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IMPROVEMENT OF THE CANAL-AQUIFER FLOW REGIME IN THE C-1N BASIN

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INTRODUCTION

Significant exchanges of water occur between canals and the Biscayne aquifer in southern and eastern Dade County. This active exchange is possible by virtue of the high transmissivity of the Biscayne aquifer, a water table aquifer in which the canals are entrenched. Generally, this ease of exchange results in a high rate of canal seepage losses, which poses a formidable management problem. The magnitude of the unregulated canal seepages appears to be closely correlated with areas of the underlying aquifer having a high transmissivity. In general, low heads develop across canal structures where the aquifer transmissivity is high thereby preventing optimal canal stages from being maintained during the dry season. With canal and subsequently water table stages below optimal levels, it is not possible to maintain sufficient heads across some coastal structures to prevent saltwater intrusion into the canals during the dry season.

Low groundwater levels and the resultant intrusion of saltwater from Biscayne Bay have adverse effects on agricultural interests, water supply wells located near the canals and to the biologic community in general. Agricultural interests east of the coastal ridge are affected by increased soil salinities residual to the evaporation of saline groundwater. Along the coastal ridge the low groundwater levels increase pumping costs, cause priming problems to suction-lift irrigation pumps and increase the need for crop irrigation. The contamination of the Biscayne aquifer by saltwater poses a threat to both domestic and municipal water supplies. The lowered water table results in adverse impacts on the biologic community by permitting the drying and subsequent oxidation or burning of muck soils. In addition, the lowered availability of soil and groundwater to plant communities results in increased plant stress and an increased fire hazard. Thus, the resolution of the canal seepage problem will benefit all water users in the south Dade area including the biologic community.

PURPOSE

The purpose of this investigation is to consider various remedial actions and management policies that will reduce the adverse effects of the largely uncontrollable discharge of groundwater via the C-1N canal above S-149. The remedial action appearing to be the most feasible and offering the greatest flexibility of use from a water management point-of-view is investigated on a field scale. The investigation is based on approximately 1 year of field data.

ACKNOWLEDGMENTS

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The work was completed under the supervision of Abe Kreitman, Chief, Groundwater Division.

HYDROLOGIC DESIGN

The designs of canals C-1, C-1N, and structures S-148, S-149 and S-21, are described in three volumes (Corps of Engineers, 1959, 1960 and 1962). Maps showing the locations of these canals and structures and the physical features of the C-1N basin are presented in Figures 1 and 2. The design functions of the system are as follows (from Corps of Engineers, 1962):

(1) Remove 40 percent of the SPF (Standard Project Flood).

(2) Reduce the depth and duration of floods greater than the 40 percent SPF.

(3) Prevent overdrainage of the area by maintaining optimum groundwater and surface water levels and discharge rates.

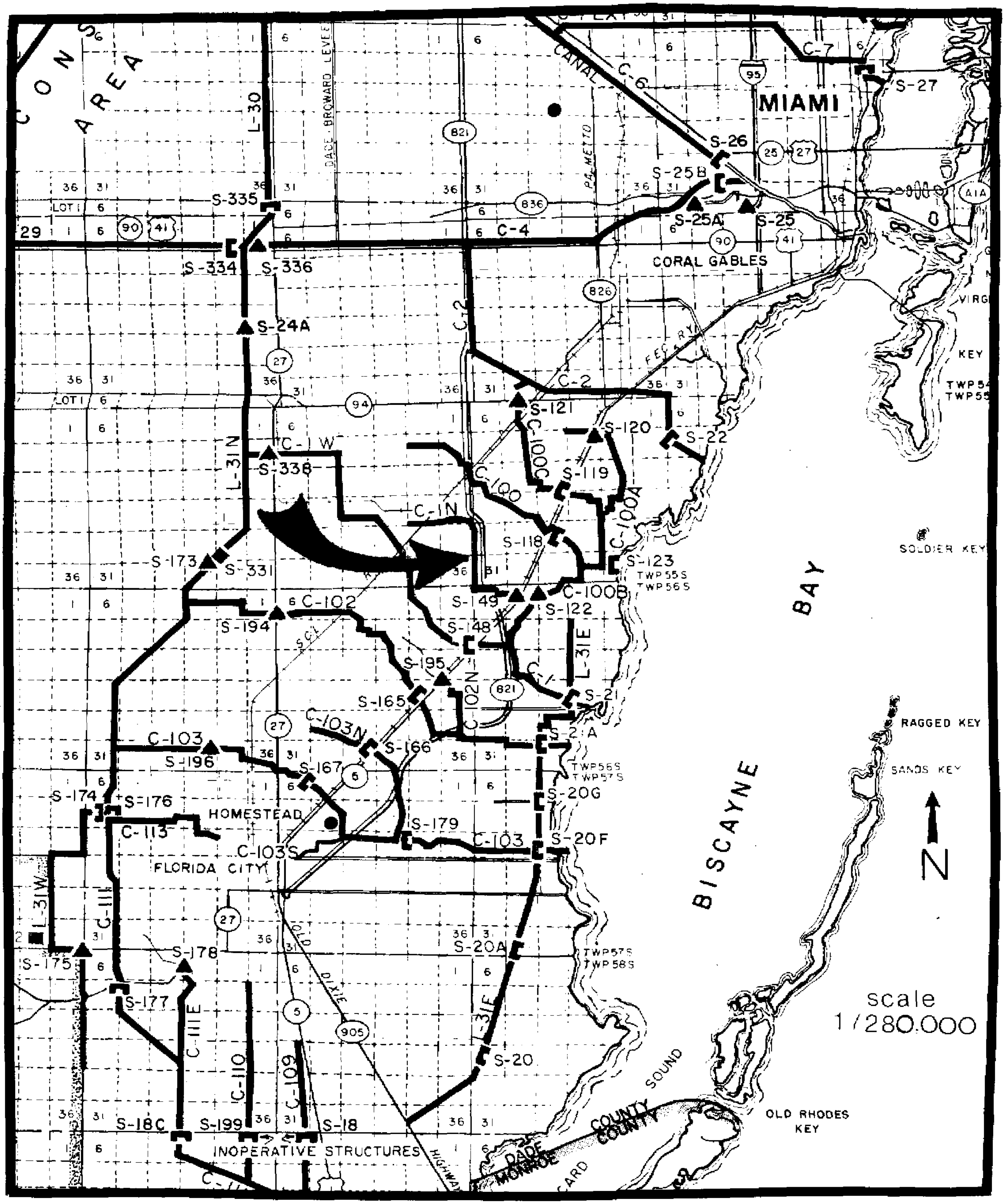


Figure 1. Map showing location of C-1N and other Project canals.

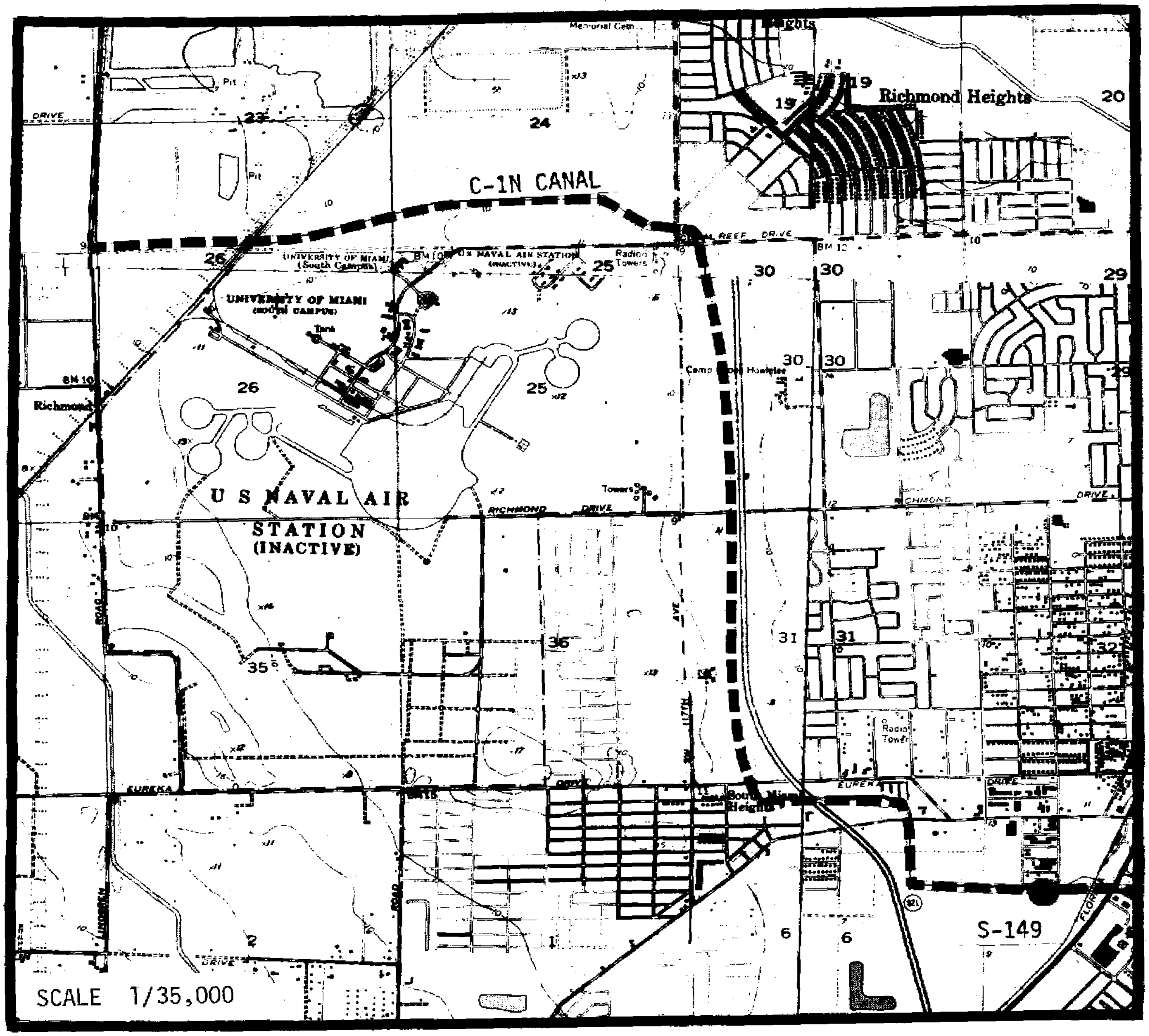


Figure 2. Map showing C-1N, S-149, and adjacent features.

In addition to these functions, the system provides the opportunity to minimize saltwater intrusion by maintaining high groundwater levels and permitting the transfer of water to areas threatened by saltwater intrusion.

A hydrologic evaluation of the proposed C-1N canal was performed by the Corps of Engineers for the 10-year and the SPF (100-year X 1.25) storm events. The 10-year peak flows were found to be approximately equal to 40 percent of the SPF flow. In most areas of the Central and Southern Florida Flood Control Project, 10-year peak flows are equal to 30 percent of the SPF flow. The hydrologic analysis technique employed was one of rainfall excess determination and the application of unit hydrographs. The rainfall depths used were derived from previous Corps of Engineers studies. The monthly ET (evapotranspiration) was proportioned to the rainfall distribution for the month. Under this method of ET application about half of the ET for the month would occur during the five day long storm event (10-year event defined by Corps), when in reality there should be little ET considered for such days. Routing of the 10-year rainfall event is listed below:

DAY	RAINFALL (INCHES)	GROUND STORAGE (INCHES)	E.T. (INCHES)	RUNOFF (INCHES)
0	0.00	0.00	0.00	0.00
1	0.60	0.42	0.18	0.00
2	0.75	0.52	0.23	0.00
3	1.05	0.73	0.32	0.00
4	2.05	0.41	0.63	1.01
5	7.75	0.06	2.37	5.32
	12.20	2.14	3.73	6.33

According to the Corps of Engineers' evaluation only 2.14 inches of the 12.20 inches of rainfall infiltrates to the aquifer. If the aquifer is assumed to have a specific yield of 0.20, this would mean that the antecedent groundwater level was considered to be about one foot below ground surface. As evidenced from a U. S. Geological Survey contour map (Figure 3) showing average October

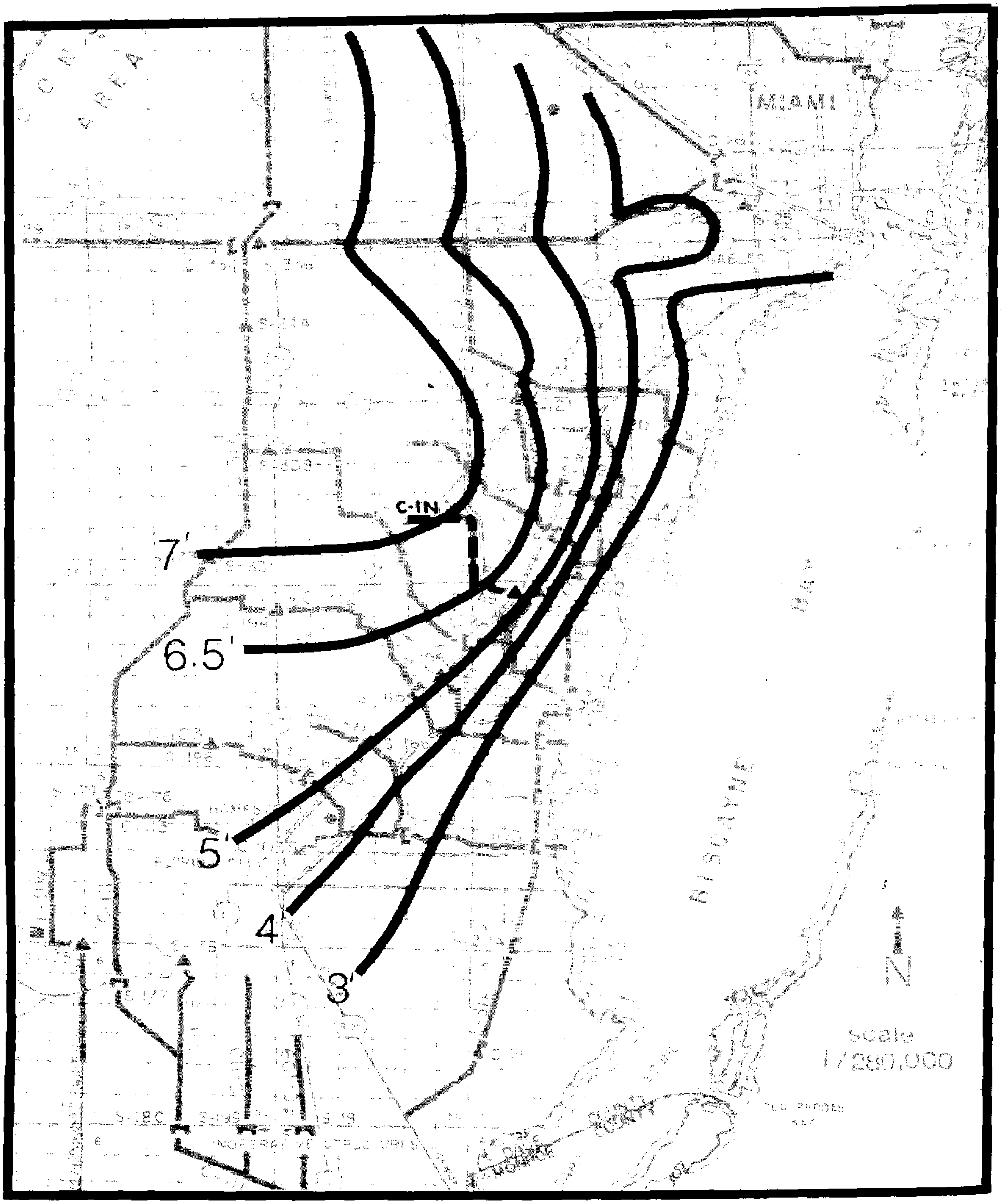


Figure 3. Contours of average October groundwater levels from 1940 to 1957 (from U. S. Geological Survey, unpublished map).

(maximum annual water table stages usually occur during October) groundwater levels for the period 1940-1957 (prior to Project construction and resultant drainage effects), the storage available would have been about 2.5 feet, which is equivalent to six inches of free water. Although more water was routed to ET than to infiltration, their sum (5.87 inches) is approximately equal to the actual ground storage available. Thus, the runoff estimate, although erroneously calculated, is quite close to that expected considering the predrainage groundwater levels.

For estimation of flow rates from direct runoff, a unit hydrograph was employed. The hydrograph relationships used were taken from previous work for the Boynton and Delray canals (C-16 and C-15), where the geology and topography would cause significantly greater runoff than could be expected in the C-1N area. The 10-year storm event for the entire 11.3 square mile drainage area of the C1N canal produced an estimated peak flow of 510 cfs (cubic feet per second), including a base flow of about 20 cfs. At structure S-149 the peak design flow is 400 cfs. Table 1 presents a summary of the hydraulic design of C-1N.

PERFORMANCE OF S-149 IN MAINTAINING DESIGN STAGES

Since S-149 was completed in June,1963, the automatic gate has only opened once (November 1, 1969), for a six hour period following a rainfall of over seven inches. The peak flow rate is not known. Following that event, the highest mean daily stage was 5.81 feet. Numerous gate openings would be expected during the 15 years of operation of S-149, having been designed for a 10-year storm event. Thus it appears that the C-1N canal above S-149 can convey the runoff from a greater than 10-year storm event.

The reason for the lack of surface water drainage is apparent when average October groundwater levels for the period 1960-1975 are considered (Figure 4). A mean October groundwater elevation of about 4.5 feet msl is indicated. This

STATION TO STATION	LOCATION	DESIGN WATER SURFACE ELEVATION (FT., M.S.L.)	BOTTOM ELEVATION (FT., M.S.L.)	BOTTOM WIDTH (FT.)	DESIGN DISCHARGE (c.f.s.)
0+00 to 103+50	Existing canal	3.2 to 3.3	(Existing canal section adequate)		
103+50 to 180+00	104+00 = U.S. Hwy. 1 107+50 = F.E.C. Ry.	3.3 to 3.5	-6.0	10	400
108+00 to 109+00		3.5 to 3.7	Transition	10	
109+00 to 116+10		3.7 to 3.8	-8.0	10	
117+10 to 118+10	117+60 = Control Structure 149	3.8 to 5.0	-8.0 to -4.0	10 to 16	
118+10 to 167+00		5.0 to 6.1	-4.0	16	370-400
167+00 to 168+00	167+50 = Eureka Drive	6.1	-4.0	Transition	370
168+00 to 358+50		6.1 to 6.8	-4.0	10	60-370
358+50 to 359+50	359+00 = S.A.L. R.R.	6.8 to 6.9	Transition	10	
359+50 to 382+00	382+00 = End	6.9 to 7.0	-1.0	10	40-60

NOTE: All side slopes are 1 vertical on 1 horizontal, design flood equals 40 percent of the Standard Project Flood

Table 1. Summary of the C-1N design (Corps of Engineers, 1959 and 1962).

