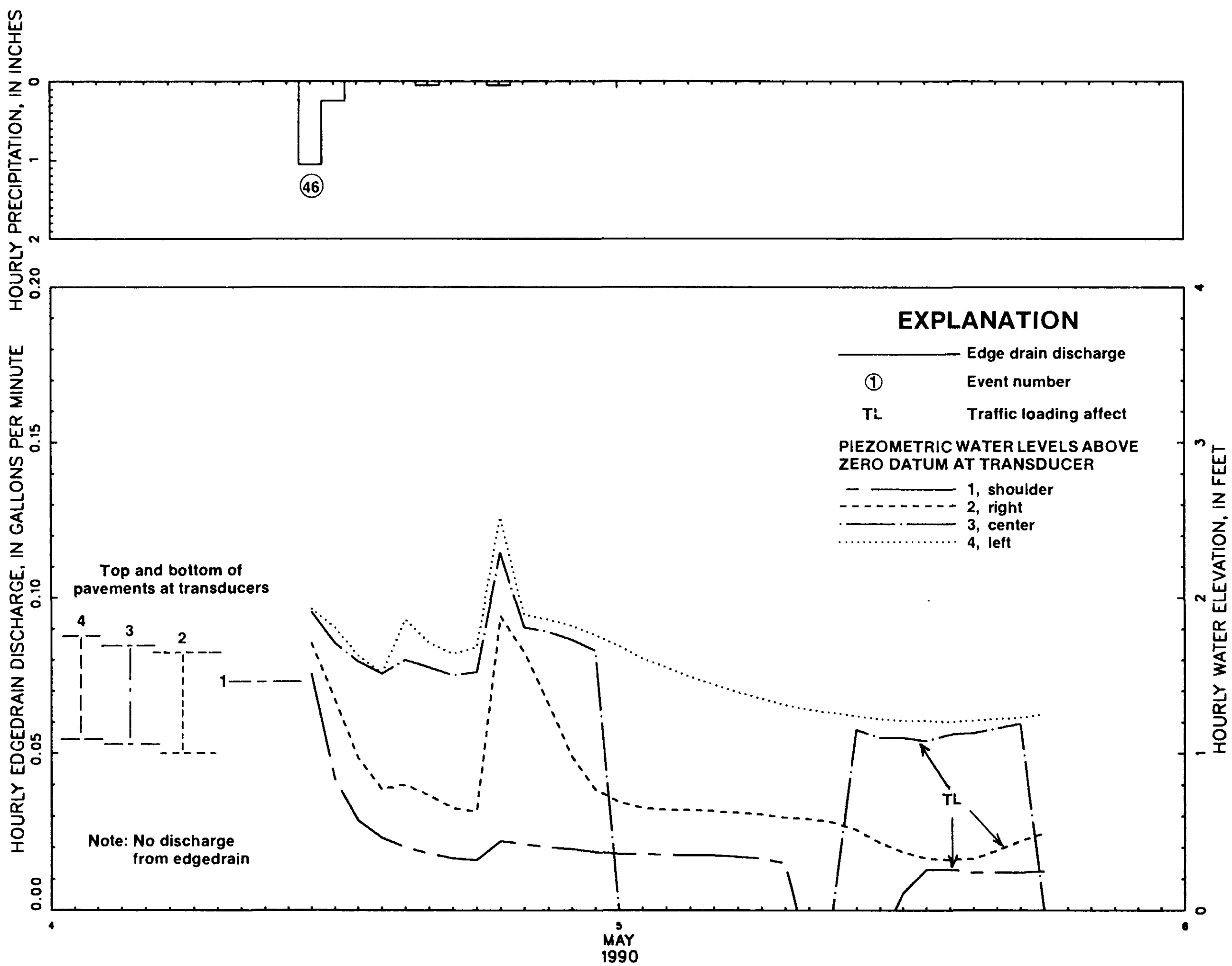

Figure 28A.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Wyoming.

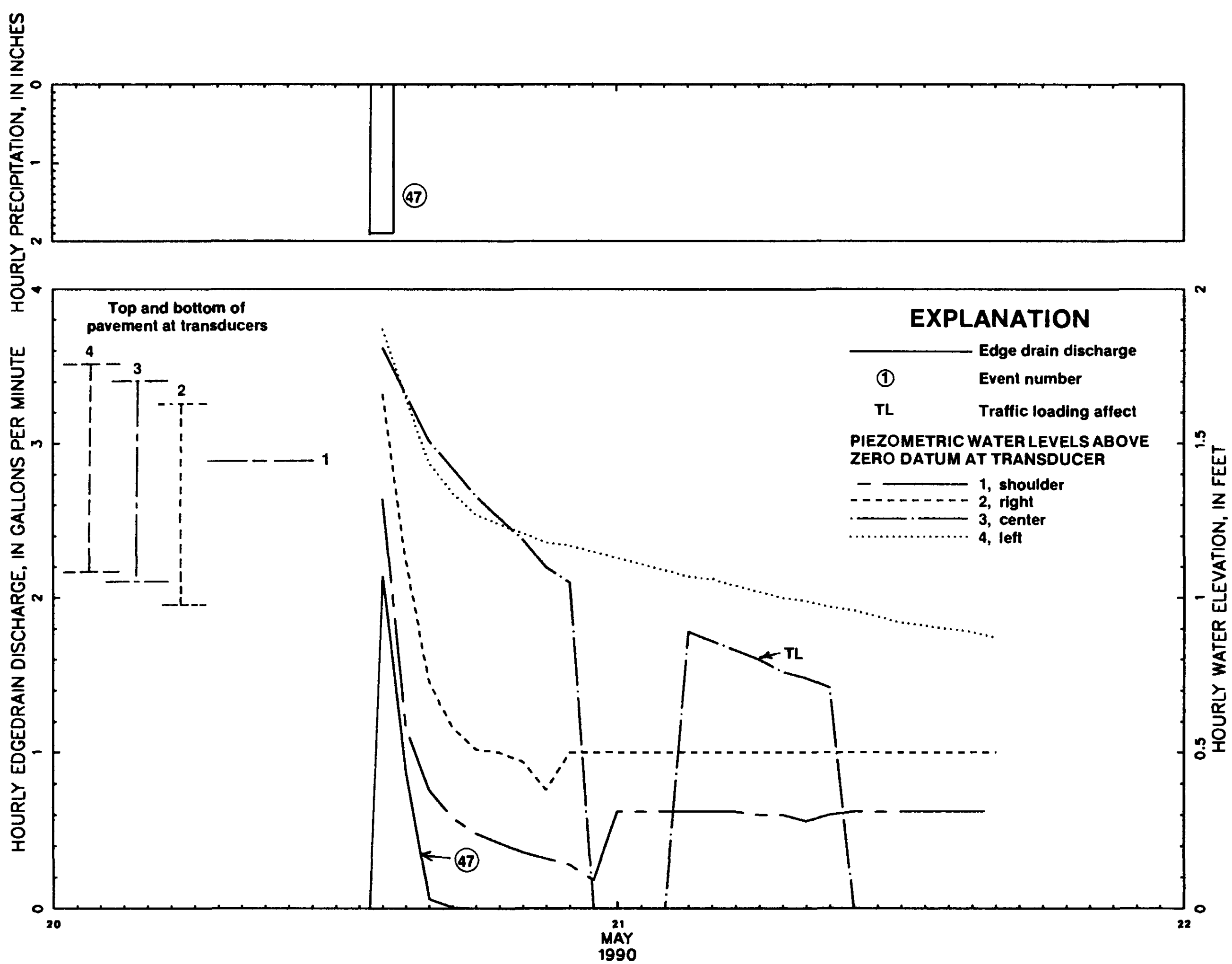

Figure 28B.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Wyoming.