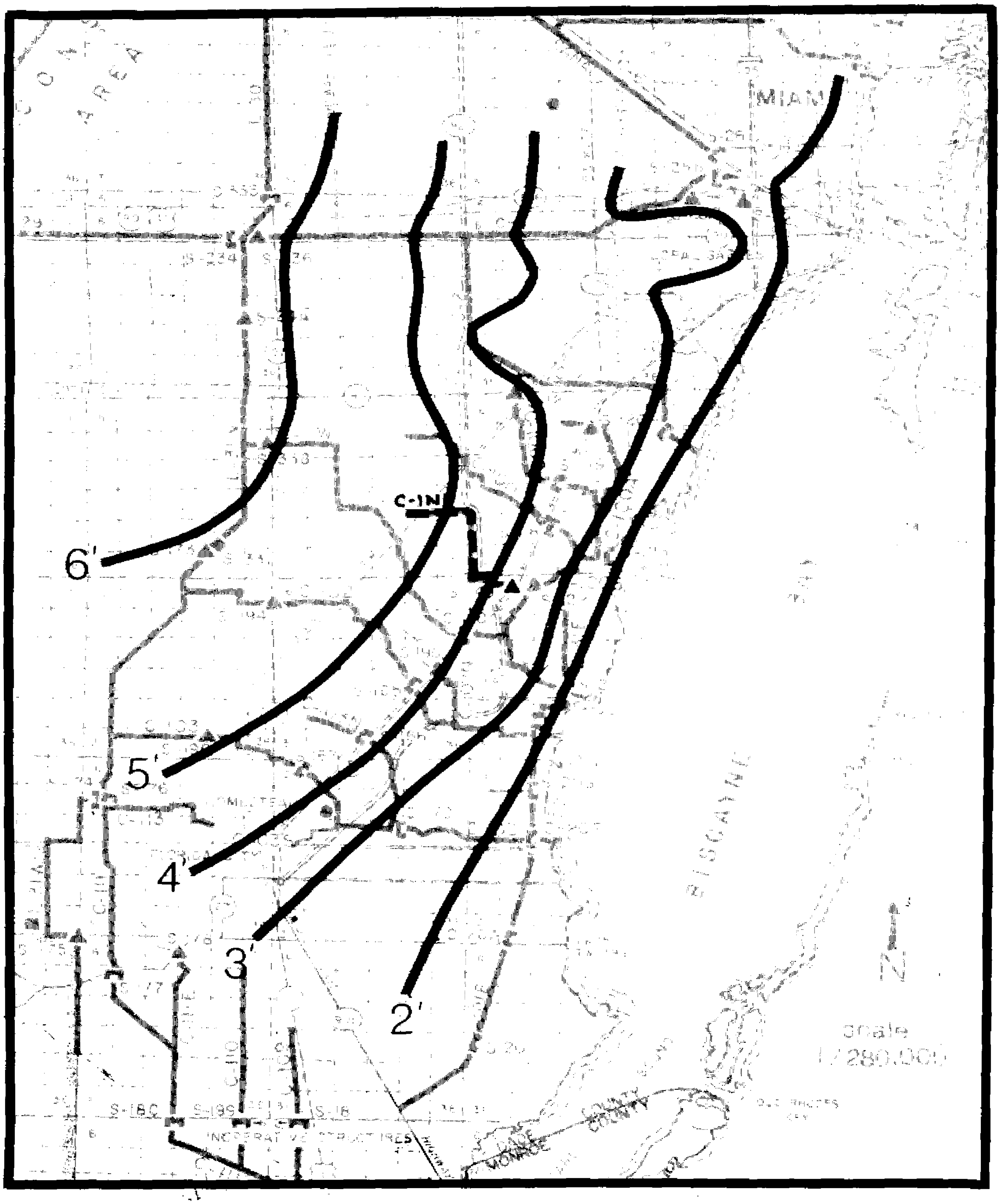


Figure 4. Contours of average October groundwater levels 1960 to 1975 (U. S. Geological Survey, 1977).

mean value is somewhat higher than the post-Project mean, because C-1N and S-149 were completed in mid-1963. The land surface elevation is fairly consistent at about 10.0 feet msl. Assuming an aquifer storage coefficient of 0.2, the entire amount of rainfall representing the 10-year event could be accommodated in aquifer storage. The storm routing would be approximately as follows:

DAY	RAINFALL (INCHES)	GROUND STORAGE (INCHES)	GROUNDWATER LEVEL (FT. MSL.)	RUNOFF (INCHES)
0	0.00	0.00	4.50	0.00
1	0.60	0.60	4.75	0.00
2	0.75	0.75	5.06	0.00
3	1.05	1.05	5.50	0.00
4	2.05	2.05	6.35	0.00
5	7.75	7.75	9.58	0.00
TOTAL	12.20	12.20		

This indicates that if all the rainfall from a 10-year event is held in aquifer storage, then there is no need for the structure to open during this event. As the ground surface immediately adjacent to the canal was a natural flow path and is lower than the surrounding area, the automatic gate is set to open when the headwater exceeds 6.2 feet msl, for the purpose of local flood protection. In reality, water is not retained in the aquifer until all aquifer storage is utilized, but it seeps into the canal and flows seaward, actuating the automatic gate when the stage reaches 6.2 feet msl. This occurred in 1969 as mentioned above. At high groundwater stages, a large portion of the basin inflow and outflow occurs from northwest to southeast across the basin topographic divides, wherein groundwater flows obliquely under the canal in the Biscayne aquifer.

Routing of the 100-year storm event, as defined by the Corps of Engineers, again assuming no ET losses and increased available aquifer storage, results in a total runoff of 5.18 inches as seen in the following tabulation.

DAY	RAINFALL (INCHES)	GROUND STORAGE (INCHES)	GROUNDWATER LEVEL (FT. MSL.)	RUNOFF (INCHES)
0	0.00	0.00	4.50	0.00
1	0.60	0.60	4.75	0.00
2	0.65	0.65	5.02	0.00
3	0.71	0.71	5.32	0.00
4	2.63	2.63	6.42	0.00
5	13.77	8.59	10.00	5.18
TOTAL	18.36	13.18		5.18

By calculation using the unit hydrograph, this runoff distribution would produce a peak flow of 330 cfs, which is significantly less than the design flow (10-year event) of 400 cfs at S-149. As evidenced by this demonstration, C-1N and S-149 appear to be very conservatively designed in order to maximize flood protection. This conservative approach in favor of one function has significantly contributed to the poor performance of the canal and structure with regard to its function of maintaining optimum water levels.

GEOLOGY ALONG C-1N CANAL

The C-1N canal is entrenched into the upper part of the Biscayne aquifer, a highly permeable limestone extending throughout Dade County. Approximately the upper 40 feet of limestone comprising the Biscayne aquifer in this part of Dade County is the relatively soft Miami Oolite formation which has a more developed secondary permeability in the vertical direction than in the horizontal (Parker, et. al., 1955). Thus, the vertical flow component would tend to be more significant than the horizontal flow component given equal pressure gradients. Below 40 feet, the highly permeable Fort Thompson formation extends to depths of approximately 100 feet, completing the Biscayne aquifer. The areal distribution of the extremely high transmissivity (product of the permeability and aquifer thickness) of the Biscayne aquifer is shown on Figure 5.

The detailed geology along the canal alignment was investigated by the Corps of Engineers prior to the construction of S-149 and the canal section upstream

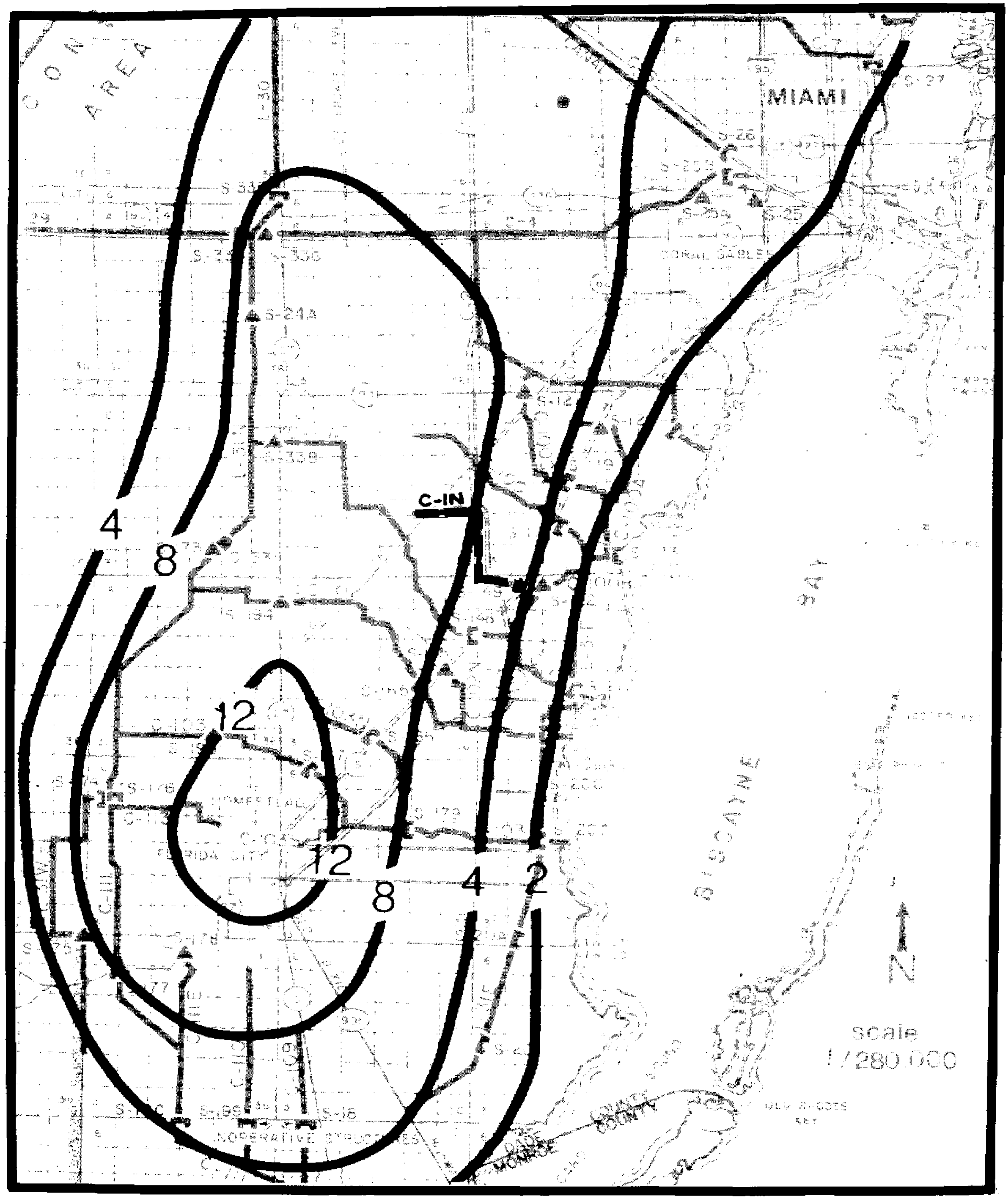


Figure 5. Contours of transmissivity for the Biscayne Aquifer in million gallons per day per foot (from U. S. Geological Survey, unpublished map).

of this structure (Corps of Engineers, 1962). Seventeen core borings were made along the canal alignment, seven of which are depicted on Figure 6. As determined by this investigation, a thin surficial layer of silty sand is underlain by a solution riddled limestone containing occasional lenses of fine grained sand to a depth of at least 45 feet. Although the solution holes in the limestone are partially filled with fine sand, the limestone is presumed to have a much higher hydraulic permeability than the fine sand. Consequently, the lenses of fine sand would behave as leaky barriers to the flow of water through the Biscayne aquifer.

At Richmond Drive, the thickest fine sand unit detected by borings along the canal forms the canal-aquifer boundary. It extends from land surface to a depth of 11 feet below the canal bottom. This sand extends 4,000 feet upstream of Richmond Drive and 7,800 feet downstream almost to Quail Roost Drive. The sand tapers in thickness to only 2.5 feet at Eureka Drive, where the top of the sand unit lies at least 4 feet below the canal bottom. Negligible sand is present at Quail Roost Drive and S-149. The considerable north-south dimension of the sand unit suggests that the east-west dimension may also be significant, having its expected maximum east-west dimension also near Richmond Drive. The importance of this sand lense, and its potential as a leaky barrier to the flow of water across the canal-aquifer interface, is discussed below.

PREVIOUS INVESTIGATIONS

The District staff conducted two investigations in the C-1N basin between 1968 and 1972. The first of these studies (from February 1, 1968 to February 8, 1972) sought to determine the configuration and dynamics of the water table in the vicinity of S-149. Twenty-eight shallow water table monitoring wells were installed within the canal right-of-way from 250 feet downstream of S-149 to 1,750 feet upstream. A typical configuration of the water table as established by 29 sets of observations is shown on Figure 7. A notable exception occurred

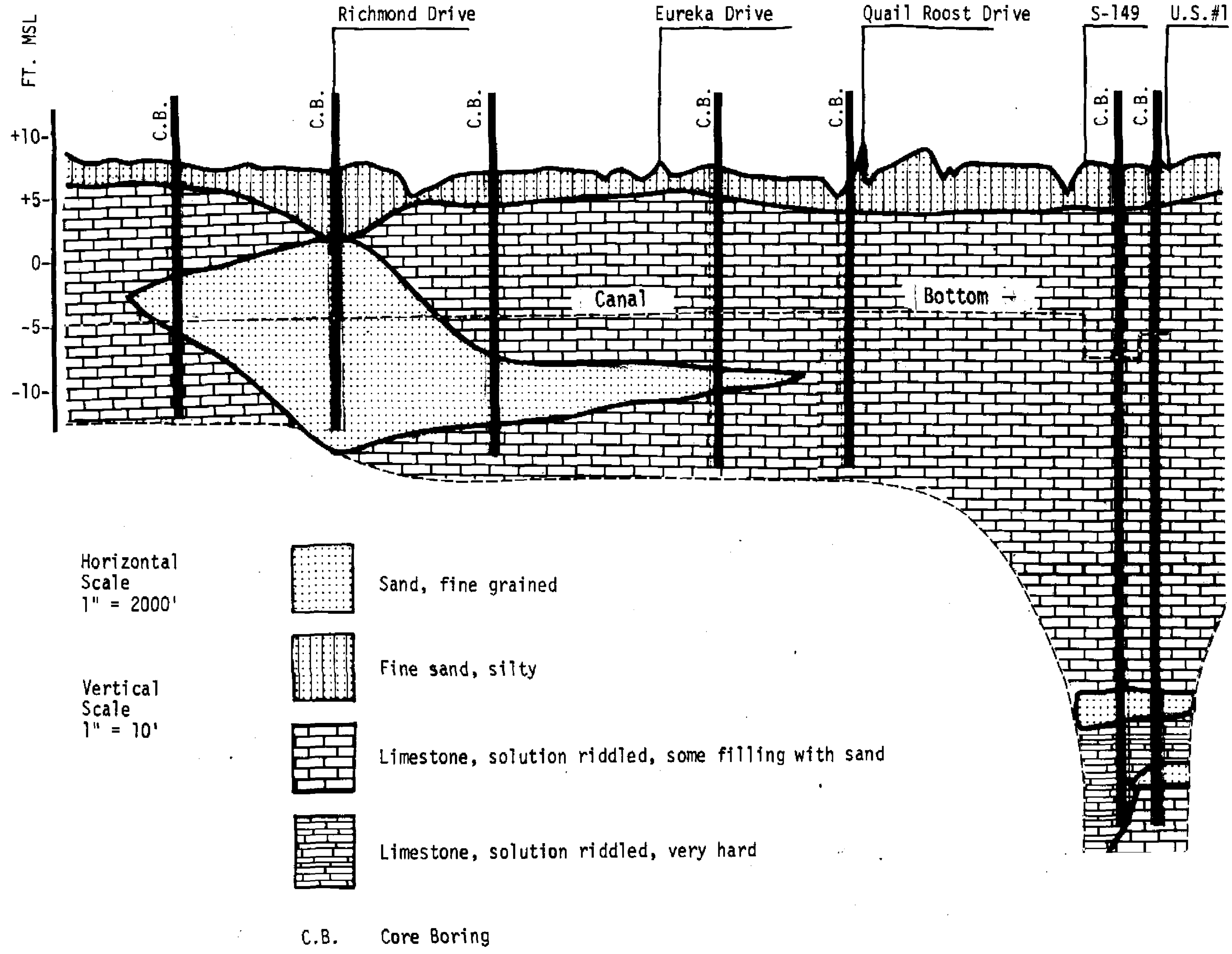


Figure 6. Geologic cross-section along the C-1N alignment showing the sand lense in the vicinity of Richmond Drive (Corps of Engineers, 1962).

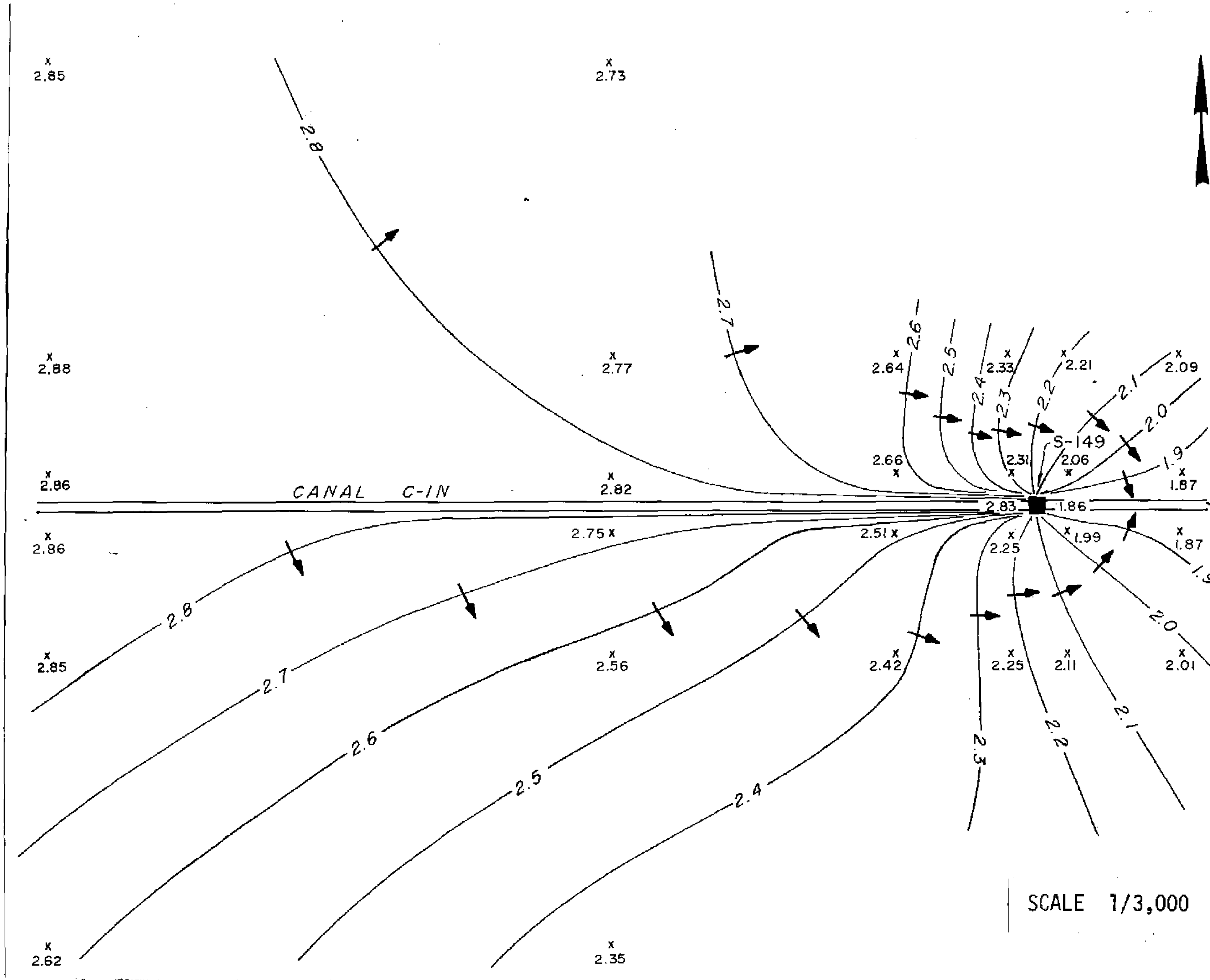


Figure 7. Groundwater level contours in feet m.s.l., showing a typical water table configuration and groundwater flow directions in the vicinity of S-149 (from South Florida Water Management District, unpublished report).

on May 14, 1971, when the head to the east of S-149 was higher than that to the west and groundwater flow was directed predominately westward. Generally, the reach of canal extending 1,750 feet upstream of S-149 experiences seepage losses. The southeastward seepage component is largely directed to the leg of the C-1N canal downstream of S-149 which is oriented NW-SE (Figure 2). Most seepage from the canal towards the north re-enters the E-W oriented reach of the canal downstream of S-149. The flow patterns depicted on Figure 7 are as would be expected with a uniform distribution of transmissivity in the Biscayne aquifer. These data suggest that the seepage problem around S-149 is regional in nature and, in combination with field observations, do not suggest faulty construction of S-149 or excessive permeability of the Biscayne aquifer in the immediate area of S-149 as could perhaps result due to blasting during construction.

In the second District study (October 29, 1970 to December 3, 1971) eight shallow water table monitor wells were installed along Richmond Drive to determine the groundwater gradient and the relationship between groundwater and canal stages. These wells were 12.5 feet deep and indicate the position of the water table. Ten sets of data were collected, all of which are graphically depicted on Figure 8.

The data derived from this study indicate that when groundwater stages are greater than 1.5 feet msl, groundwater flow in the main body of the limestone aquifer has a flow component directed from the west towards the east. At stages below 1.5 feet msl, the sand lense apparently complicates flow patterns to the extent that no firm conclusions can be drawn as to flow directions. A depression in the groundwater level profile immediately west of the canal between canal stages 2.4 feet and 3.6 feet msl, may be interpreted as indicating a westward flow of water from the canal within or above the bed of sand, after which the groundwater seeps vertically downward to the underlying zone of high transmissivity.

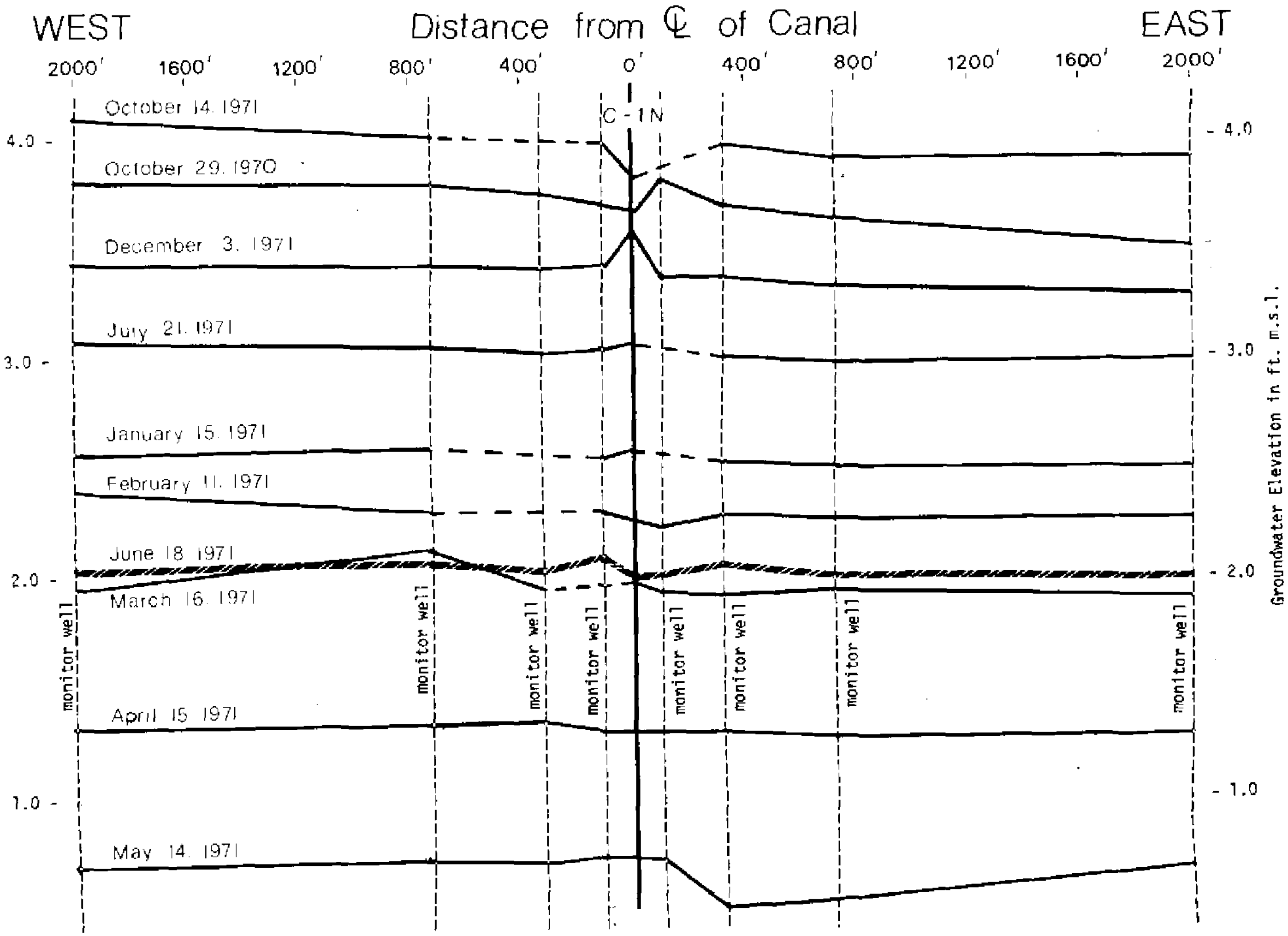


Figure 8. Cross-section of C-1N at Richmond Drive showing canal stages and adjacent groundwater stages (from South Florida Water Management District, unpublished report).

At canal stages above 3.6 feet msl, the canal appears to receive inflow from the aquifer, whereas below this elevation the canal loses water to the aquifer. As will be seen below, the Richmond Drive reach of the canal appears to be a critical point in that the upstream portion of the canal generally gains water from the aquifer while the downstream portion loses water to the aquifer. The slight migration of this gaining-losing junction along the canal in response to basinwide groundwater levels may explain the significance of the 3.6 feet msl canal stage.

ALTERNATIVES CONSIDERED

To mitigate the water management problems (i.e. the high rate of seepage from the canal), all apparently feasible alternatives were considered and compared to isolate the most promising. The following text describes the alternatives identified, the problems and the expected performance of each.

Canal Liners

Canal liners of flexible plastic were considered to decrease the loss of water from the canal above S-149. The problem of effectively anchoring the plastic to the irregular walls and the canal bottom, which are covered by sediment and debris, would be formidable. If groundwater flow were directed towards the canal, as would likely occur if S-149 were to open, the hydraulic pressure directed into the canal would tend to lift the liner away from the canal walls and bottom. If the liner were not securely anchored to the canal perimeter, it would "float" and seriously obstruct the flow of water in the canal. The chance that this would happen is considered significant; the result could be considerable property damage due to flooding.

Bentonite Sealing

The filtering process that takes place during the migration of water around and under S-149 suggests that a sealant material, such as bentonite or other clay material, added to the canal waters above S-149 would enhance both the plugging of the Biscayne aquifer and the bottom sediments within the excavated canal. The easily eroded laminae of material built up on the canal walls would have to be removed to avoid repetitive bentonite treatments. This would permit penetration of the sealant into the walls and prevent the loss of the sealant due to erosion. Erosion inducing conditions, such as weed clearing operations or high discharge in the canal, would also remove any sealant resting on the canal bottom sediments as well as on the walls; thus, permanently sealing the bottom of the canal could prove to be impossible. Because no method was identified that would allow satisfactory cleaning of the canal walls prior to treating the canal waters with a sealant such as bentonite, an experimental sealant treatment was not attempted.

A problem common to all sealants and liners is that groundwater is prevented from flowing into the canal during flooding when such inflows are desirable; a bentonite sealant would be no exception.

"V" Shaped Cement Grout Curtain

A grout curtain emplaced via angled boreholes along each canal bank and forming a "V"-shaped barrier under the canal was considered to reduce seepage losses. Vertical grout curtains would be ineffective because of the large and highly permeable open area in the horizontal plane between the two curtains. In order to significantly reduce the seepage loss, at least 2,000 feet of the canal upstream from S-149 would require a grout curtain. As in the alternatives considered above, subsurface drainage into the canal during flooding conditions would be eliminated, thereby decreasing the flood protection afforded by the canal.

Structure Wing-Wall Grout Curtain

Instead of placement along each bank of the canal, a grout curtain could be placed perpendicular to the canal at S-149. Hydrologic conditions and the canal configuration, however, are not favorable for this scheme. The presence of the C-1N canal which behaves as a groundwater sink to the southeast of S-149 would decrease the effectiveness of a cutoff wing wall by facilitating flow around the southern end of the grout curtain (the normal flow direction). The high vertical and horizontal permeabilities of the Miami Oolite and the underlying Fort Thompson formation would permit significant flow under a grout curtain that did not extend to the full depth of the aquifer. In the presence of these unfavorable conditions, it is believed that a wing wall grout curtain having a length of approximately 500 feet would not reduce the seepage losses by more than about 25 percent.

Historically, grout curtains have not been particularly effective in reducing seepage losses around dams and similar structures (Casagrande, 1961 and Cedergren, 1977). To effectively reduce the seepage loss, the cutoff wall of grout would have to have an extremely small percentage of open area. Casagrande has demonstrated that a thin cutoff wall of sheet pile having 1/16" slots, with a 0.1 percent overall open area would allow 71 percent of the normal flow to pass. The efficiency of cutoffs also depends on the distribution of the open area. If the open area occurs at one point, the cutoff efficiency is greater than if the area is distributed over several openings. For example, if 5 percent of the area is open and concentrated in one opening or distributed among eight openings, the seepage reductions are 62 percent and 18 percent respectively. These data illustrate the difficulty and uncertainty of constructing an effective grout curtain.

Additional Channel Structure

The construction of an additional channel structure above S-149 is considered to offer the greatest potential for maintaining groundwater levels and significantly reducing canal induced seepage from the basin. The construction of a water-tight channel structure poses few problems in comparison to the problems of effectively sealing the canal perimeter or constructing a grout curtain with less than 5 percent open area. Favorable hydrogeologic conditions in the vicinity of the channel, especially at the Richmond Drive site, would be expected to considerably enhance the effectiveness of a channel structure in retarding stream flow. Some measurable result can be assured with a channel structure whereas the effectiveness of a channel liner or grout curtain is uncertain; the later two are also considerably more difficult to construct. A structure above S-149 should have little effect on groundwater levels in the urbanized area near S-149, thus maintaining the existing degree of flood protection in that area. Upstream of the added structure groundwater levels would be measureably increased. Flood protection in this area would be decreased but still within design limits. The net result of emplacing a new structure would be greater retention of water in the basin, thus reducing seepage losses. If an added structure were fitted with an operable gate, the increased groundwater in storage could be transferred downstream to the area where saltwater intrusion occurs in the channel by opening the gate. This management action combined with a partial opening of the coastal structures during low tides would result in less extensive saltwater intrusion in the canals. An alternative and less expensive structural arrangement would be an erodable earthen plug which would yield to overtopping and allow flood crests to pass. The relatively high probability of maintaining groundwater levels and reducing the seepage losses from the basin, as well as increasing the management options as a result of using an additional structure convinced the authors to pursue the investigation of this alternative in greater detail.

No Action As An Alternative

In addition to the alternatives described above, the no-action alternative deserves consideration.

Since the construction of C-1N and S-149, fine grained sediments and organic muck have accumulated in the canal, especially in the reach of the canal where flow velocities reduce due to seepage losses. It was observed that canal water is turbid upstream of S-149 but clear downstream of the structure. The materials causing turbidity are obviously being filtered from the water as it seeps through the aquifer and discharges downstream of S-149. The filtered material can be expected to reduce the rate of seepage from the canal as has been documented near the Miami Springs - Hialeah Wellfields (Meyer, F. W., 1972).

The meager canal discharge data available suggest that canal seepage rates above S-149 have decreased since the construction of the canal. Simultaneous measurements of flow at Eureka Drive and head differential (difference between the upstream and downstream stages) at S-149, on May 28, 1968, August 23, 1976, and June 1, 1977, document this phenomena. The three ratios of flow to head differential, which is directly proportional to the canal loss by seepage, are chronologically 27.66 cfs/ft., 18.70 cfs/ft., and 18.30 cfs/ft. These data indicate a reduction of seepage loss of about 34 percent in a period of 10 years.

Assuming the next 10 year period will witness an additional reduction in seepage loss, the no-action alternative is given consideration.

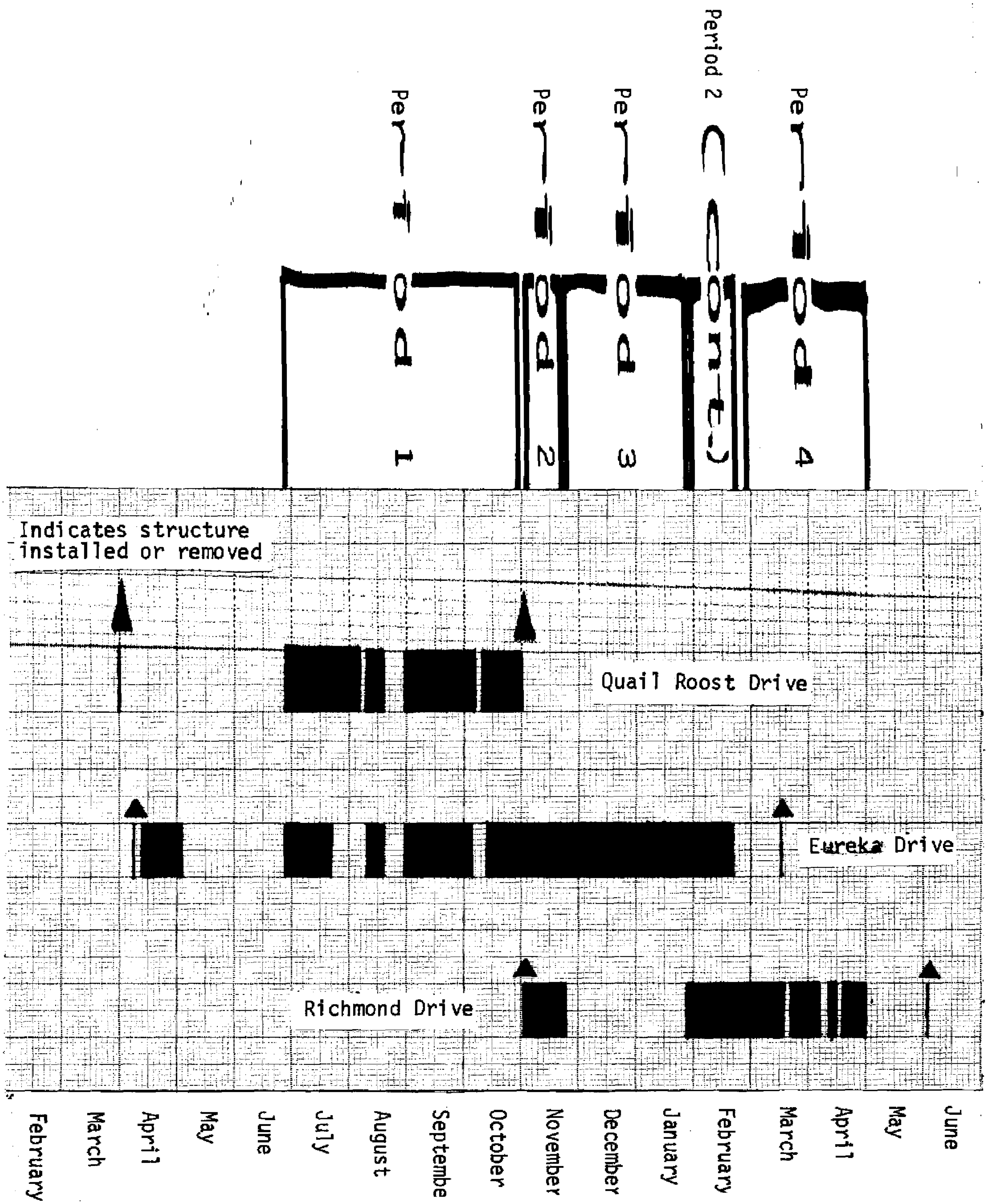
FIELD STUDY PROCEDURES

To further evaluate the effectiveness of the additional permanent channel structure alternative, three culverts in the C-1N canal were blocked at various times by temporary structures. The most favorable of the three sites was expected to show the highest head differential, thus retarding the most seepage loss.