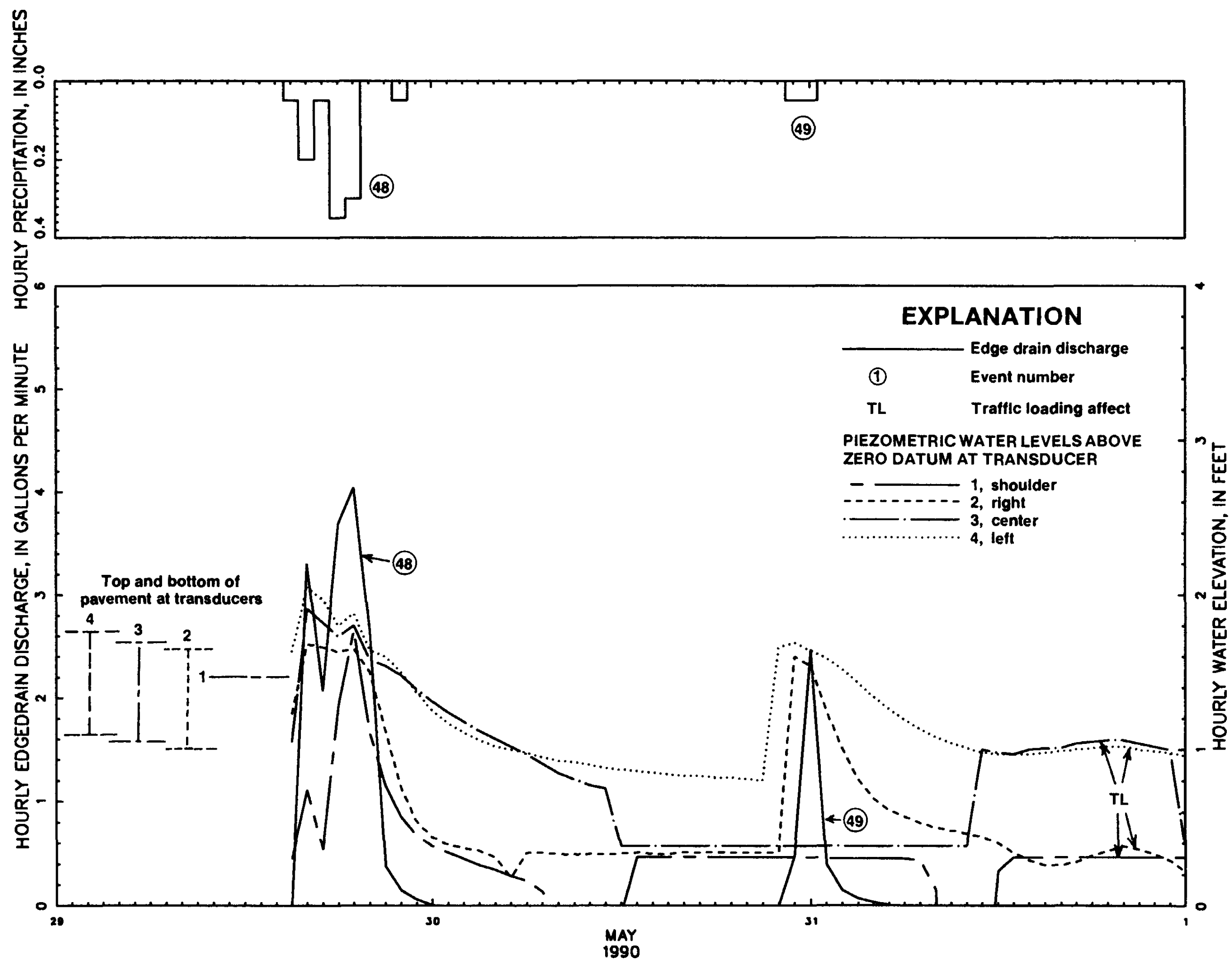

Figure 28C.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Wyoming.