Two distinct sets of data were collected during the 17 month field study. Simultaneous stage and discharge measurements were made on a monthly basis to establish an understanding of the drainage regime. In addition, daily differential stage measurements were made at S-149 and at each operating temporary structure to enable comparisons of their individual performances.

The three temporary structures were installed at Quail Roost Drive, Eureka Drive and Richmond Drive. The operation schedule and the location of each structure are shown on Figures 9 and 10, respectively. The Quail Roost Drive location was chosen to test whether most seepage loss was occurring between Quail Roost Drive and S-149. If this were true, a large head differential would be expected at Quail Roost Drive, assuming similar hydrogeologic conditions at the two sites. The Richmond Drive site was chosen to evaluate the effectiveness of the very favorable hydrogeologic condition (i.e. the sand lense) at this site. The Eureka Drive site was chosen as an intermediate site between Quail Roost Drive and Richmond Drive.

The structures installed at Quail Roost Drive and Eureka Drive were constructed of fencing draped with plastic sheeting, held in place by 4" X 4" timbers spanning the upstream end of the culverts. The structure at Richmond Drive was constructed of plastic overlying stacked 4" X 4" timbers. The tops of the structures were set 1.5 ft. below design stages to allow for overtopping. This was considered necessary to minimize the probability of damage due to flooding. Leakage through the dams and into the culverts was estimated to be


Period 1
Period 2
Period 3
Period 2 (cont.)
Period 4
Indicates structure installed or removed
Quail Roost Drive
Eureka Drive
Richmond Drive
February
March
April
May
June
July
August
Septembe
October
November
December
January
February
March
April
May
June

generally less than 1 cfs; this leakage would not significantly affect the measured differential head. The structures were frequently rendered inoperative by vandals. Elevations of reference points for canal stage measurements were determined by SFWMD survey crews.

STAGE PROFILES

Canal stage data were collected at 10 sites on 20 occasions (Appendix A). Stages were determined by chalked tape measurements at all sites except at S-149 where permanent staff gages were used. Figures 11 to 25 indicate stages in feet msl versus distance upstream of S-149 and the differential heads across operational structures. The chalked tape measurements were occasionally affected by wind deflection of the tape and waves up to several inches on the canal surface. At S-149, significant downstream stage fluctuations were observed due to gate openings and closings on downstream structures, especially during the wet season. On analysis, the stage data are believed to be of acceptable accuracy.

Canal stages respond to rainfall events in some unexpected ways. Generally, the canal flow was directed towards S-149 during the study period; however, on May 6, 1977, the flow was directed upstream from S-149 apparently due to the large storage capacity provided by a rock pit adjacent to the canal. This strongly suggests that the basin response to rainfall is not uniform. Unusually heavy or a nonuniformly distributed rainfall would likely require several days for near steady-state equilibrium conditions to develop throughout the canal system. Thus, the stage data should be considered representative of an instant in time as taken from a constantly varying time-dependent system. Daily stage differential data collected at the temporary structures for the purpose of structure site comparisons, suggest that stages require up to 5 days for stabilization after a structure is placed into operation.

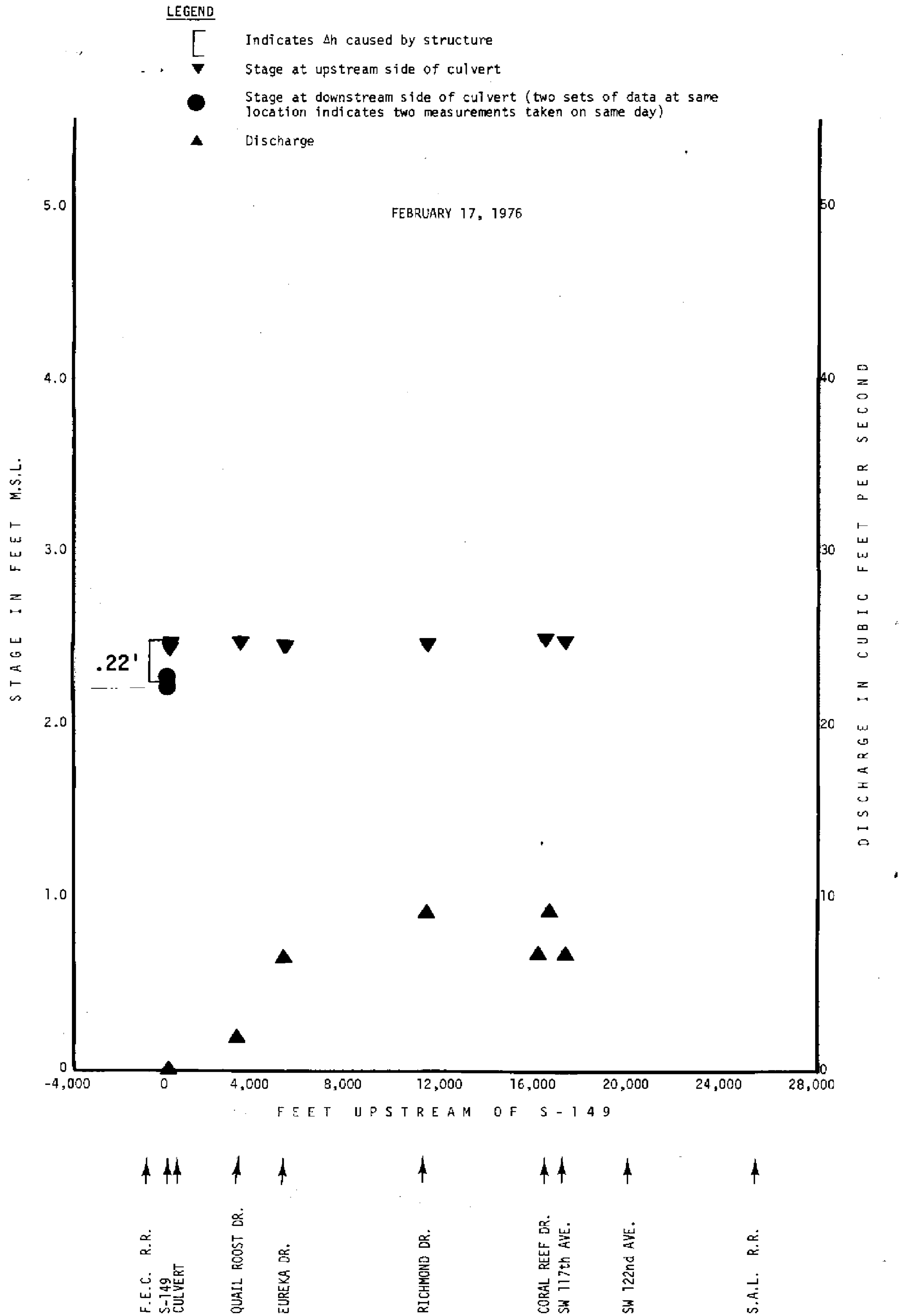


Figure 11. Stage and discharge profiles on February 17, 1976.

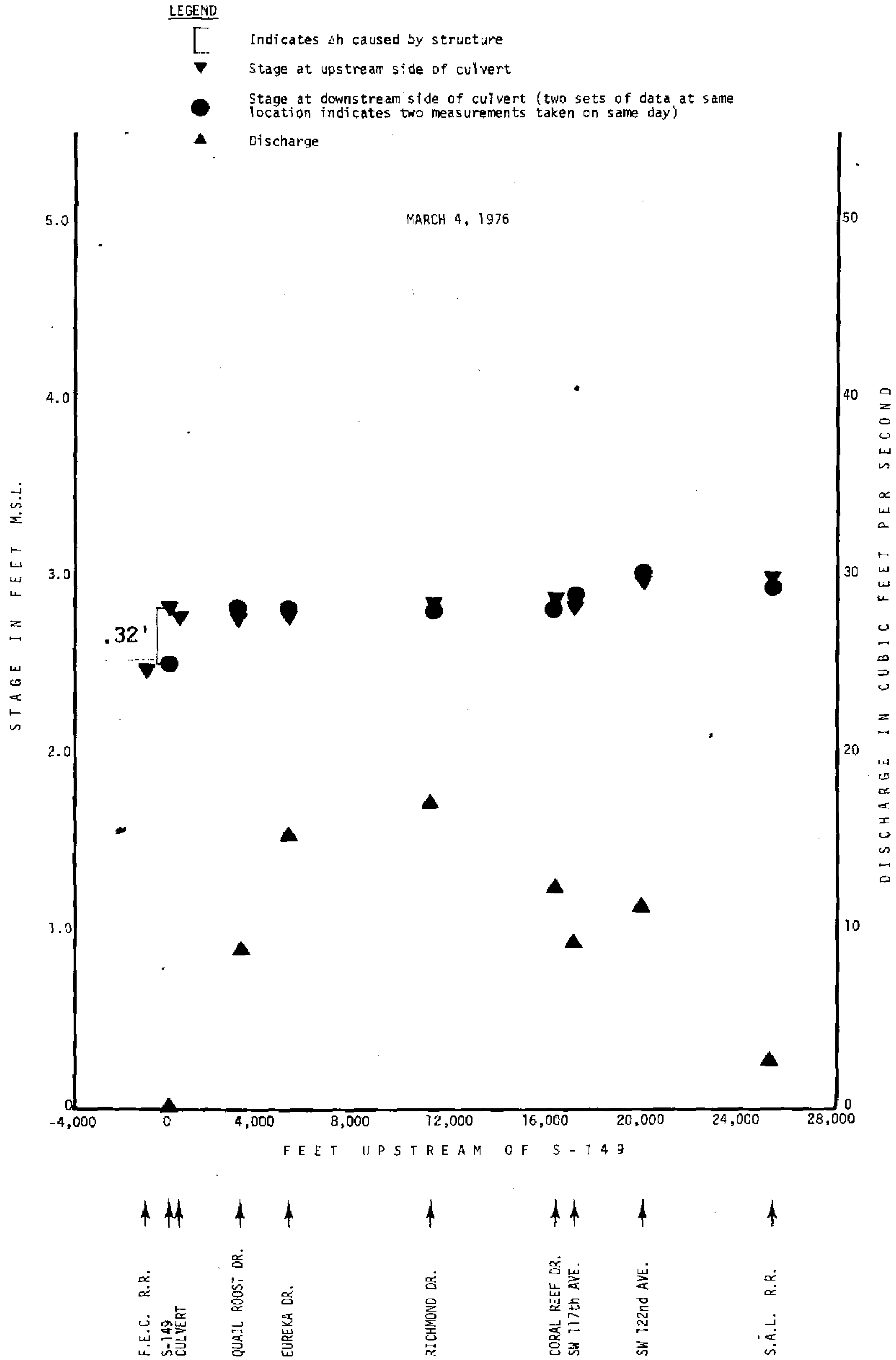


Figure 12. Stage and discharge profiles on March 4, 1976.

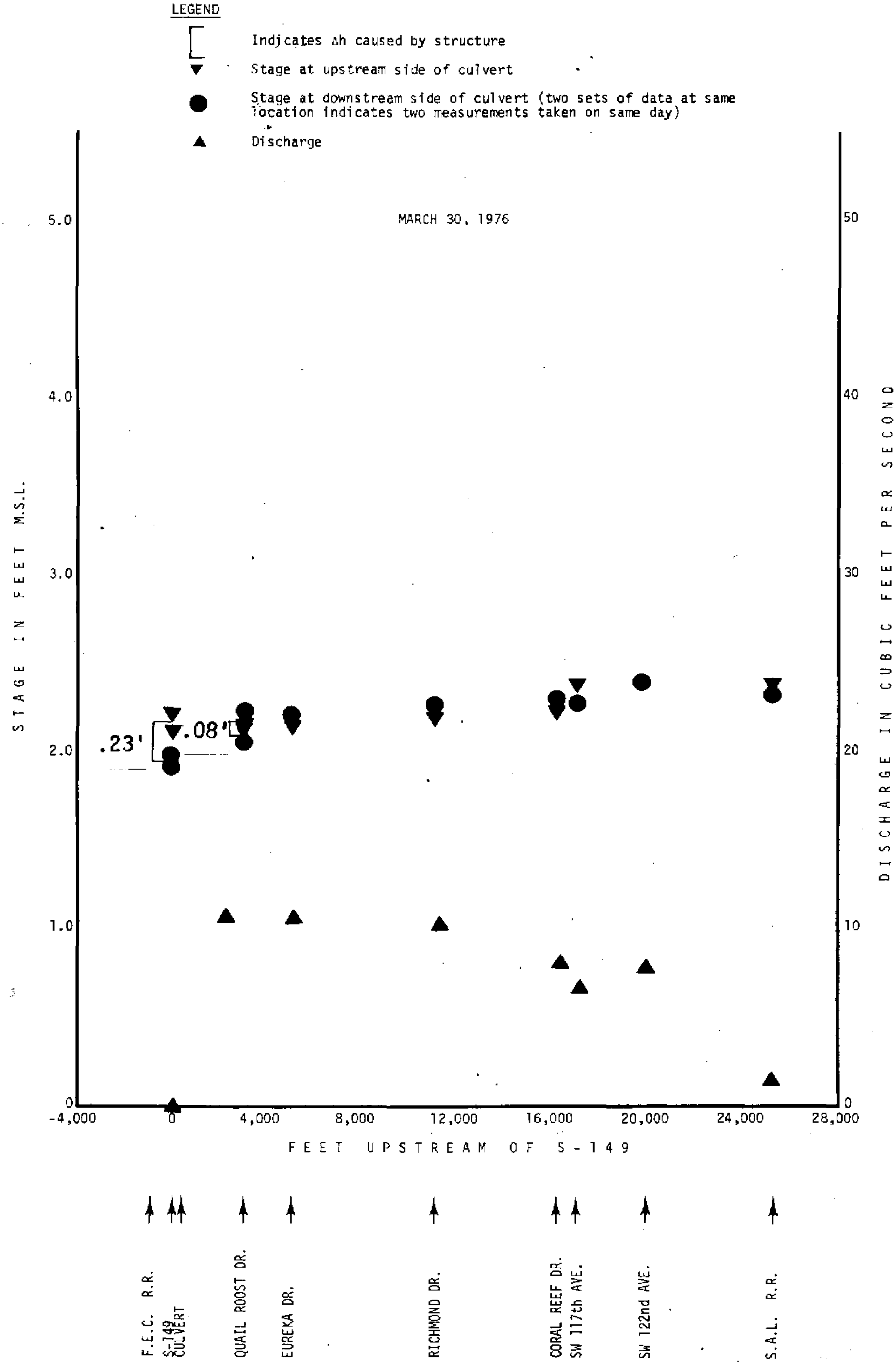


Figure 13. Stage and discharge profiles on March 30, 1976.

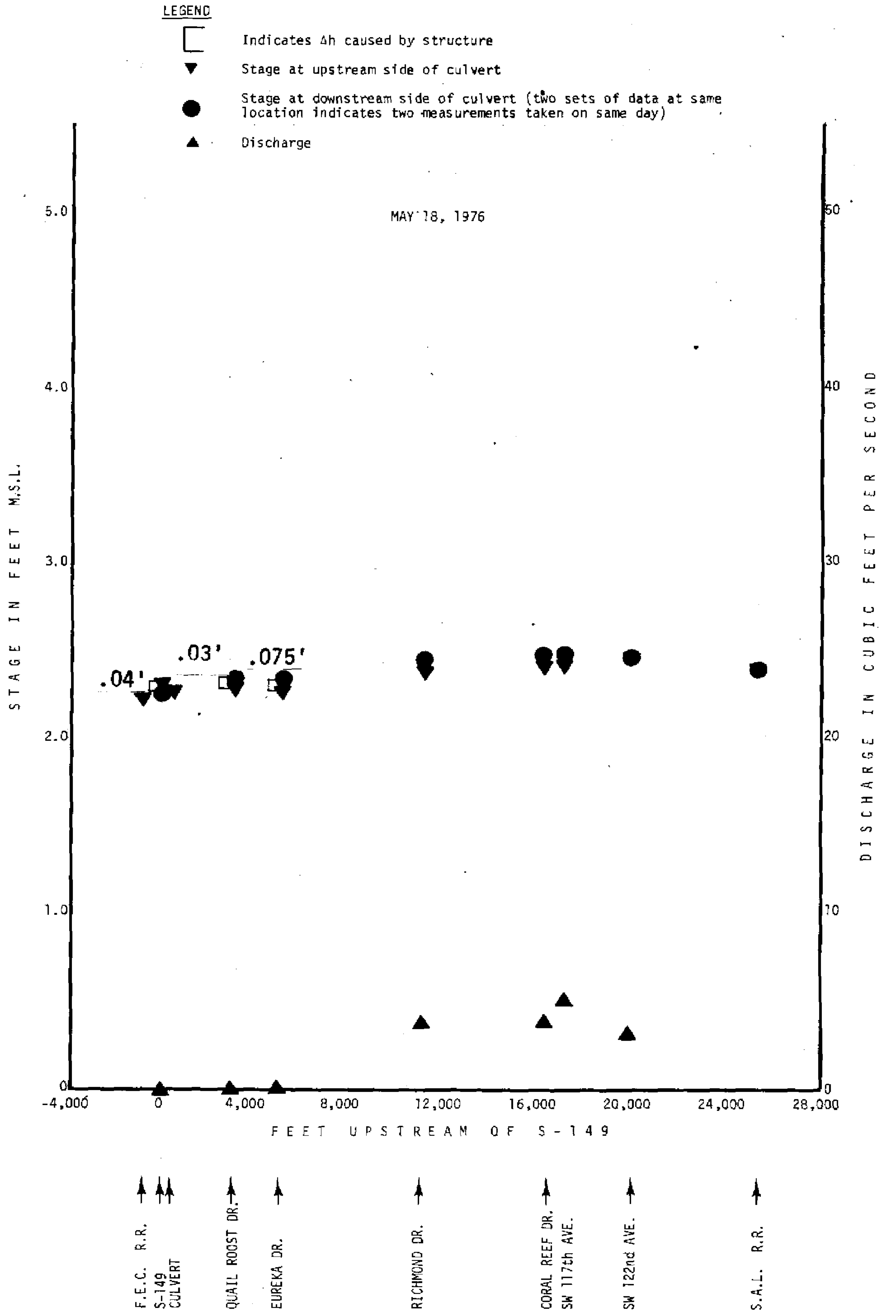


Figure 14. Stage and discharge profiles on May 18, 1976.

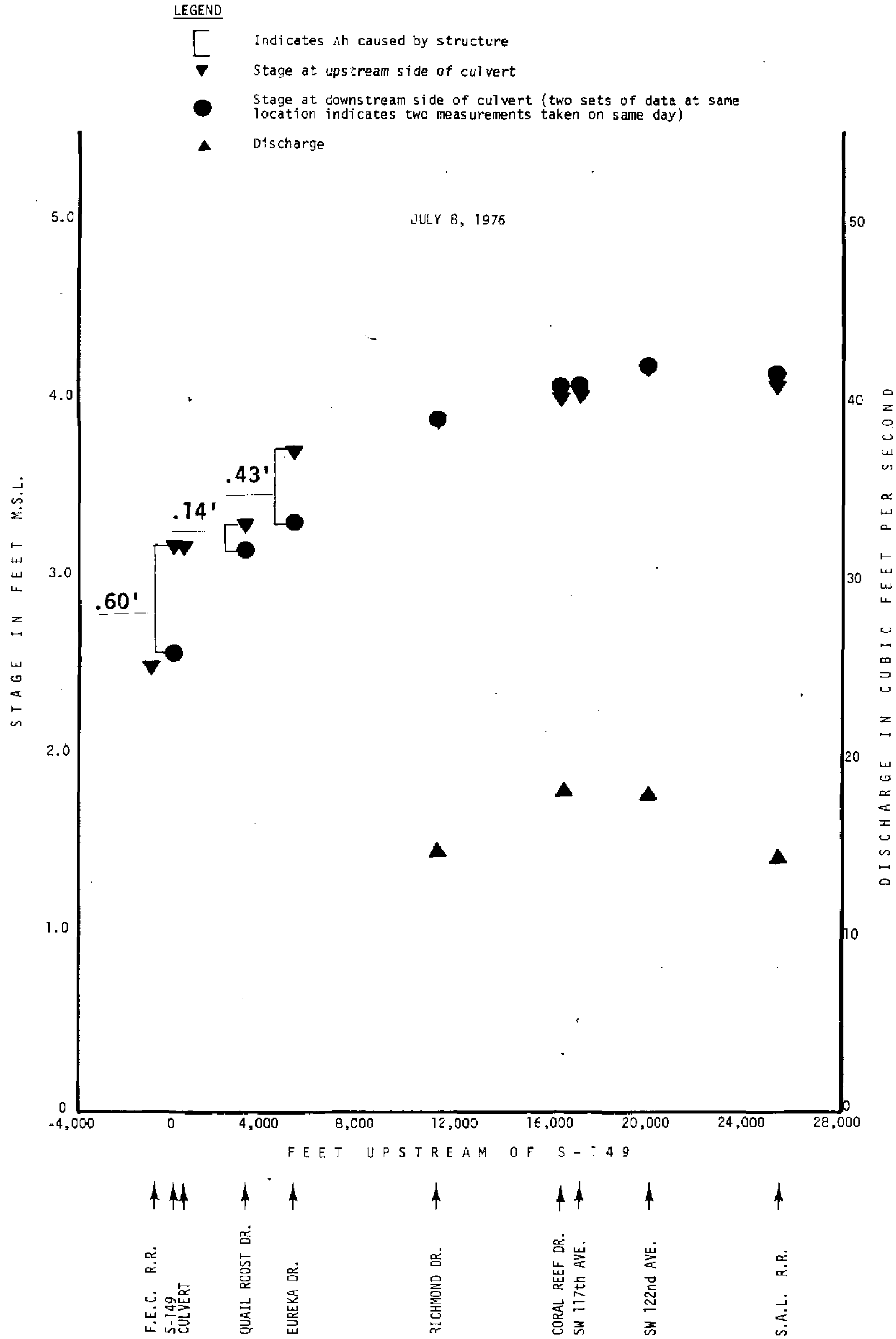


Figure 15. Stage and discharge profiles on July 8, 1976.

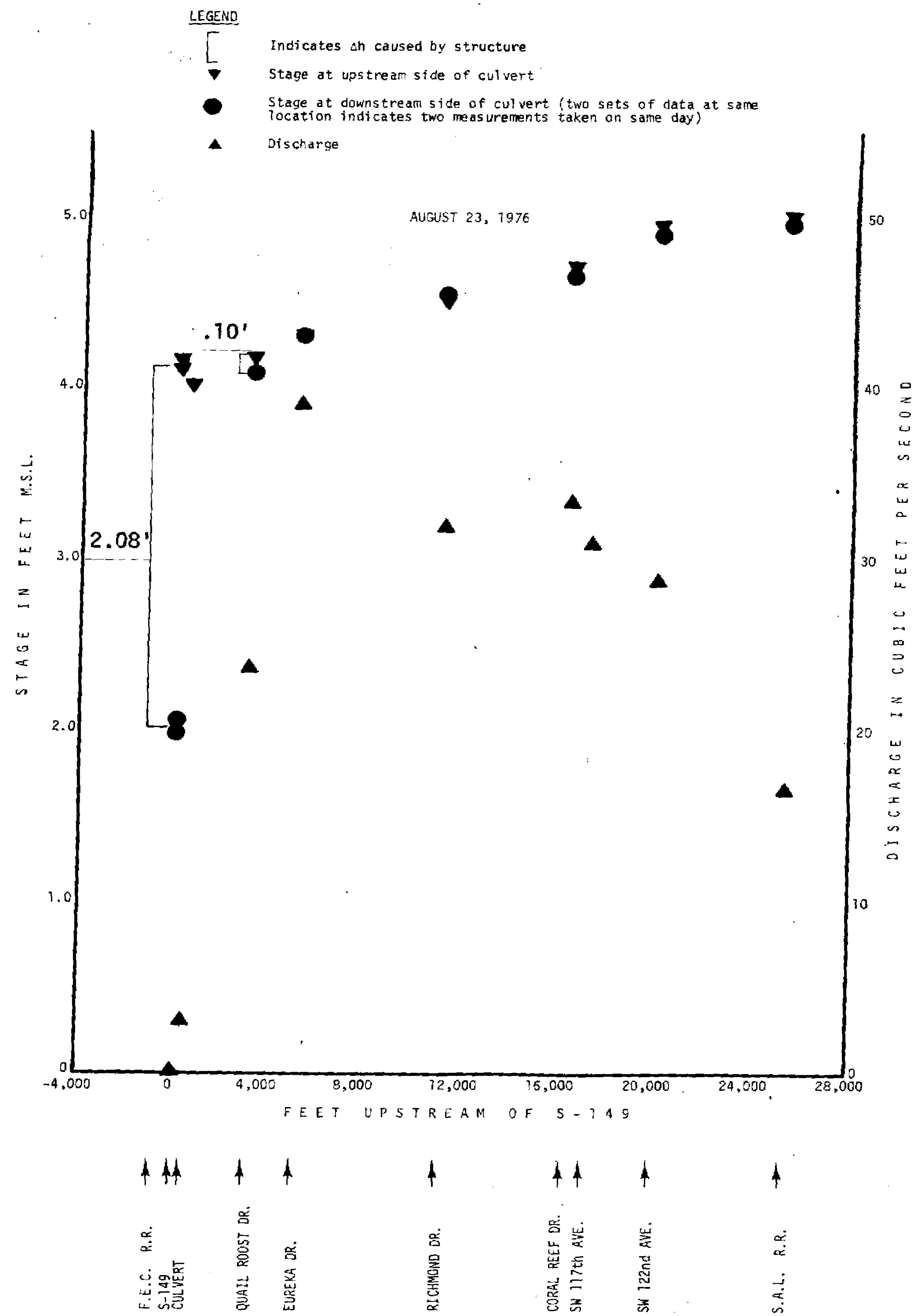


Figure 16. Stage and discharge profiles on August 23, 1976.

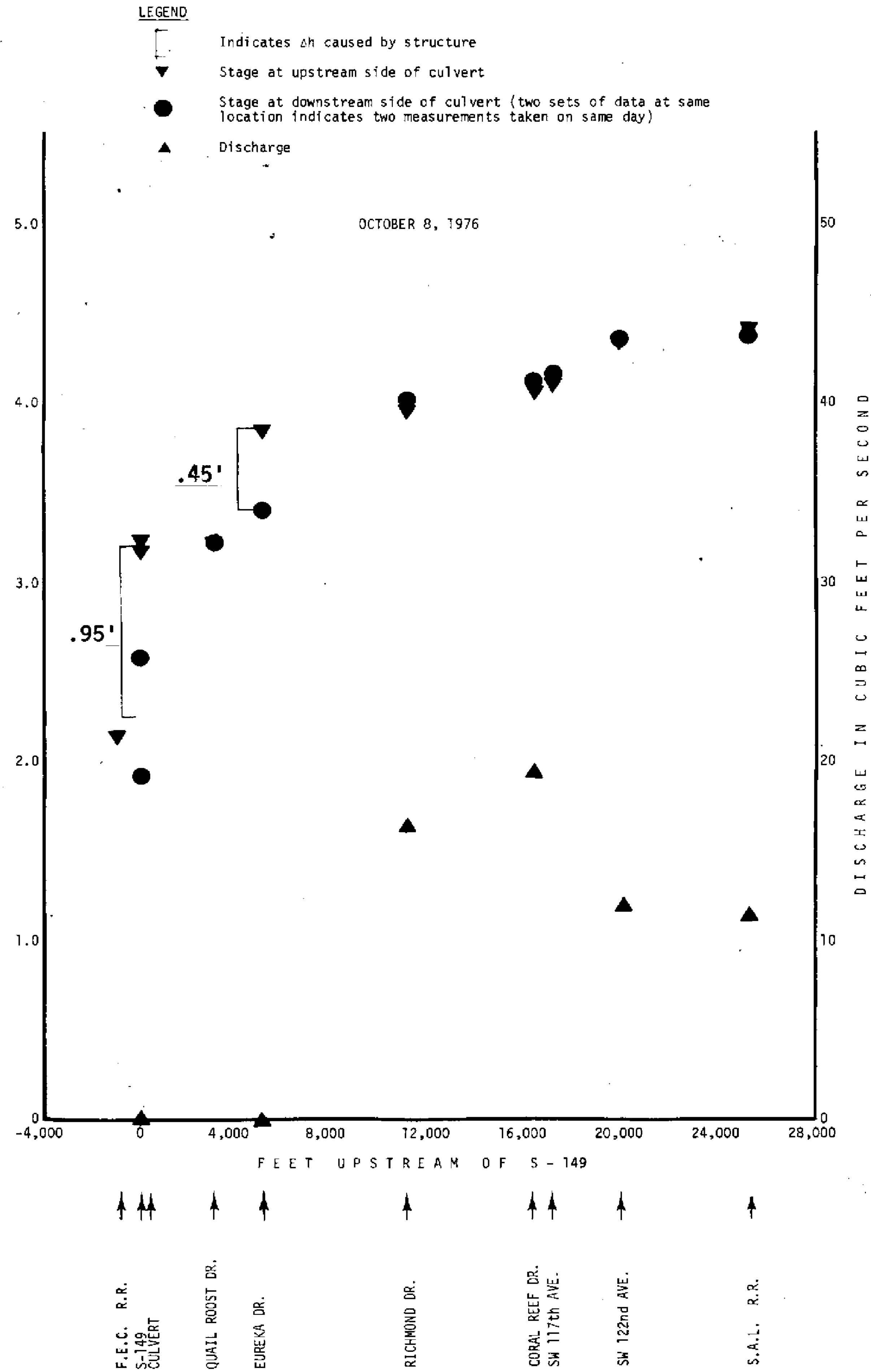


Figure 17. Stage and discharge profiles on October 8, 1976.

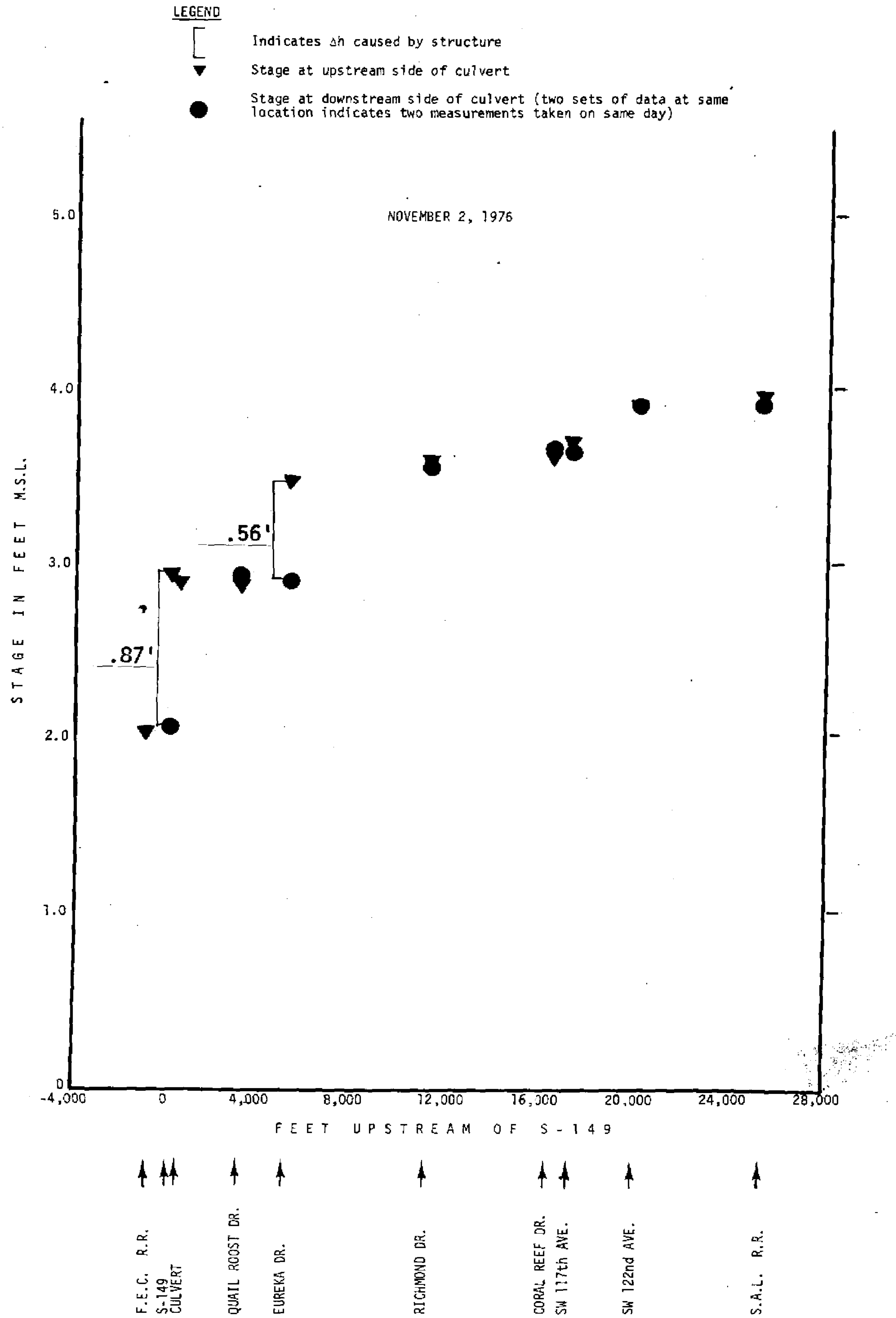


Figure 18. Stage and discharge profiles on November 2, 1976.

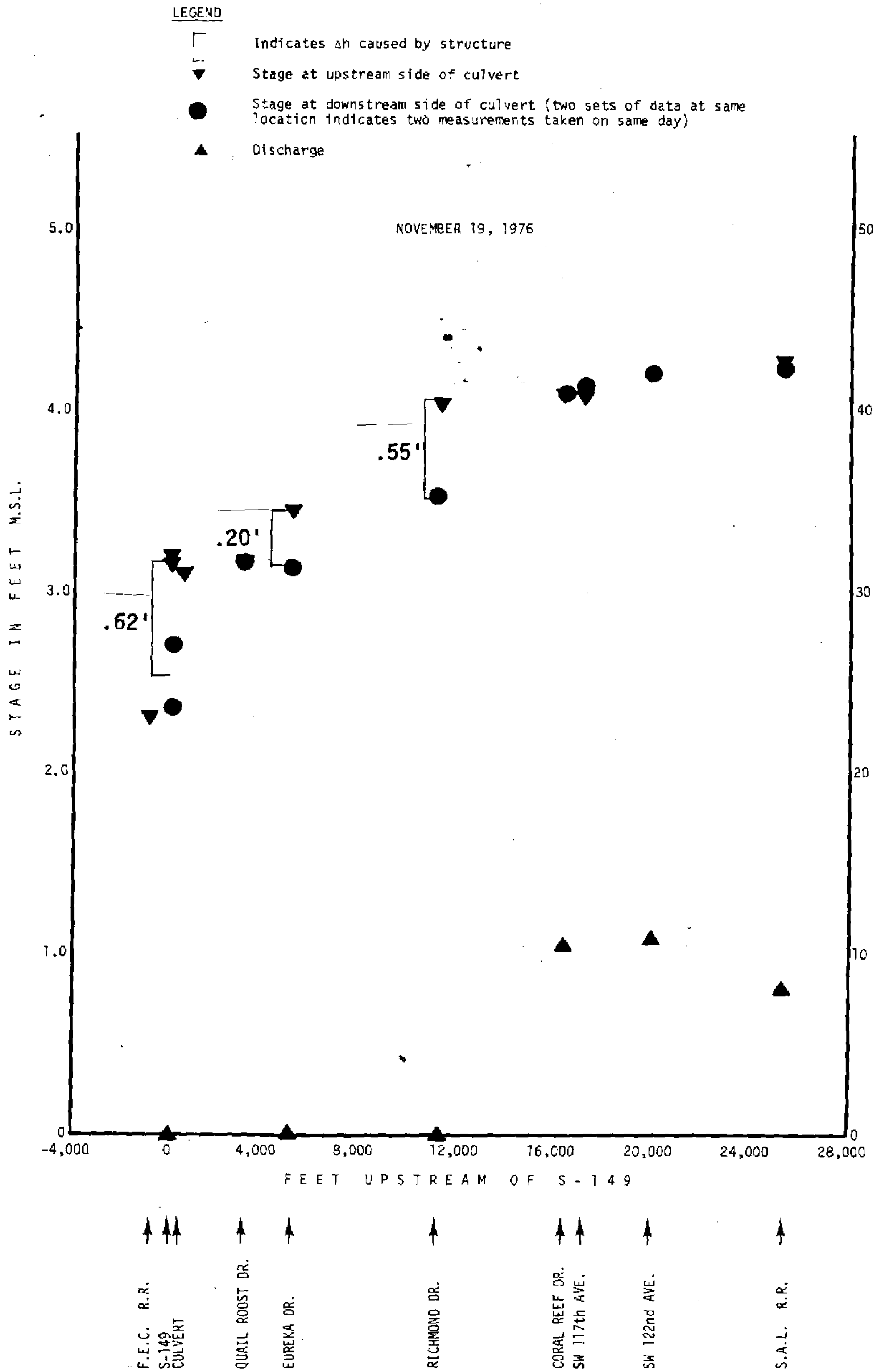


Figure 19. Stage and discharge profiles on November 19, 1976.