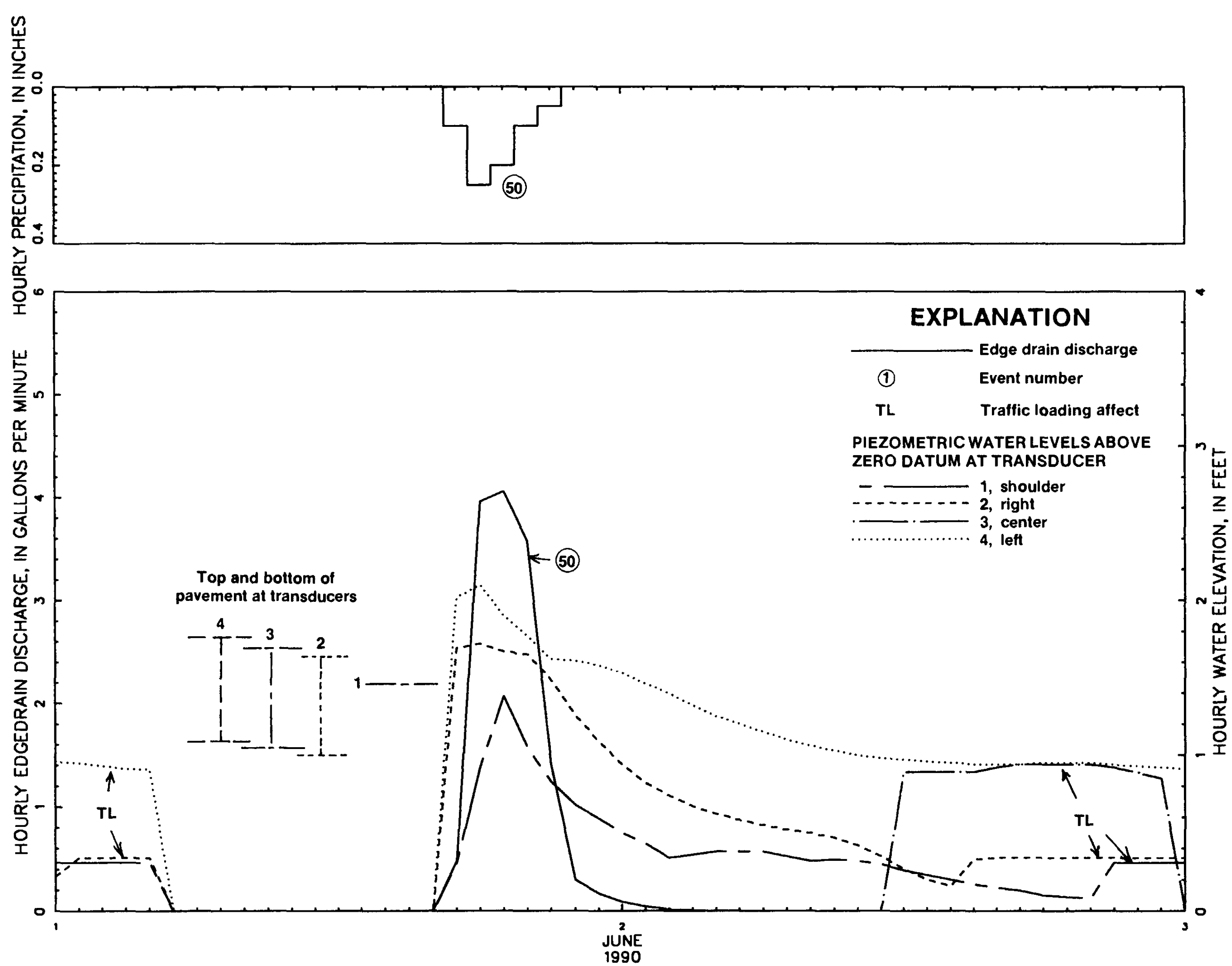

Figure 28D.--Plots of hourly precipitation, edgedrain discharge, and water levels at test site in Wyoming.

Of particular interest is how little rainfall is required to produce edgedrain discharge. The rainfall tipping bucket measures 0.05 inches with each tip. There were 6 events (numbers 2, 12, 15, 21, 33, and 35) with a total rainfall volume of 0.05 inches and all 6 produced discharge from their edgedrains. Most noticeable was event number 35 in North Carolina where 0.05 inches produced discharge of over 0.5 gpm. This at first seems impossible, but not if the total volume of water produced by 0.05 inches of rainfall on the area of pavement serving this edgedrain is considered. At the North Carolina site, the distance between edgedrain outlets is 225 feet and the pavement is 21-feet wide. If it is assumed that this is the drainage area supplying the edgedrain, a total volume of water that could reach it is 148 gallons per 0.05 inches of rainfall. This assumes no evaporation or drainage off the pavement surface except what might enter the edgedrain via the longitudinal edge joint.

It is not possible to accurately define the area of pavement draining to each edgedrain; some surface flow may transit from one highway segment to an adjoining one. Nevertheless, if it is assumed, as above, that the pavement surface area between edgedrain outlets is the drainage area for a given outlet, the total volumes of discharge can be compared with the total rainfall volumes producing them.

In this case, the total discharge volumes are the areas of the discharge hydrograph in figures 19 through 28. For some events, the hydrographs of succeeding events overlapped and had to be estimated; in particular events (9)-(10), (24)-(25), (30), and (42)-(43). Also, at the West Virginia site there appears to be a low sustained flow that cannot be attributed to a particular rainfall. This flow was separated from the rainfall produced runoff hydrographs in estimating the individual discharge volumes.

Total edgedrain discharge volume was compared to the total rainfall volume for the 50 selected events (table 3, column 8). At over half the sites, ten percent or more of the rain penetrates the highway surface and joints or drains into the edgedrain at the pavement-shoulder joint and eventually is discharged via the edgedrains. For eight events, more than 50 percent of the rainfall runoff infiltrates to the edgedrains. Rates of rainfall infiltration were consistently high for sites in California, New York, and North Carolina. At the California site, additional rainfall runoff may, in part, be coming from the next up-gradient pavement section; that is, its effective drainage area is more than the indicated 4,400 square feet (table 3, column 6).

By contrast, infiltration rates are very low at the Oregon and West Virginia sites where an asphalt overtopping was applied as part of the highway rehabilitation process.

Table 3--Percentage of total rainfall on pavement areas discharging through edgedrains for selected events

Event (1)	Location (2)	Date Year Mo. Day (3)	Total edgedrain volume (gallons) (4)	Rainfall volume (inches) (5)	Pavement area draining to edge-drain (square feet) (a) (6)	Total rainfall volume on pavement area (cubic feet) (7)	Edgedrain runoff volume as percent of total rainfall volume on pavement, (8)	Remarks (b) (9)
1	AL	1989 Feb 21	603	2.20	14400	2640	3	D
2		1989 Feb 21-22	49	0.05		60	11	W
3		1989 Mar 5- 6	468	0.75		900	7	D
4		1989 Mar 29-30	1313	2.25		2700	7	D
5		1989 May 20-21	560	1.20		1440	5	D
6	AR	1989 May 22-23	389	0.90	6600	495	11	D
7		1989 July 15-16	274	1.10		605	6	D
8		1989 Sept 2- 3	187	1.40		770	3	D
9	CA	1989 Sept 28-29	1588	0.75	4400	275	77	D
10		1989 Sept 29	493	0.20		73	90	W
11		1989 Nov 25-26	1316	0.90		330	53	D
12		1990 Jan 16	30	0.05		18	22	D
13		1990 Jan 16-17	845	0.35		128	88	W
14		1990 Feb 3- 4	902	0.55		202	60	D
15		1990 Feb 6	145	0.05		18	107	W
16	IL	1989 Apr 28	855	0.90	11500	862	13	D
17		1989 May 8- 9	605	0.45		431	19	D
18		1989 May 25	347	0.20		192	24	D
19		1989 May 25-26	1108	1.25		1198	12	W
20		1989 July 18-19	468	1.45		1390	5	D
21		1989 July 19	45	0.05		48	12	W
22		1989 July 19-20	904	0.55		527	23	W
23	MN	1989 July 29-30	572	1.57	7700	1007	8	D
24		1989 Aug 19-20	900	0.89		571	21	D
25		1989 Aug 19-20	450	1.32		847	7	W
26		1989 Aug 26-27	148	0.79		507	4	D
27		1989 Sept 7- 8	107	1.13		725	2	W
28	NY	1989 July 4- 6	3924	2.80	12100	2823	19	D
29		1989 July 20-21	1950	1.20		1210	22	W
30		1989 Sept 14-15	1317	0.70		706	25	D
31		1989 Oct 2- 3	1801	0.50		504	48	D
32	NC	1989 May 15	299	0.40	4950	165	24	D
33		1989 May 15	20	0.05		21	13	W
34		1989 July 19-20	242	0.10		41	79	D
35		1989 July 30-31	116	0.05		21	74	D
36		1989 Sept 22	682	2.95		1217	7	D
37		1989 Sept 25-26	1110	1.80		763	19	W
38	OR	1989 Aug 22	8	0.30	12100	302	0	D, A
39		1990 Apr 26	0	0.15		151	0	D, A
40		1990 Apr 27	32	0.60		605	1	W, A
41		1990 June 6- 7	56	0.10		101	7	D, A
42	WV	1989 Aug 23-25	414	1.45	11000	1329	4	D, A
43		1989 Aug 24-26	151	0.25		229	9	W, A
44		1989 Oct 16-20	574	1.60		1467	5	D, A
45		1989 Nov 15-19	421	1.15		1054	5	W, A
46	WY	1990 May 4	0	1.40	13200	1540	0	D
47		1990 May 20	185	1.90		2090	1	D
48		1990 May 29	981	1.00		1100	12	W
49		1990 May 30-31	216	0.10		110	26	D
50		1990 Jun 1- 2	848	0.70		770	15	D