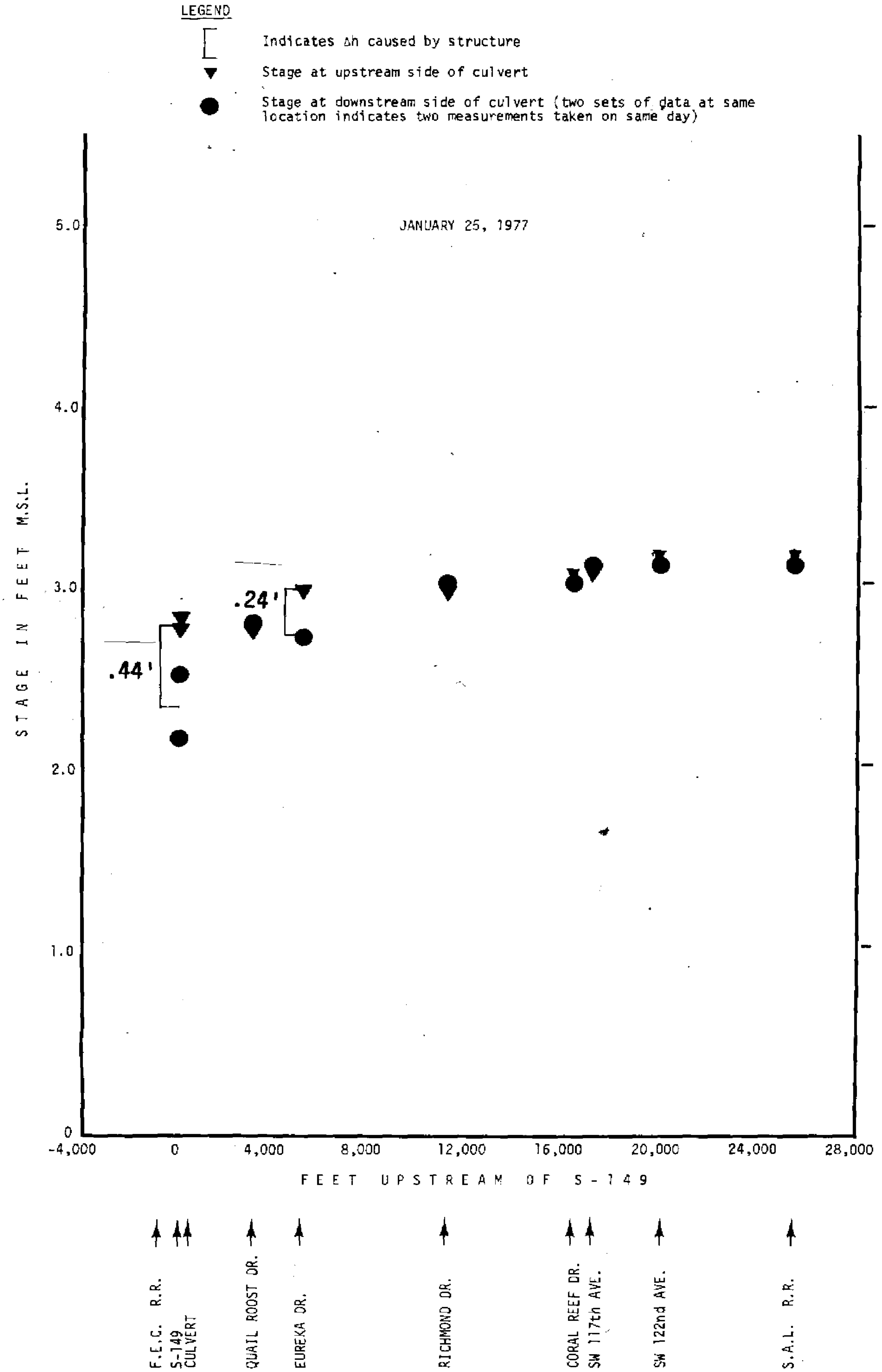


Figure 21. Stage and discharge profiles on January 25, 1977.

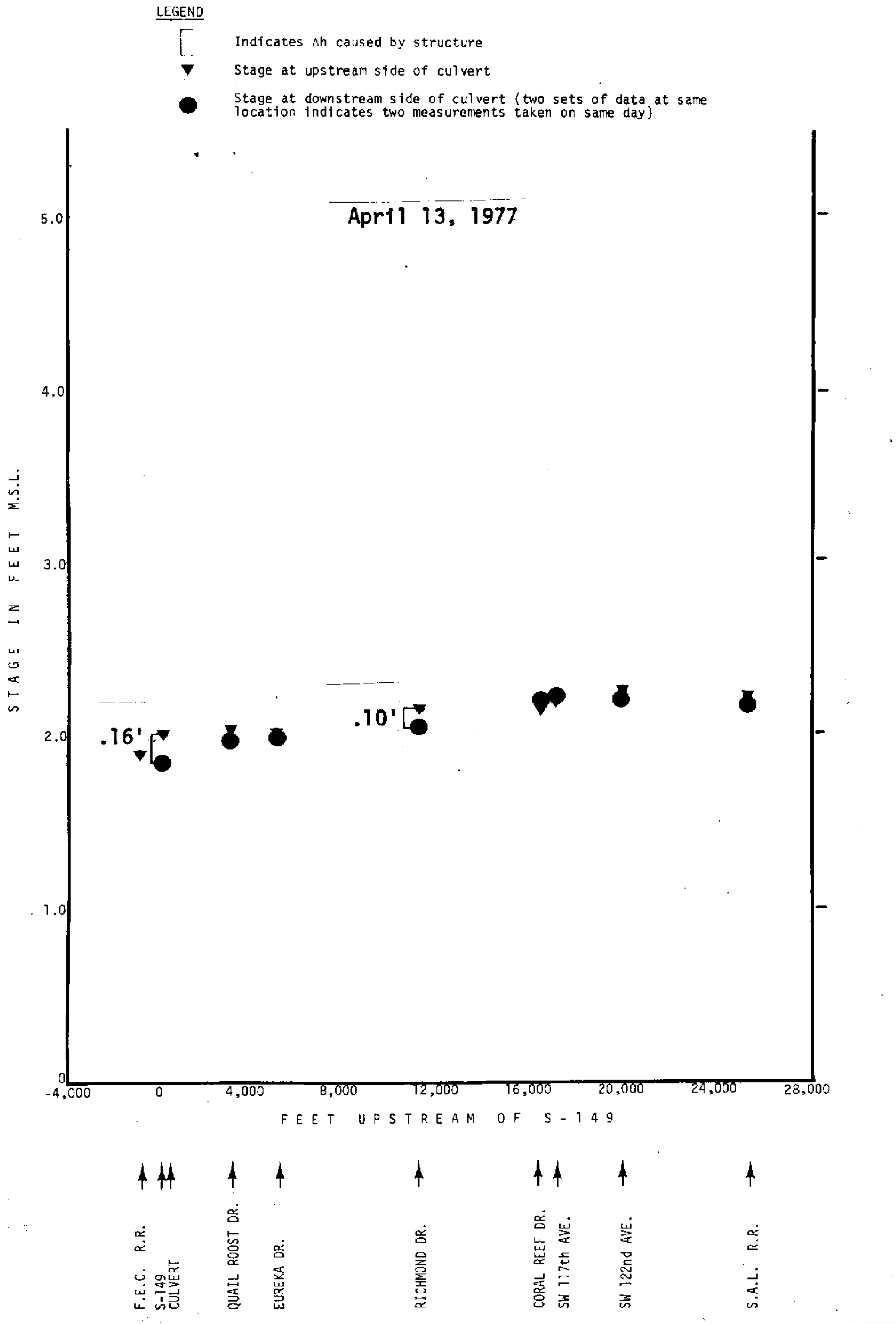


Figure 23. Stage and discharge profiles on April 13, 1977.

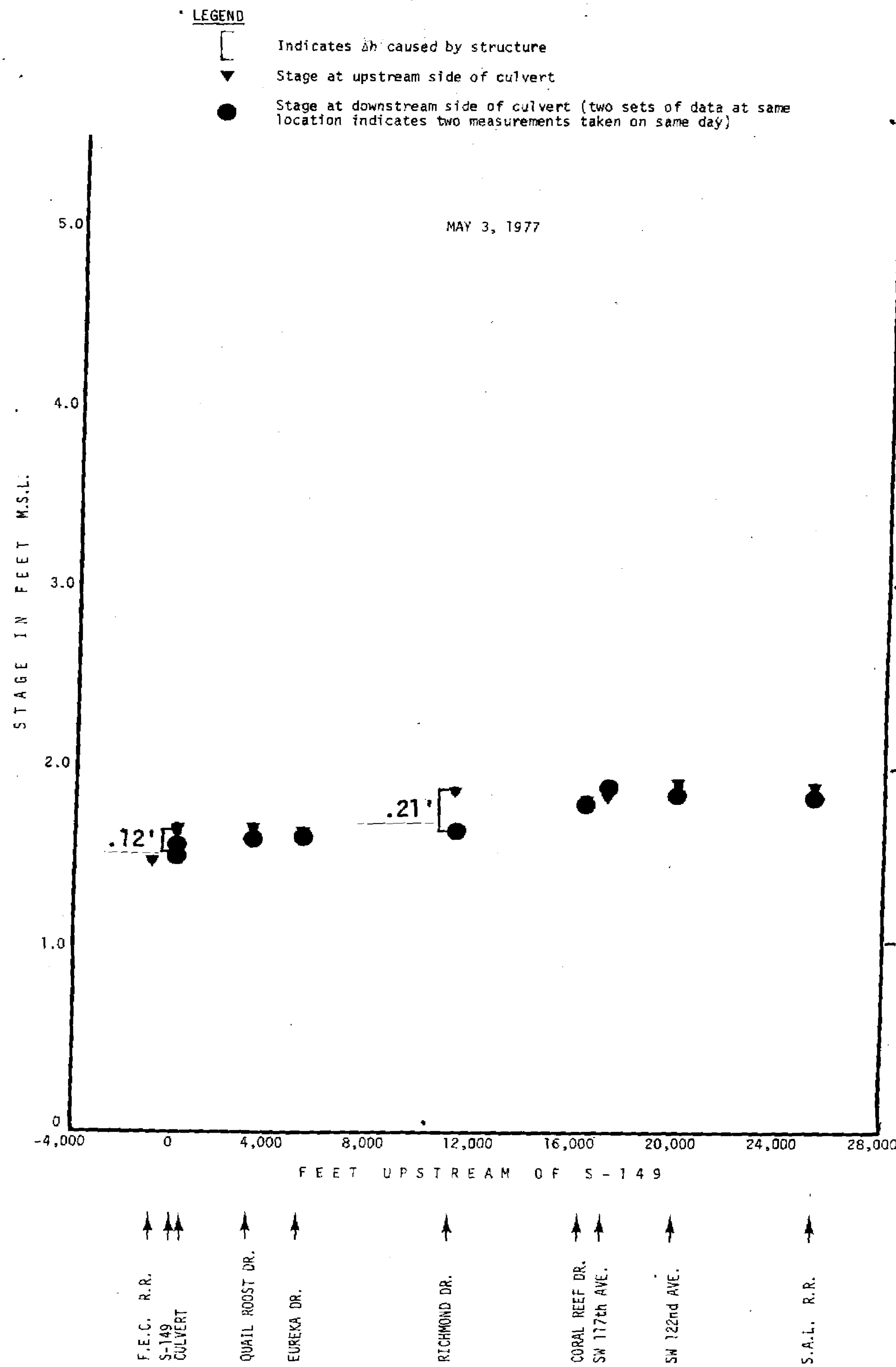


Figure 24. Stage and discharge profiles on May 3, 1977.

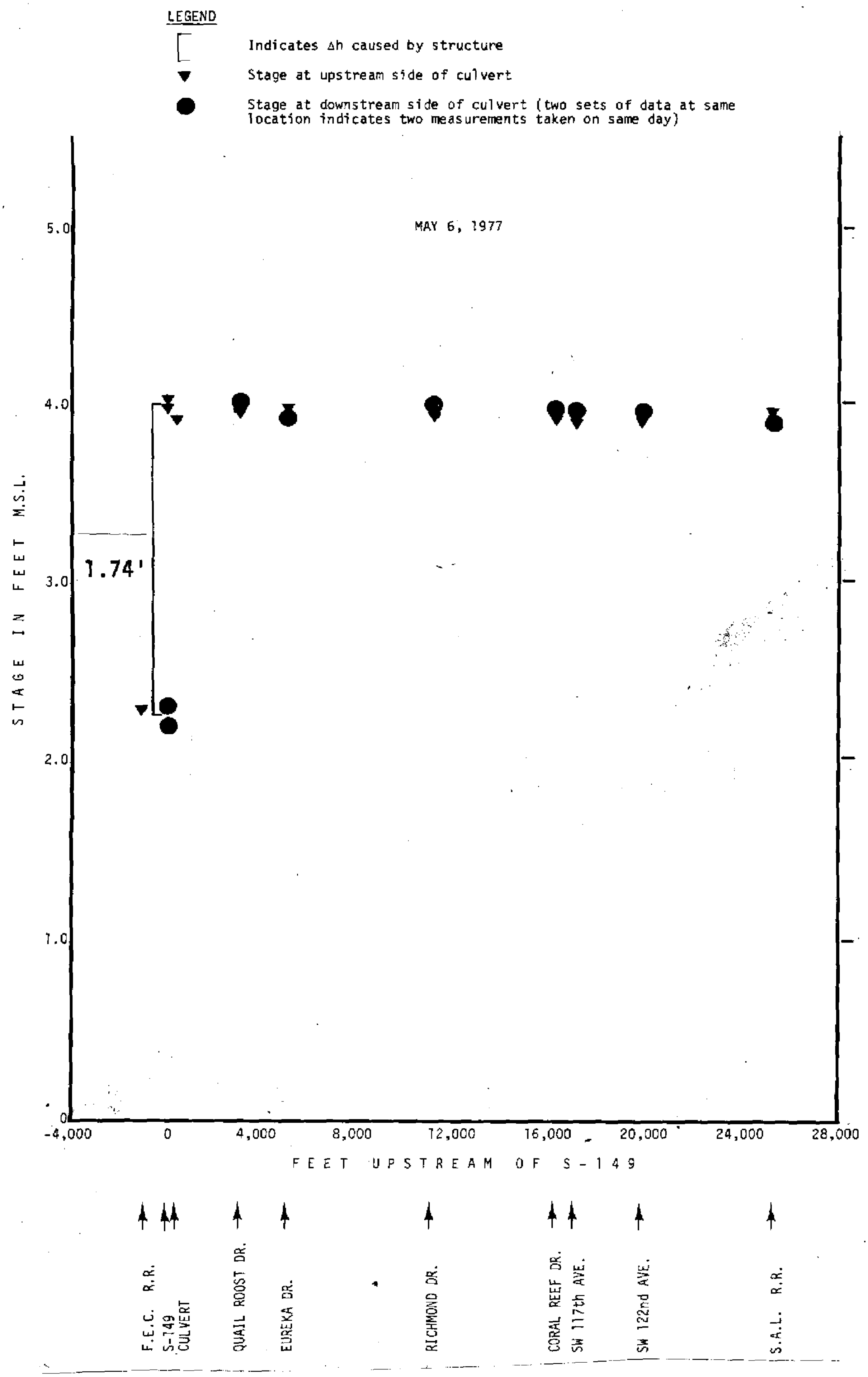


Figure 25. Stage and discharge profiles on May 6, 1977.

In the isolated reach between SW 122nd Ave. and the S.A.L. R.R., the gradient was observed to be occasionally reversed, due to a slight but consistent error in measurement. Eastward flow was observed at this site on all occasions except May 6, 1977. Factors that influence flow directions in this area, such as manipulation of the rock pit water levels and local rainfall distribution, have not been documented.

Figures 18 to 22 show a reversed hydraulic gradient between Eureka Drive and S-149 when the Eureka Drive structure was in operation. The reason for this is uncertain, but among the possible reasons are: simple measurement error, error caused by wind-induced waves on the water surface, concentration of water at the eastern end by wind action, error introduced in surveying the measurement points, or the existence of an unknown groundwater sink (pumping well) near Eureka Drive. The maximum reversed gradient is about 0.10 feet per mile, and is not considered to detract from the overall quality of data.

With few exceptions, hydraulic gradients in the canal vary with the stage. At relatively low stages, when a near equilibrium regime has been established, the gradient approaches zero. The maximum hydraulic gradient observed was about 0.26 feet per mile.

DISCHARGE PROFILES

Discharge measurements were made at eight sites along the length of the canal on nine occasions during the course of the study (Appendix A, Figures 11 to 17, 19 and 20). Measurements were made by both dye velocity and flow meter techniques.

The limited data available indicate that when flow in the canal is unrestricted by the temporary structures (Figures 11 and 12), and canal stages are less than 3 feet msl, it (flow) reaches a maximum near Richmond Drive (data are not available on unrestricted flow profiles when stages exceed 3 feet msl or flows exceed 17 cfs). Upstream of Richmond Drive where groundwater levels exceed canal levels, groundwater enters the canal at a decreasing rate toward Richmond Drive. Downstream of Richmond Drive, the canal loses water to the Biscayne aquifer. In this reach, S-149 stops canal flow with the result that canal levels exceed groundwater levels and canal water re-enters the aquifer. As indicated by the high slope on the discharge profile (Figures 11 and 12), the leakage rate from the canal reaches a maximum (per unit canal length) near Quail Roost Drive.

Water table contour maps in this area indicate that groundwater flows in a direction slightly south of east (Figures 3 and 4). In the north-south oriented reach of C-1N upstream from Eureka Drive, when the canal stage ranges from 2.5 to 3.0 ft. msl, the discharge profile shows a flattening tendency. This tendency is due to the orientation of the canal and groundwater contour lines, as the contour lines intersect the canal at a very slight angle. This situation results in minimal exchange of water between the canal and aquifer, because of the small head differences between the canal and the water table.

The significance of a reach near Richmond Drive is that it is pivotal with regard to the direction of interflow between the aquifer and the canal. At lower stages, as during the dry season, Richmond Drive appears to mark the point along the canal where groundwater inflows cease and canal outflows (seepage) begin. The Richmond Drive site is thus the point of highest canal flows during the dry season. The degree to which the fine sand lense beneath Richmond Drive affects this phenomenon is uncertain. The 1970-71 District study appears to support the above conclusions.

Generally, discharge measurements made when the temporary structures were operational indicate zero discharge at the structures. Two exceptions occurred at Quail Roost Drive on March 30 and August 23, 1976, when the

The performance of each temporary structure is considered proportional to the Δh maintained by that structure. A high Δh would thus indicate high performance of a particular structure. In the data analysis, Δh maintained by a temporary structure was compared to the Δh maintained at S-149 at the same time. While there are three distinct measures of performance possible by the Δh comparison, a direct comparison of the mean Δh values from a sample of Δh values observed over any period of days provides a good example. If the mean Δh at a temporary structure is greater than the mean Δh at S-149 over a defined period, then the temporary structure is concluded to be the more effective structure.