(a) Pavement area between edge drain outlets

$$(8) = \frac{\text{Column 4}/\left[7.48 \text{ gal./ft}^3\right]}{\text{Column 7}} \times 100$$

(b) A - Asphalt topped
W - Wet antecedent condition
D - Dry antecedent condition

Piezometric Water Levels Beneath the Pavements

Figures 19 through 28 also show plots of the piezometric water levels at the three locations under the concrete pavement and at one location under the shoulder; each is identified by different line plots. Also shown on these figures, are vertical lines indicating the upper and lower surface of the concrete pavement at a particular piezometer. The same type of lines used in plotting piezometric water levels is also used with each of the vertical lines for identification.

Saturation of Pavement Sections

For at least 21 events, piezometric water levels are momentarily slightly above the pavement surfaces. This is seen to exist only briefly for small rainfalls but for nearly a day at certain sites in response to heavy sustained rainfalls. This occurred at the Minnesota and North Carolina sites in particular, as seen in column 7 of table 2. These "artesian" heads can exist because water has entered the subgrade of the pavement through a joint or crack some distance up-gradient (fig. 4) resulting in water levels potentially higher than the pavement at the locations of the transducers. This requires that little or no vertical leakage exist between the elevated intake joint or crack and the transducer. This condition is significant in that fines are likely to be washed out ultimately from under the pavement down-gradient, accelerating the formation of voids and channels.

For some rainfall events, it is evident that water has filled open joints near a transducer to a height to cause overflow before receding with the termination of rain and lateral drainage from beneath the pavement. For example, for event 4, figure 19, the piezometric level is even with the top of the pavement at the right transducer. Piezometric levels are below the pavement surface but above the bottom in the left and center transducer holes. The "topping off" of water in the joint near certain transducers is evident for event 10, center transducer; and events (18), (19), (20), (22), and (29) in most cases for the right transducers.

Examination of these piezometric hydrographs also reveals that water levels were potentially within the concrete pavement section for several hours <u>for virtually every rainfall event, except the most minor ones</u>. Examination of table 2, column 8, indicates that <u>the pavements are resting in a "sea" of water, for some events in excess of a day, even with the existence of the edgedrains</u>. It must be concluded that water still must get to the edgedrain to prevent "flooding" of the highway section for prolonged periods of time.

Response to Rainfall

The plots of piezometric water levels in the pavement subgrade show, as do the edgedrain discharge hydrographs, almost immediate response to rainfall. In fact, as might be expected, in most cases the levels respond more rapidly than the discharge hydrographs. While it is not practical to examine here in detail the piezometric hydrographs of each event, it is evident that they mimic the edgedrain discharge hydrographs. Thus, it must be assumed that at least part of the water draining to the edgedrains has its source beneath the pavement and hence has infiltrated through interior pavement joints and cracks.

Water Movement Through Shoulders

A transducer (number 1) was placed beneath the shoulders at most sites to ascertain if flow from under the main highway was transiting through the shoulder as well as via the edgedrain. At most sites, the piezometric hydrographs in the shoulders mimic the other transducer and edgedrain discharge hydrographs. At the Arkansas, California, and West Virginia sites, water levels in the shoulders are momentarily higher than those under the roadway and decrease slowly. This suggests that in some instances, shoulders having very low permeabilities may act as dams to lateral subgrade water movement. The edgedrains are in effect short-circuiting the shoulders and conveying water through them.

Traffic Loading Effects

Examination of the edgedrain discharge and piezometric hydrographs of figures 19 through 28 reveal many instances of heavy midday traffic having a loading effect. While no traffic counters were in place to verify this conclusion, the timing of the responses on the hydrographs would seem to suggest traffic loading as being the cause. This effect was most noticeable at the Arkansas, Illinois, Minnesota, North Carolina, and West Virginia sites. In numerous cases, water was actually "squeezed" out of the base or subgrade material sufficient to not only cause piezometric water levels to rise but to produce periodic midday or afternoon discharge from the edgedrains. Noteworthy evidence of this is shown in figure 25D for the September 21 through 27 period of record for the North Carolina site; note the midday timing of the edgedrain discharges.

Hydraulic Properties of Subgrade Materials

The core samples collected beneath the pavements at the ten sites were analyzed in the laboratory[1] for their physical and hydraulic properties. This included bulk density (table 4) and soil-size distribution (tables 5 and 6).

The bulk density is the dry weight of the total soil-sample mixture of solids and voids divided by its total volume. The absolute soil density is dependent on the relative volumes of the solids, their specific gravities, and void space. A well-compacted aggregate subbase will have minimum void spaces and a high-bulk density. Bulk-density values under the roadways ranged from 101 to 127 pounds per cubic feet (table 4). Slightly lower values were found to exist under the shoulders. Table 5 shows the percent soil passing a number 10 sieve. Table 6 shows the grain-size distribution of that which passed the number 10 sieve. Of particular concern is the percentage of clay and sand, as it may be expected to affect the soils ability to store and transmit water. In addition, saturated hydraulic conductivity (table 4) and water-retention curves (fig. 29) were determined for each site. It was necessary to develop soil water pressure retention curves in order to relate soil moisture using the gypsum blocks. The water-retention data also provide a measure of the ability of the soils to store as well as drain water.

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(1)All laboratory analysis were performed in the soils laboratory at Auburn University, Auburn, Alabama, by Dr. Jacob H. Dane and followed standard procedures as outlined by Klute (1986).

Table 4--Bulk density, saturated hydraulic conductivity, and permeability of soil samples taken beneath the pavements at the ten test sites

Sample	Bulk density		Hydraulic conductivity		Permeability
	g/cm^3	lbs/ft^3	m/day	ft/day	$\frac{gal/day}{ft^2}$
	(1)	(2)	(3)	(4)	(5)
Alabama (pavement)	1.77	110	2.122	6.96	5.20
(shoulder)	1.55	97	1.537	5.04	3.76
Arkansas (pavement)	1.70	106	0.102	0.34	0.25
(shoulder)	1.67	104	0.324	1.06	0.79
California	2.03	126	1.98	6.50	4.85
Illinois	1.81	113	0.00	0.00	0.00
Minnesota	1.76	110	1.77	5.81	4.34
New York	2.00	125	0.001	0.003	0.005
North Carolina (pavement)	1.62	101	0.001	0.003	0.005
(shoulder)	1.41	88	0.111	0.36	0.27
Oregon	2.03	127	0.026	0.085	0.063
West Virginia	1.81	113	0.00	0.00	0.00
Wyoming	1.74	108	0.00	0.00	0.00

Table 5--Percent of soil passing the no. 10 sieve for samples taken beneath the pavement at the ten test sites [p, pavement; s, shoulder]

Sample	Percent Passing
Alabama (p)	97.8
(s)	98.4
Arkansas (p)	41.1
(s)	55.8
California	51.3
Illinois	98.0
Minnesota	58.2
New York	44.6
North Carolina (p)	98.0
(s)	93.8
Oregon	71.5
West Virginia	77.4
Wyoming	82.0

Table 6--Grain-size distribution of soil passing the no. 10 sieve for samples taken beneath the pavements [p, pavement; s, shoulder; mm, millimeter]

	Particle-size distribution			Sand-size distribution in percent				
	Sand	Silt	Clay	2-1	1-.5 mm	.5-.25 mm	.25-.1 mm	.1-.05 mm
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Alabama (p)	81.20	5.12	13.68	9.61	36.58	33.13	14.66	6.03
(s)	83.45	1.59	14.95	0.12	0.36	51.94	46.25	1.33
Arkansas (p)	30.44	40.33	29.23	51.19	25.42	8.14	5.08	10.17
(s)	19.54	43.67	36.79	30.89	17.28	8.38	9.95	33.51
California	83.15	10.62	6.23	19.57	6.52	50.12	17.39	6.40
Illinois	4.00	41.63	54.36	12.82	15.38	25.64	28.21	17.95
Minnesota	81.12	13.08	5.80	23.95	31.27	29.78	11.79	3.23
New York	49.14	34.79	16.06	40.25	25.26	14.78	10.88	8.83
N. Carolina (p)	28.98	36.71	34.32	12.50	17.14	22.50	27.50	20.36
(s)	25.69	24.29	50.02	41.20	23.20	14.80	11.20	9.60
Oregon	58.08	29.19	12.73	22.16	18.32	22.34	23.73	13.44
West Virginia	11.66	52.72	35.62	13.27	15.93	15.93	20.35	34.51
Wyoming	63.47	18.14	18.39	1.91	9.87	20.86	38.38	28.98

Soil Moisture

The laboratory-determined soil-water retention curves for the soil samples taken beneath the pavement at the ten highway sites are shown in figure 29. The volumetric water content of the sample was determined and paired with known pressure values between 0 and -350 cm of water. The data shows the water content of the samples at zero pressure and complete saturation and the water content at incremental steps of increased negative pressure (-350 cm of water). For all ten soil samples, any significant water content changes occur primarily between 0 and -40 cm of water pressure. The samples from the Illinois, North Carolina, Oregon, and West Virginia sites retained at least 90 percent of the initial water content. Under the same conditions, the samples from the Alabama, California, and Minnesota sites retained less than 46 percent.

The water-retention curves show the drainage and storage characteristics of the subgrade materials at the ten highway sites. The subgrade materials at the West Virginia, Illinois, and North Carolina sites retain relatively high and nearly constant moisture levels, which indicates poor drainage. Conversely, the curves for California, Alabama, and Minnesota show that better drainage probably exists.

The output from the gypsum soil-moisture blocks provides a measure of negative soil pressure or suction. The soil-moisture blocks can, therefore, be used to estimate soil-water content, if soil-water retention curves are available for the soil in question. Figure 30 shows the soil moisture indicated by the gypsum block and expressed in feet of water as well as in percent water content for the three soil-moisture blocks placed beneath the pavement and shoulder of the Alabama site. At 0600 hours on February 21, 1989, event number 1 (fig. 19A) the soil moisture under the shoulder is about 13.6 percent and under the pavement about 13.7 percent. This assumes the soil-moisture retention curve under the shoulder is the same as under the pavement as shown in figure 29. These numbers, and the water-content scale on the right, show the lack of sensitivity of the gypsum blocks. At this time, water levels in the adjacent transducer holes under the pavement were at or above the bottom of the pavement slab (fig. 19A) and discharge from the edgedrain was approximately 1.3 gpm. More important, figure 30, shows little or no response after at least 36 hours following the 2.2 inches of rain which comprise event number 1.

Figure 31 shows the soil moisture in feet of water and percent water content for the gypsum blocks under the shoulder (number 5) and pavement (number 6) at 2400 hours for the period of record January through December 1989, at the Alabama site. Also shown are the time of events, 1 through 5. These data show the soil moisture variation in the subgrade under the shoulder and pavement to be about 13 to 13.5 percent in the winter months, increasing to