During the course of the field study (Figure 9), individual temporary structures and combinations of temporary structures were in operation during four well-defined periods. Each period was isolated for a Δh comparison. The structures compared in these four periods, in chronological order, were those at Quail Roost Drive and Eureka Drive vs S-149, Eureka Drive and Richmond Drive vs S-149, Eureka Drive alone vs S-149, and Richmond Drive alone vs S-149. Each of these four combinations was analyzed by fitting a regression line through the coordinated Δh values (Appendix B). The regression lines alone for the Δh comparisons are shown on Figure 26. The regression coefficients, the correlation coefficients and other selected data are given in Table 2.

The three measures of comparative performance aluded to above, are the mean Δh values of each variable (structure), and the slope and intercept of the line of regression. The range of the Δh values is also of considerable importance in the performance analysis.

In the first data collection period, the structures at Quail Roost Drive and Eureka Drive were simultaneously compared with S-149 as depicted on Figure 26, lines No. 1 and 2. The range of Δh in this period was good and a high correlation coefficient resulted (Table 2). The mean Δh at the Quail Roost

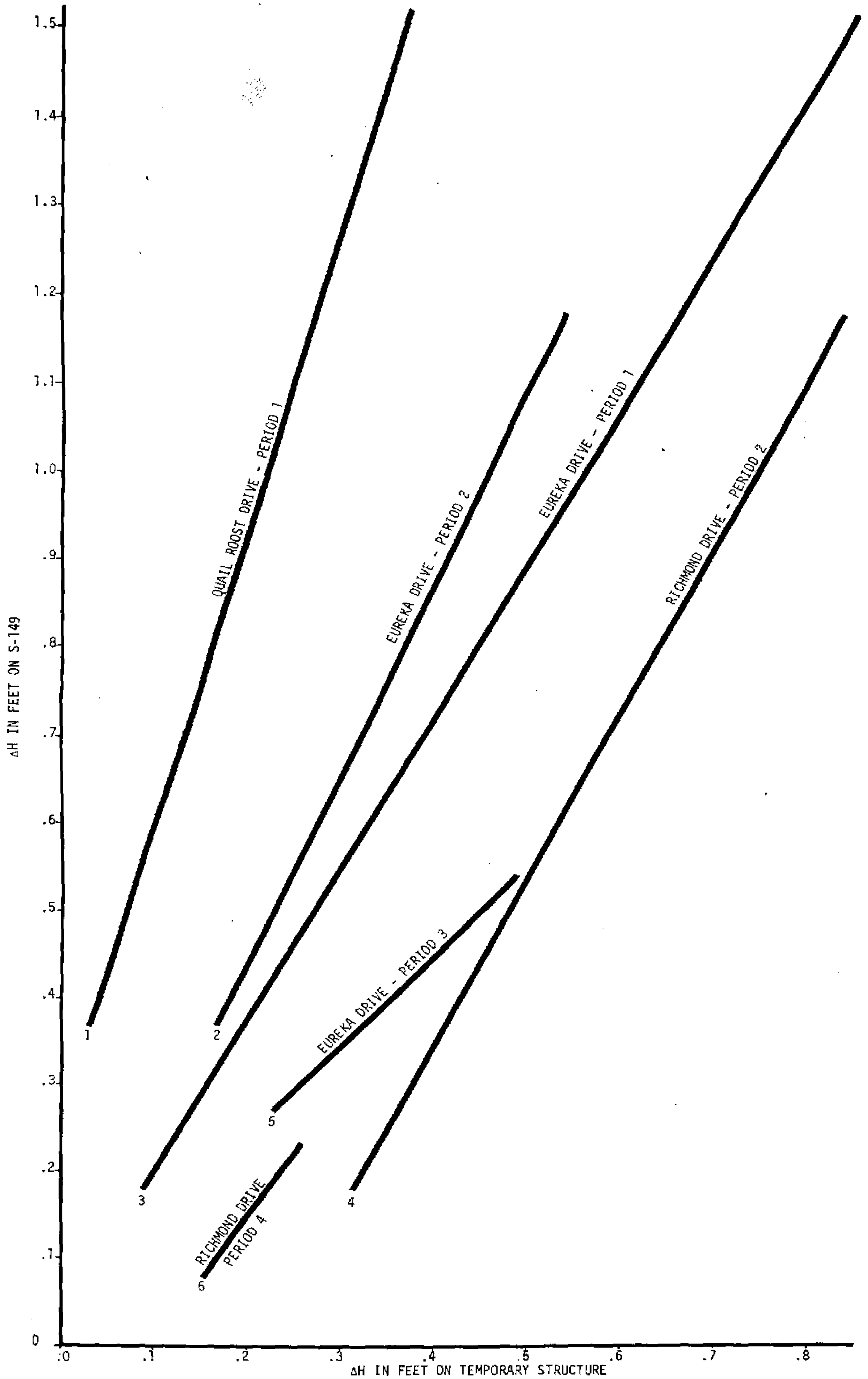


Figure 26. Regression lines formulated from Δh data during four collection periods. Relationships shown are Δh at S-149 vs. Δh at Quail

site, the Eureka site, and at S-149, were 0.175, 0.462, and 0.866 respectively, indicating a performance order from high to low of S-149, Eureka and Quail Roost. The slopes and intercepts of the regression lines confirm these findings. A slope of 3.373 for the Quail Roost site indicates that for each unit of Δh at the Quail Roost site, 3.373 units of Δh correspond at S-149. A slope of less than one would indicate that the temporary structure site performs better than S-149. The regression line intercept of +0.270 for the Quail Roost site indicates that there must be a Δh of 0.27 feet at S-149 before a Δh at Quail Roost develops, signifying that S-149 performs better than Quail Roost Drive. A negative intercept would indicate that the temporary structure site performs better than S-149. An analysis of the field data was conducted in a similar fashion for the remaining three data collection periods. A summary of results is presented in Table 3.

As indicated in Table 3, the overall order of performance from high to low for the four sites tested is S-149, Richmond Drive, Eureka Drive, and Quail Roost Drive. There are three minor inconsistencies in the basis for this ranking, which may have resulted from an insufficient range in Δh data during period 4, and the fact that the upstream structure of any pair will have a certain advantage (with regard to holding a high Δh) over the downstream structure as in periods 1 and 2.

When only the Richmond Drive and the S-149 structures were operational (in data collection period 4), the range in Δh values was limited. The data were collected during the dry season when little rainfall occurred, restricting the stage and Δh range and thereby yielding a regression line, the slope and mean value of which may not be indicative of the Δh relationship at higher stages. The correlation coefficient for period 4 is 0.58. This coefficient is exemplary of a scattering of points, due in this instance to the low Δh range. It is noted that Richmond Drive performs better than S-149 on the basis

Data Collection Period	Order of Rank	RANKING OF SITE PERFORMANCE AS DETERMINED ON THE BASIS OF THE MEAN, SLOPE AND INTERCEPT		
		Mean Δh	Slope of the Regression Line	Intercept of the Regression Line
1.	1st	S-149	S-149	S-149
	2nd	Eureka Dr.	Eureka Dr.	Eureka Dr.
	3rd	Quail Roost Dr.	Quail Roost Dr.	Quail Roost Dr.
2.	1st	S-149	S-149	Richmond Dr.
	2nd	Richmond Dr.	Richmond Dr.	Eureka Dr.
	3rd	Eureka Dr.	Eureka Dr.	S-149
3.	1st	S-149	S-149	S-149
	2nd	Eureka Dr.	Eureka Dr.	Eureka Dr.
4.	1st	Richmond Dr.	S-149	Richmond Dr.
	2nd	S-149	Richmond Dr.	S-149

Table 3. Summary of relative performances of structure sites.

of the intercept of the regression line. This is fortuitous as it is especially desirable to maximize the Δh at Richmond Drive during the dry season.

The fact that the upstream structure affects those downstream somewhat weakens the value of the Δh analyses. This is because the upstream structure blocks the higher flow in the canal, having received influent seepage from a larger portion of the basin. Flow blocked by the downstream structure is only from influent seepage in the relatively small portion of the basin between the two structures. To determine the quantitative significance of this structure interference would be a major effort, requiring additional data and probably computer modeling. The authors believe that the conclusion as to the performance ranking of the sites is basically correct.

EXPECTED PERFORMANCE OF AN ADDITIONAL STRUCTURE AT RICHMOND DRIVE

If it is assumed that the regression line for period 4 depicted on Figure 26 is correct, it is seen that the placement of an additional structure at Richmond Drive would create a Δh of at least 0.23 feet during the dry season when the Δh on S-149 is 0.20 feet. This would increase the dry season storage in the 6 square mile basin upstream from Richmond Drive by about 180 acre-feet. Data collected during period 2 indicates that a structure at Richmond Drive would maintain a Δh of 0.32 feet when the Δh at S-149 is 0.20 feet. The difference in performance of the structure at Richmond Drive during periods 2 and 4 is probably due to a difference in the established drainage regimes as well as a possible difference in leakage through the Richmond Drive structure.

Emplacement of structures at both Eureka Drive and Richmond Drive would increase storage by about 360 acre-feet; this is because Eureka Drive, although not capable of maintaining a Δh as great as Richmond Drive, controls a larger portion of the upstream basin.

Increasing groundwater levels in the basin above any additional structure will have the effect of decreasing flood protection because of the decrease in the groundwater storage capacity. If we assume that a structure at Richmond Drive could raise upstream groundwater levels 2.0 ft. above the present average October water levels as an example of an antecedent condition to a storm event, then a rainfall routing of the 10-year, 5-day event at Richmond Drive would be as follows:

DAY	RAINFALL (INCHES)	GROUND STORAGE (INCHES)	GROUNDWATER LEVEL (FT. MSL.)	RUNOFF (INCHES)
0	0.00	0.00	6.50	0.00
1	0.60	0.60	6.75	0.00
2	0.75	0.75	7.05	0.00
3	1.05	1.05	7.50	0.00
4	2.05	2.05	8.35	0.00
5	7.75	3.96	10.00	3.79

This amount of runoff would create a flow of about 150 cfs at Richmond Drive and 170 cfs at S-149. While flood protection upstream of the Richmond Drive structure would decrease to a level still in excess of the design event, flood protection downstream of Richmond Drive would remain or increase to above the 100-year, 5-day event level. This situation is propitious as the basin is predominantly urbanized in its lower half, as compared to semi-rural in the headwaters.

Among the types of structures that are considered near Richmond Drive are an erodable plug (a simple backfilling of the channel) and a fixed-crest weir. Because of the extremely high conveyance capacity of the aquifer, and the low frequency of demand for full channel capacity, the necessity of having an adjustable gate is reduced.

CONCLUSIONS

(1) The C-1N canal is entrenched in the upper part of the Biscayne aquifer which has an estimated transmissivity ranging from 9 mgd/ft. (million gallons per day per foot) in the west of the basin, to 4 mgd/ft. in the east. The canal seepage problem is aggravated by the high vertical permeability of the Miami Oolite formation directly underlying most of the canal. Borings locate a low permeability silt and sand lense having its maximum vertical extent at the Richmond Drive intersection with the canal. This lense pinches out to the south between Eureka Drive and Quail Roost Drive.

(2) Nearly all flow in the canal is derived from the aquifer rather than surface sources. The rainfall infiltration capacity of the ground surface is sufficiently high that the agricultural land users have not developed secondary drainage systems to C-1N.

(3) Because of the unforeseen and uncontrollable transfer of groundwater via the canals in this region, which modified the antecedent design conditions, the actual flood protection is greatly in excess of that intended by the original design. The canal (C-1N) actually provides protection for the 100-year, 5-day storm (approximately) as opposed to the 10-year, 5-day storm for which it was designed. Construction of the canal system in this area has been the major factor contributing to a decrease in average October groundwater levels of nearly two feet below pre-Project levels.

(4) The cumulative canal flow during the one year study period at Richmond Drive is estimated to be 400 million cubic feet (approximately 29 inches over the upstream basin), practically all of which reenters the aquifer in the reach of the canal between Richmond Drive and S-149.

(5) A District study conducted between 1968 and 1972 determined that seepage losses are regional in extent, and that seepage from C-1N takes place upstream of S-149 to at least Quail Roost Drive. The structure (S-149) does not leak because of improper construction or because of fracturing of the underlying rock during construction.

(6) A District study conducted at Richmond Drive and C-1N in 1970 and 1971, suggests that the canal receives inflow from the aquifer at this site, when the canal stage is above 3.6 feet msl., whereas below this stage the canal loses water to the aquifer.

(7) Alternatives considered for the amelioration of canal seepage losses included:

    (a) Flexible plastic liners (rejected because of anchoring problems).

    (b) Bentonite sealing (rejected because of uncertainties in achieving a successful seal considering other more favorable alternatives).

    (c) Grout curtains (rejected because of the low probability of achieving a reasonably water tight barrier at considerable expense).

    (d) An additional channel structure (considered the most favorable alternative). A structure at Richmond Drive was considered most desirable because:

        (1) Discharge profiles of C-1N indicate that near maximum canal discharge occurs at Richmond Drive, which lies approximately at mid-basin.

        (2) The channel at Richmond Drive is straight, both upstream and downstream, for approximately 1 mile. This configuration eliminates the possibility of flow bypassing a bend in the channel by flow through the aquifer such as could occur at Eureka Drive.

        (3) Hydrogeologic conditions at Richmond Drive are highly favorable

because of an underlying lense of fine sand. A structure at this site would tend to minimize seepage loss and increase upstream stages.

(4) A structure at Richmond Drive would maintain flood protection for the downstream portion of the basin at, or in excess of, present levels. Increased stages upstream will provide flood protection no less than in the original design specifications.

(5) A structure at Richmond Drive would not significantly decrease maintenance accessibility to the canal. Likewise, the structure itself would be readily accessible by heavy equipment.

(6) A structure at Richmond Drive would increase the groundwater level by at least 0.23 feet during the dry season when the corresponding differential head at S-149 is 0.20 feet. Thus, the degree of effectiveness of a Richmond Drive structure could be greater than that provided by S-149.

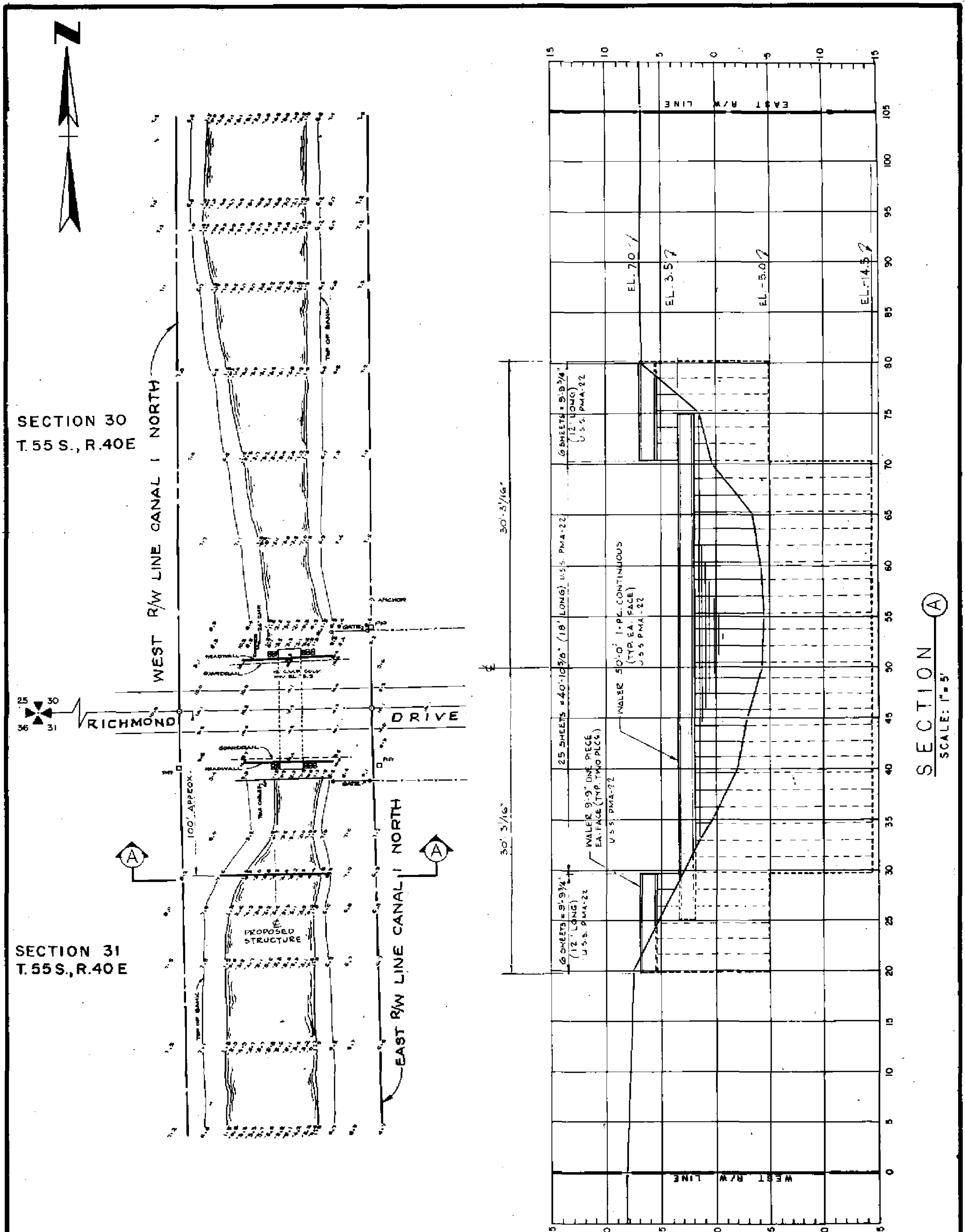
(e) The no-action alternative. This alternative is rejected as a positive step in the improvement of the drainage regime because it is desirable to alter the regime to yield measureable and near-term benefits. Natural plugging of the canal bottom will continue to reduce seepage loss in combination with the chosen alternative.

RECOMMENDATION

As a result of the investigation and conclusions above, and consideration of all feasible configurations for an additional channel structure, it is recommended that a steel sheet pile fixed-crest weir be constructed 100 feet south of the center line of Richmond Drive. The weir dimensions, elevations, and location are shown on Figure 27.

With a crest elevation of 3.5 feet msl, the weir will be submerged at the design water elevation of 6.3 feet msl; however, with a crest length of 40.89 feet, it will pass the design flow (230 cfs) with a downstream water elevation of 6.3 feet msl and an upstream water elevation of 6.45 feet msl.