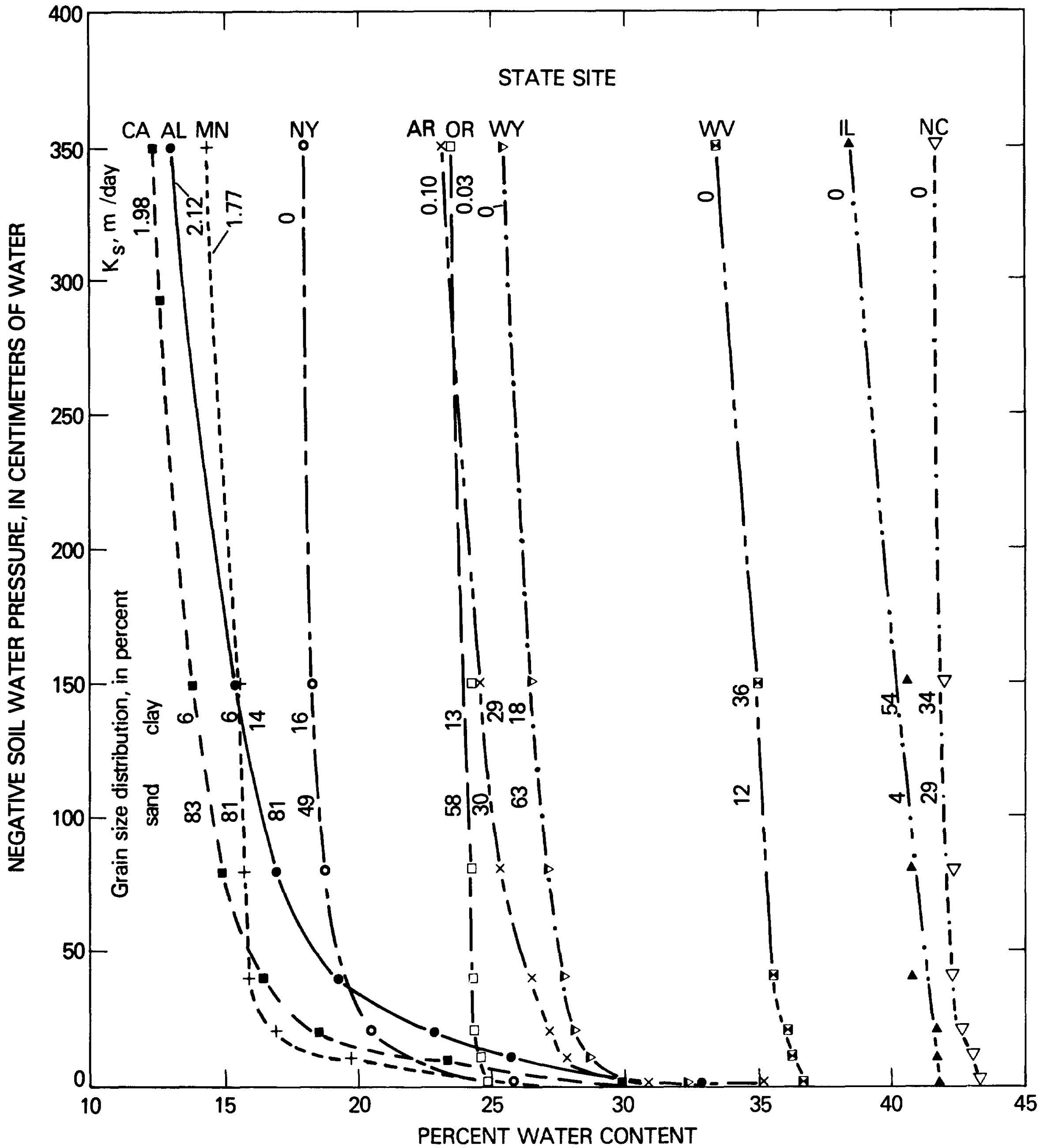


Figure 29.--Water-retention curve plots developed from core samples obtained below pavement at 10 highway sites.

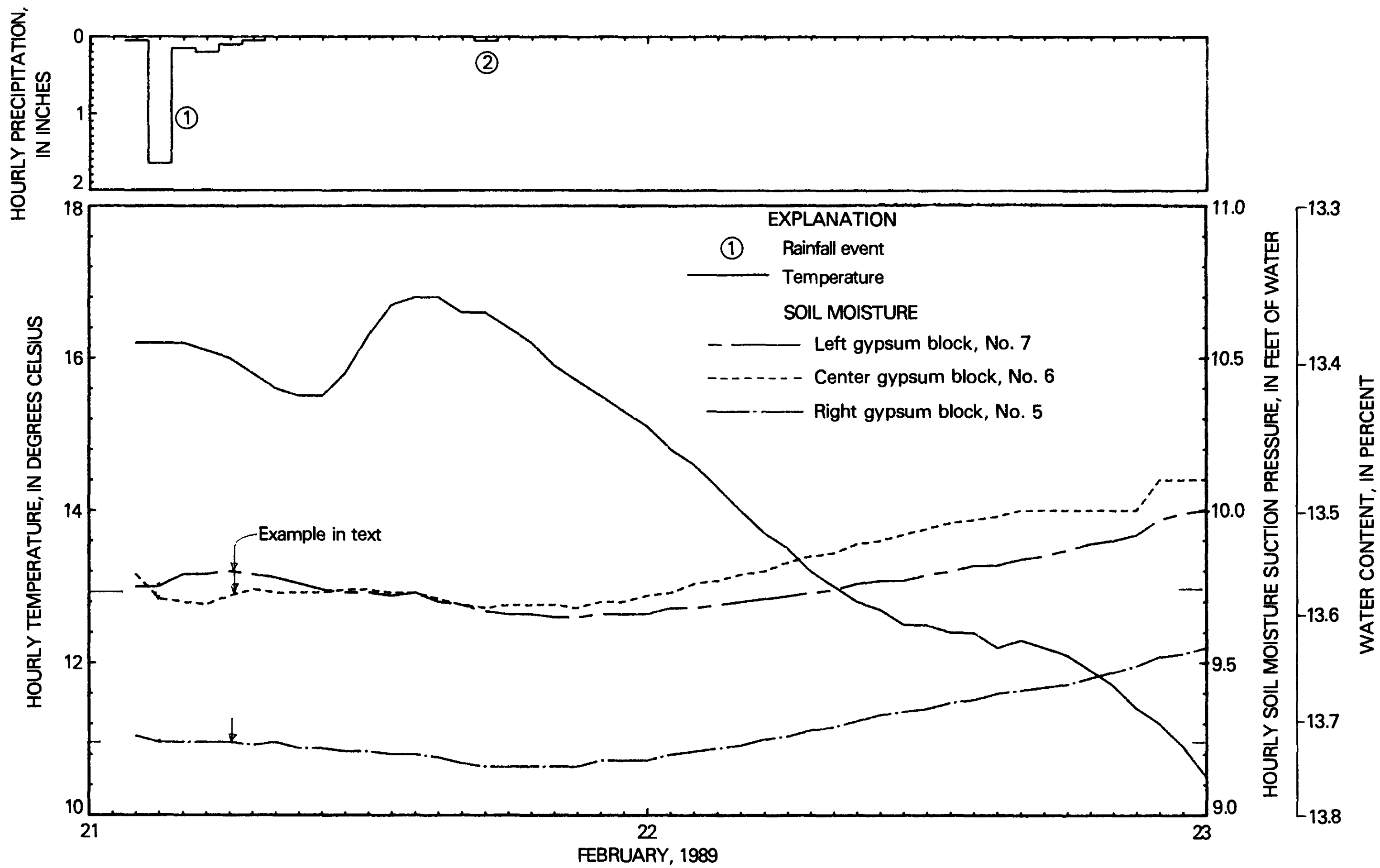


Figure 30.--Plots of temperature, precipitation, and soil moisture at Alabama test site.

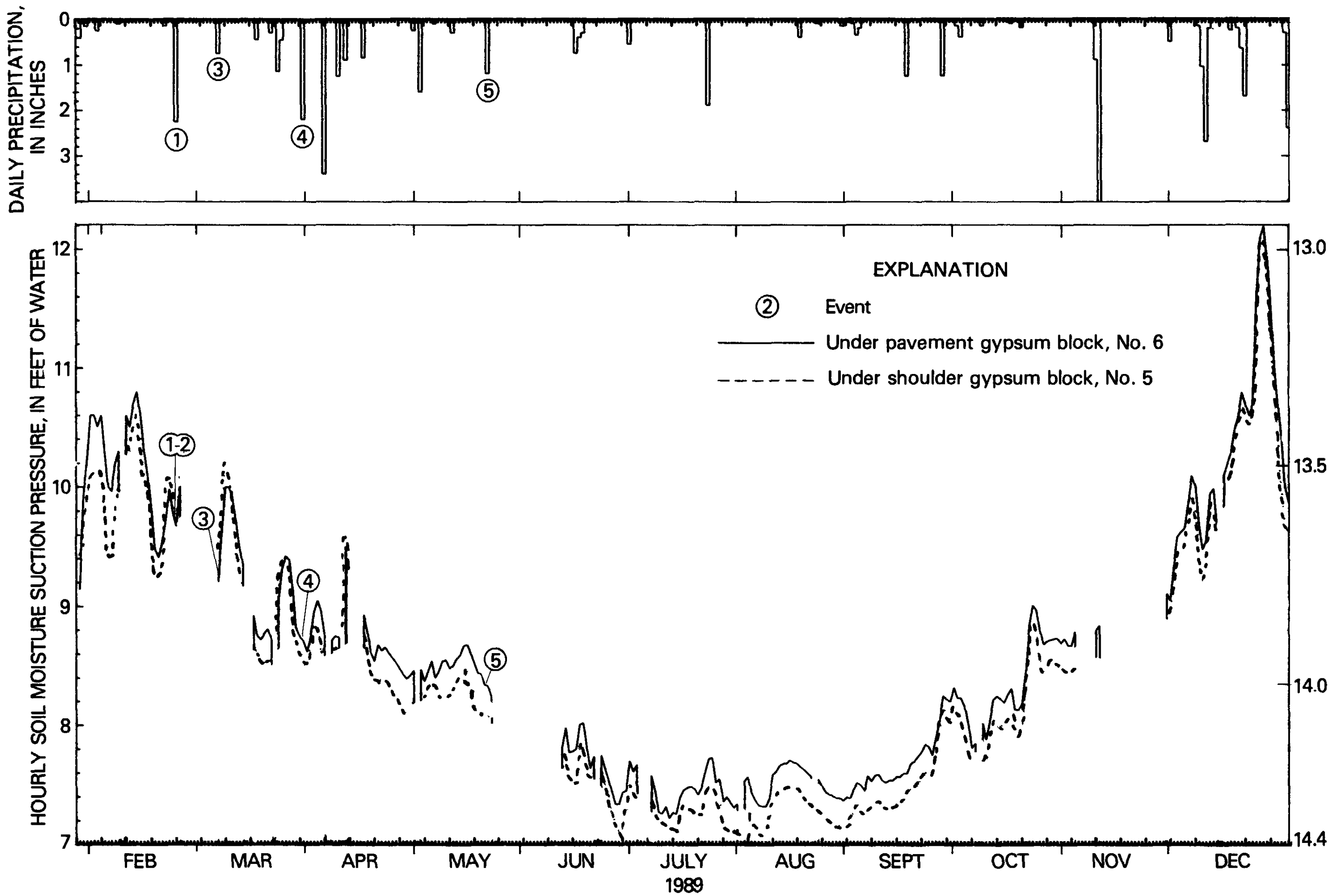


Figure 31.--Plots of soil moisture variation under pavement and shoulder at Alabama test site.

about 14.2 percent in the summer months. Except that the absolute magnitude of the moisture contents varied, most sites showed only minor seasonal change, as with the Alabama site. These very limited long-term seasonal changes in soil moisture suggests that the subgrade materials are very tight and do not experience significant water movement in either the unsaturated or saturated state unless via voids and channels beneath the pavements.

Obviously, some of the holes in which the soil-moisture blocks were placed became filled with water, as did the transducer holes, and short-term saturated conditions existed. Apparently, the gypsum blocks failed to indicate sudden changes in moisture because of their inherently slow response. Furthermore, the placement of the blocks back in the walls of the hole and just beneath the bottom of the pavement may have shielded them from direct water contact for any significant length of time. This limitation of gypsum blocks has also been reported by Wisler and Brater (1959). For these reasons, the soil-moisture data for all 50 events is not presented.

Hydraulic Conductivity

The hydraulic conductivity or permeability is a measure of the soil's ability to transmit water. Hydraulic conductivity and permeability are the same and are used interchangeably, but usually with different units as shown in table 4, columns 3, 4, and 5. Table 4 shows saturated hydraulic conductivity, K_s, and permeability for all the samples as measured in the laboratory. Laboratory values of K_s are usually in meters per day; column 4 shows it in feet per day to give a better comprehension of the transmitting capability (or lack thereof) of the different subbase materials.

These K_s values are also shown in figure 29 on their respective soil-water retention curves. Note that the retention curves showing the storage and retention of the least water, California, Alabama, and Minnesota have the highest K_s values. By contrast, West Virginia, Illinois, and North Carolina have K_s values of zero and retained 34 to 43 percent of their saturated water content and hence do not drain readily. Under the same conditions, the samples for Alabama, California, and Minnesota retained on the order of 15 percent for negative pressures above 100-cm of water. Only the samples from these sites demonstrate much drainage capacity.

According to Klute (1986), the water-retention function of a soil is primarily dependent upon its texture or particle size. The grain-size distribution analysis (table 5) was performed primarily to show the percents of sand and clay. A high percentage of sand and low percentage of clay would be expected to yield higher hydraulic conductivity values and vice versa. Comparison of these data reveals this to be the case and furthermore, it is reflected in the soil- moisture retention curves of figure 29. As seen in figure 29, those samples having a high-sand content and low-clay content correspond to those having the higher K_s values and greater drainage potential. By contrast, the samples for West Virginia, Illinois, and North Carolina, where zero K_s values were determined, have high percentages of clay which inhibits drainage and increases water retention.

Water-Tracer Tests

Results from the dye-tracer tests were inconclusive. No dye was detected in any of the edgedrain discharges, except at the North Carolina site. Here, dye was observed, minutes after placement, as there was edgedrain discharge at the time.

These results are perhaps not surprising as the dye was placed in a 1-inch hole into the subgrade away from any transverse joints. No obvious voids or channels were intercepted, especially when only a 1-inch diameter hole was drilled. Unless a void or channel was intercepted, as in the North Carolina case, the tightness of the subgrade soils, as clearly indicated by the laboratory tests on the core samples, precludes rapid movement. Voids and channels are more apt to develop under the pavement along transverse joints, rather than along longitudinal joints (fig. 4) which is near to where the dye was injected. The failure to detect tracer in the edgedrain discharges perhaps further confirms the tightness and lack of permeability of the subgrade materials.

CONCLUSIONS

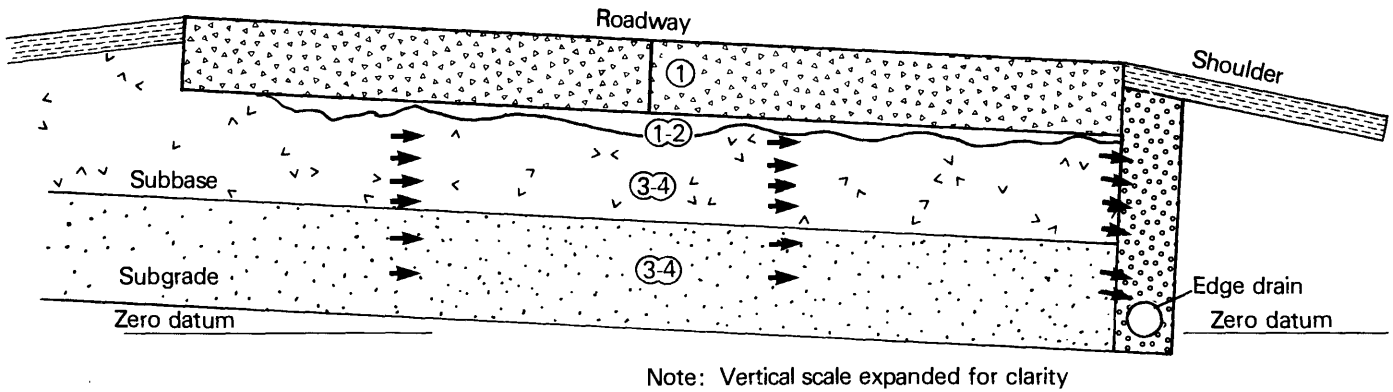
Concept of Flow Beneath Highway Pavement Where Edgedrains are Employed

The data collected in this study to measure the edgedrain discharge hydrographs and piezometric water level hydrographs that occurred in response to 50 rainfall events reveals to some extent the nature of flow from beneath the pavements to the edgedrains. The soil-moisture data, to a limited extent, and the laboratory tests on the subgrade soil samples, supplements and confirms conclusions as to the nature of flow.

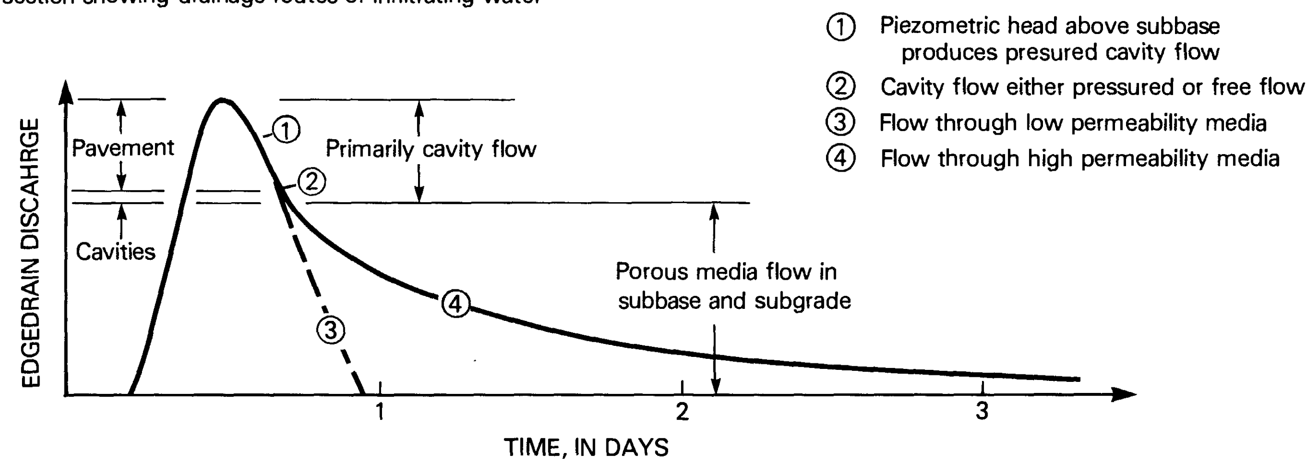
Figure 32 is a conceptual diagram of flow under the pavement. It is apparent from both the piezometric water level data and the soil-test data that the subgrade materials have poor drainage characteristics. Furthermore, all pavements will ultimately suffer joint damage and infiltration into the subgrade. Because of the initial tightness of the subgrade, water will flow into up-gradient joints and discharge down-gradient at the other joints eroding fines from the subbase. Ultimately, voids and channels form to drain off excess infiltrated water. At times, sections of pavement may be literally sitting in a shallow basin of water. This is why, in many instances, the smallest amount of rain produced immediate piezometric water level increases and discharge from the edgedrains.

In many instances, the outside shoulders of the highway form a restraining dam to transverse subgrade drainage. Retrofitting an edgedrain along the pavement edges with outlet pipes through the shoulders serves to short-circuit the shoulders. A major component of the edgedrain discharge is the surface flow off the pavement which enters the longitudinal edgedrain directly. The edgedrain in this case is analogous to a gutter on a house roof.

Pressured conduit flow exists when piezometric water levels are above the pavement surface and until they recede to below the bottom of the pavement (fig. 32, locations 1 and 2). For a brief period, depending on the extent of the voids and channels, and the duration of the rain, free surface-water flow exists toward the edgedrain. When piezometric water levels drop below the elevations of the channels, flow to the edgedrains must be through the porous media making up the subgrade. If the permeability of the subgrade material is very low, the discharge to the edgedrain will terminate quickly (fig. 32, location 3). If some permeability exists, there will be some continuous slow drainage to the edgedrain (fig. 32, location 4). Comparison of the drainage durations for the different events, table 2, column 6, with the hydraulic conductivity data in table 4, column 3, tends to confirm this last concept.



A. Highway section showing drainage routes of infiltrating water



B. Edgedrain discharge hydrograph

Figure 32.--Sketches showing concept of flow beneath highway pavement where cavities have developed.

Effectiveness of Edgedrains

Retrofitting longitudinal edgedrains to an existing highway provides a sink to collect water draining laterally off the pavement surface as well as water reaching the edgedrain through subgrade voids and channels; the edgedrain outlets then serve to short-circuit the water through the highway shoulders. It is nevertheless true that the lateral drainage must be able to reach the edgedrains. Tight, low permeability subgrade material precludes ready, lateral drainage with or without edgedrains. The data obtained indicates that most of the lateral subgrade water movement is via voids and channels that develop under the pavements. In a sense, "the horse is already out of the barn" if the deterioration of the highway has reached this state. If highway restoration as well as new construction includes providing a permeable subgrade as well as edgedrains, the two together should prove the most efficient in restoring the highway. Of note too is the effectiveness of an asphalt topping, such as that applied at the Oregon and West Virginia sites .

SUGGESTIONS FOR FURTHER STUDY

One if the difficulties in defining the efficiency of longitudinal edgedrains is the lack of comparison of the same or like highway element before and after the addition of edgedrains. It is suggested that data, similar to those collected in this study, be collected at one or more highway sites for at least one year before the addition of edgedrains, and that data be collected for about a year after their addition. Analysis of the results of this study indicate that the collection of soil-moisture data and water-tracer tests could be omitted. Laboratory analysis of subgrade core samples for physical and hydraulic properties might need to be amplified compared to that performed in this study.

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