Estimated cost of the structure using SFWMD forces for the construction is $20,000.



SECTION 30
T. 55 S., R.40E
WEST R/W LINE CANAL 1 NORTH
RICHMOND
DRIVE
100' APPROX.
PROPOSED STRUCTURE
SECTION 31
T. 55 S., R.40 E
EAST R/W LINE CANAL 1 NORTH
EAST R/W LINE
WEST R/W LINE
EL. 7.0
EL. 3.5
EL. -5.0
EL. -14.5
6 SHEETS = 9'-9 3/4" (12' LONG) U.S.S. PMA-22
25 SHEETS = 40'-10 5/8" (18' LONG) U.S.S. PMA-22
WALER 50'-0" 1-PL. CONTINUOUS (TYP. EA. FACE) U.S.S PMA-22
WALER 9'-9" ONE PIECE EA. FACE (TYP. TWO PLCS.) U.S.S. PMA-22
30'-3/16"
SECTION A
SCALE: 1"=5'

Figure 27

APPENDIX A

STAGE AND DISCHARGE DATA
COLLECTED AT TEN SITES ON C-1N.

		F.E.C. R.R. *1,000	S-149	Culvert Upstream of S-149 *240	Quail Roost Drive *2,990	Eureka Drive *4,990	Richmond Drive *10,990	Coral Reef Drive *16,340	S.W. 117th Avenue *17,140	S.W. 122nd Avenue *19,940	S.C.L. R.R. *25,240
February 17, 1976	Time		1110 1449								
	Q		0		2.12 **1.92	6.57 **6.57	9.43 *9.43	9.4 10.2	6.83		
	Up Stream		2.48 2.47		2.49	2.46	2.48	2.51	2.50		
	Down Stream		2.24 2.27								
	Dam		0.24 0.20								
March 4, 1976	Time	0942	1245	1250	1305	1325	1335	1405	1420	1435	1510
	Q		0		**9.70	**15.95	**17.63	**12.82	**9.67	**11.67	**2.85
	Up Stream	2.48	2.84	2.78	2.80	2.79	2.85	2.85	2.85	2.98	2.98
	Down Stream		2.52		2.82	2.79	2.83	2.84	2.88	3.00	2.96
	Dam		0.32								
March 30, 1976	Time		1515		1046 1505						1340
	Q				**10.81 0	**10.95	**10.25	**8.24	**6.78	**7.95	1.13
	Up Stream		2.22 2.13		2.16 2.17	2:18	2.23	2.27	2.39		2.37
	Down Stream		1.93 1.97		2.21 2.09	2.19	2.24	2.28	2.30	2.39	2.35
	Dam		0.30 0.16		-- 0.08						

*Distance in feet upstream of S-149
**DYE Velocity measurements

		F.E.C. R.R. *1,000	S-149	Culvert Upstream of S-149 *240	Quail Roost Drive *2,990	Eureka Drive *4,990	Richmond Drive *10,990	Coral Reef Drive *16,340	S.W. 117th Avenue *17,140	S.W. 122nd Avenue *19,940	S.C.L. R.R. *25,240
April 7, 1976	Time		1656		1640						
	Q				0						
	Up Stream		2.03		2.08						
	Down Stream		1.99		2.04						
	Dam		0.04		0.04						
April 8, 1976	Time		0809 1445		0916 1435	1411					
	Q				0						
	Up Stream		2.13 2.15		2.10 2.06	2.12					
	Down Stream		2.04 2.03		2.07 2.06	2.07					
	Dam		0.09 0.12		0.03 0.02	0.075 (meas.)					
May 18, 1976	Time	1055	1050	1035	1000	1110	1120	1336	1351	1422	1430
	Q				0	0	**3.96	**3.94	**5.09	**3.28	<5
	Up Stream	2.24	2.31	2.29	2.31	2.27	2.40	2.43	2.44	2.46	2.41
	Down Stream		2.27		2.33	2.33	2.42	2.44	2.46	2.46	2.41
	Dam		0.04		0.03 (meas.)	.075 (meas.)					

*Distance in feet upstream of S-149
**DYE Velocity measurements

Date		F.E.C. R.R. *1,000	S-149	Culvert Upstream of S-149 *240	Quail Roost Drive *2,990	Eureka Drive *4,990	Richmond Drive *10,990	Coral Reef Drive *16,340	S.W. 117th Avenue *17,140	S.W. 122nd Avenue *19,940	S.C.L. R.R. *25,240
January 25, 1977	Time		1100 1245								
	Q										
	Up Stream		2.84 2.77		2.78	3.00	3.00	3.08	3.09	3.19	3.20
	Down Stream		2.54 2.19		2.79	2.76	3.02	3.06	3.10	3.18	3.16
	Dam		0.30 0.58			0.24					
February 17, 1977	Time	1401	1032 22.4 °C 1350	1340	1330 20.1 °C	1310 20.5 °C	1250 18.8 °C	1133 18.5 °C	1122 18.8 °C	1103 19.9 °C	1047 18.4 °C
	Q		0				0				
	Up Stream	2.50	2.78 2.77	2.71	2.71	2.84	3.18	3.17	3.16	3.18	3.21
	Down Stream		2.47 2.51		2.73	2.67	2.83	3.17	3.19	3.19	3.19
	Dam		0.31 0.25			0.17	0.35				
March 30, 1977	Time										
	Q										
	Up Stream					2.35					
	Down Stream										
	Dam										

*Distance in feet upstream of S-149
**DYE Velocity measurements

		F.E.C. R.R. *1,000	S-149	Culvert Upstream of S-149 *240	Quail Roost Drive *2,990	Eureka Drive *4,990	Richmond Drive *10,990	Coral Reef Drive *16,340	S.W. 117th Avenue *17,140	S.W. 122nd Avenue *19,940	S.C.L. R.R. *25,240
November 2, 1976	Time & Temp.	1210 26.0°C	1205 26.5°C	1200 25.2°C	1150 25.6°C	1143 24.6°C	1152 24.7°C	1120 24.7°C	1115 24.6°C	1110 24.6°C	1100 24.4°C
	Q										
	Up Stream	2.05	2.95	2.90	2.91	3.48	3.59	3.63	3.69	3.94	3.96
	Down Stream		2.08		2.93	2.92	3.56	3.64	3.67	3.94	3.94
	Dam		0.87			0.56					
November 19, 1976	Time & Temp.	1450 26.1°C	1030 26.3°C 1445	1440	1420 26.7°C	1400 26.4°C	1340 26.7°C	1315 25.3°C	1300 24.4°C	1235 25.5°C	1100 25.0°C
	Q		0 0			0		10.4		10.8	8.00
	Up Stream	2.30	3.16 3.15	3.10	3.12	3.43	4.04	4.08	4.09		4.24
	Down Stream		2.72 2.35		3.12	3.13	3.49	4.08	4.12	4.20	4.21
	Dam		0.44 0.80			0.20	0.55				
January 5, 1977	Time & Temp.		1025 24.2 °C 1600	1445	1415 22.8°C	1400 21.8°C	1315 21.8°C	1300 22.7°C	1230 22.7°C	1140 22.5°C	1045 21.0°C
	Q		0		0		12.8	12.6		9.5	1.40
	Up Stream		2.85 2.85	2.78	2.80	3.06	3.06	3.10	3.11	3.27	3.22
	Down Stream		2.53 2.56		2.81	2.79	3.07	3.11	3.19	3.28	3.15
	Dam		0.32 0.29			0.27					

*Distance in feet upstream of S-149
**DYE Velocity measur[illegible]

		F.E.C. R.R. *1,000	S-149	Culvert Upstream of S-149 *240	Quail Roost Drive *2,990	Eureka Drive *4,990	Richmond Drive *10,990	Coral Reef Drive *16,340	S.W. 117th Avenue *17,140	S.W. 122nd Avenue *19,940	S.C.L. R.R. *25,240
July 8, 1976	Time	1510	1500	1440	1430	1415	1345	1120	1330	1257	1130
	Q		0				14.8	18.1		17.9	14.4
	Up Stream	2.50	3.17	3.17	3.30	3.73	3.88	4.01	4.03	4.18	4.10
	Down Stream		2.57		3.16	3.30	3.88	4.05	4.05	4.18	4.12
	Dam		0.60		0.14	0.43					
August 23, 1976	Time		1120 1510	1440	1420	1400	1345	1325		1310	1215
	Q		0	3.08	23.7	38.9	31.9	33.5	30.8	28.6	16.6
	Up Stream		4.15 4.10	4.03	4.18	4.31	4.53	4.69		4.95	5.01
	Down Stream		2.02 2.07		4.08	4.31	4.54	4.68		4.90	4.98
	Dam		2.13 2.03		0.10	0					
October 8, 1976	Time	1500	1040 1600					1310	1255	1235	1110
	Q		0			0	16.4	19.6		12.0	11.40
	Up Stream	2.13	3.22 3.17		3.22	3.85	3.96	4.07	4.12	4.35	4.41
	Down Stream		2.57 1.92		3.22	3.40	3.97	4.08	4.14	4.35	4.39
	Dam		0.65 1.25		0.00	0.45					

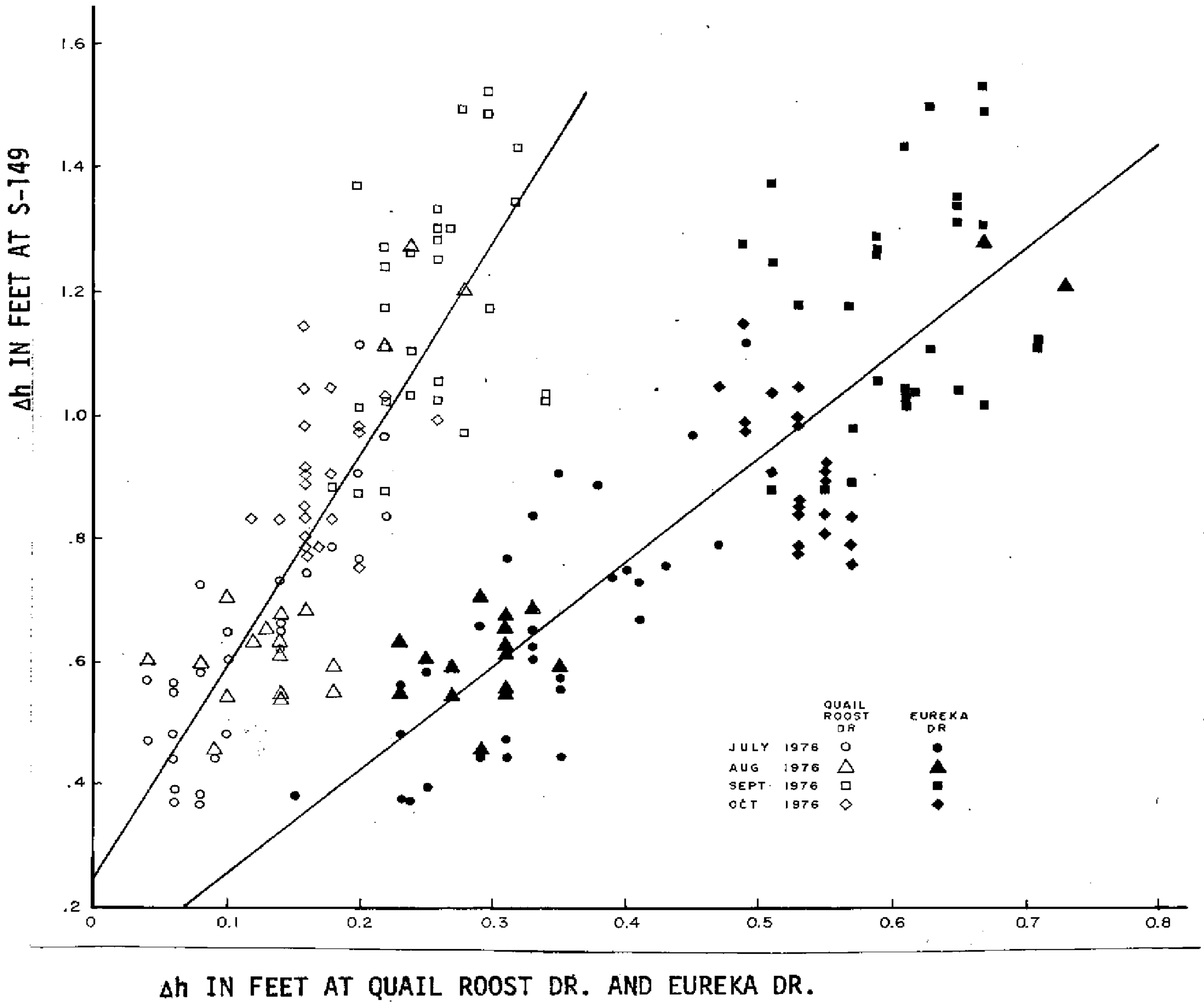
*Distance in feet upstream of S-149
**DYE Velocity measurements

		F.E.C. R.R. *1,000	S-149	Culvert Upstream of S-149 *240	Quail Roost Drive *2,990	Eureka Drive *4,990	Richmond Drive *10,990	Coral Reef Drive *16,340	S.W. 117th Avenue *17,140	S.W. 122nd Avenue *19,940	S.C.L. R.R. *25,240
June 1, 1977	Time		1249								
	Q				13.6	20.5	18.0	21.8		21.5	
	Up Stream		3.34								
	Down Stream		2.22								
	Dam		1.12								
June 7, 1977	Time										
	Q										
	Up Stream		4.38				4.53				
	Down Stream		2.42								
	Dam		1.96								
	Time										
	Q										
	Up Stream										
	Down Stream										
	Dam										

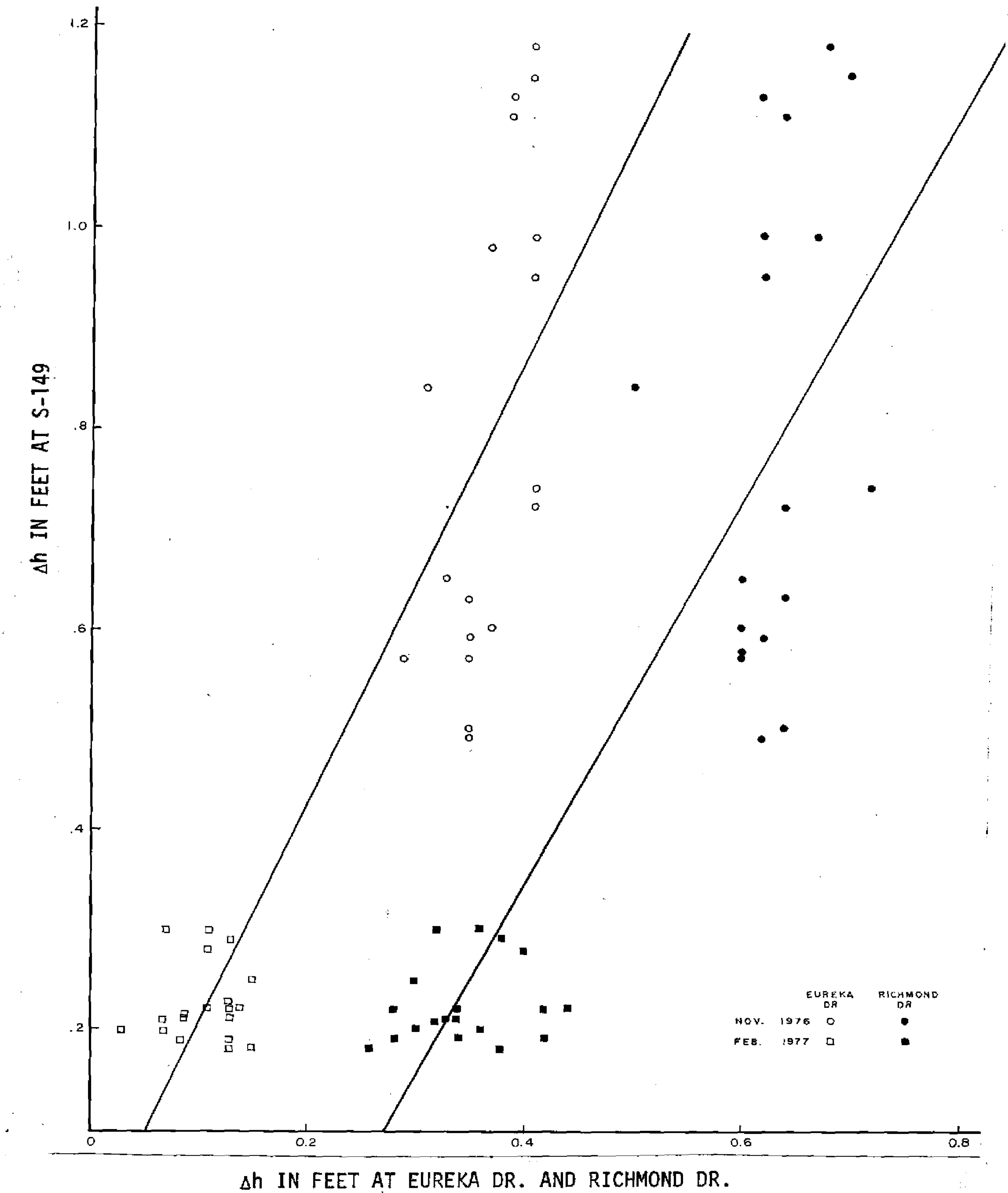
*Distance in feet upstream of S-149
**DYE Velocity measurements

Date		F.E.C. R.R. *1,000	S-149	Culvert Upstream of S-149 *240	Quail Roost Drive *2,990	Eureka Drive *4,990	Richmond Drive *10,990	Coral Reef Drive *16,340	S.W. 117th Avenue *17,140	S.W. 122nd Avenue *19,940	S.C.L. R.R. *25,240
April 13, 1977	Time										
	Q										
	Up Stream	1.90	2.02		2.03	2.02	2.16	2.17	2.21	2.26	2.23
	Down Stream		1.86		2.02	2.02	2.06	2.18	2.21	2.24	2.20
	Dam		0.16				0.10				
May 3, 1977	Time		1100 1240								
	Q										
	Up Stream	1.49	1.68 1.68		1.68	1.66	1.89	1.85	1.88	1.92	1.91
	Down Stream		1.55 1.56		1.66	1.66	1.68	1.85	1.89	1.90	1.89
	Dam		0.13 0.12				0.21				
May 6, 1977	Time	1048	1043 1250	1055	1100	1110	1120 1235	1130	1140	1145	1150
	Q		0 0								
	Up Stream	2.30	4.02 3.97	3.93	3.96	3.96	3.96 3.96	3.94	3.94	3.92	3.95
	Down Stream		2.30 2.20		3.97	3.95	3.98	3.96	3.95	3.93	3.92
	Dam		1.72 1.77				0.02				

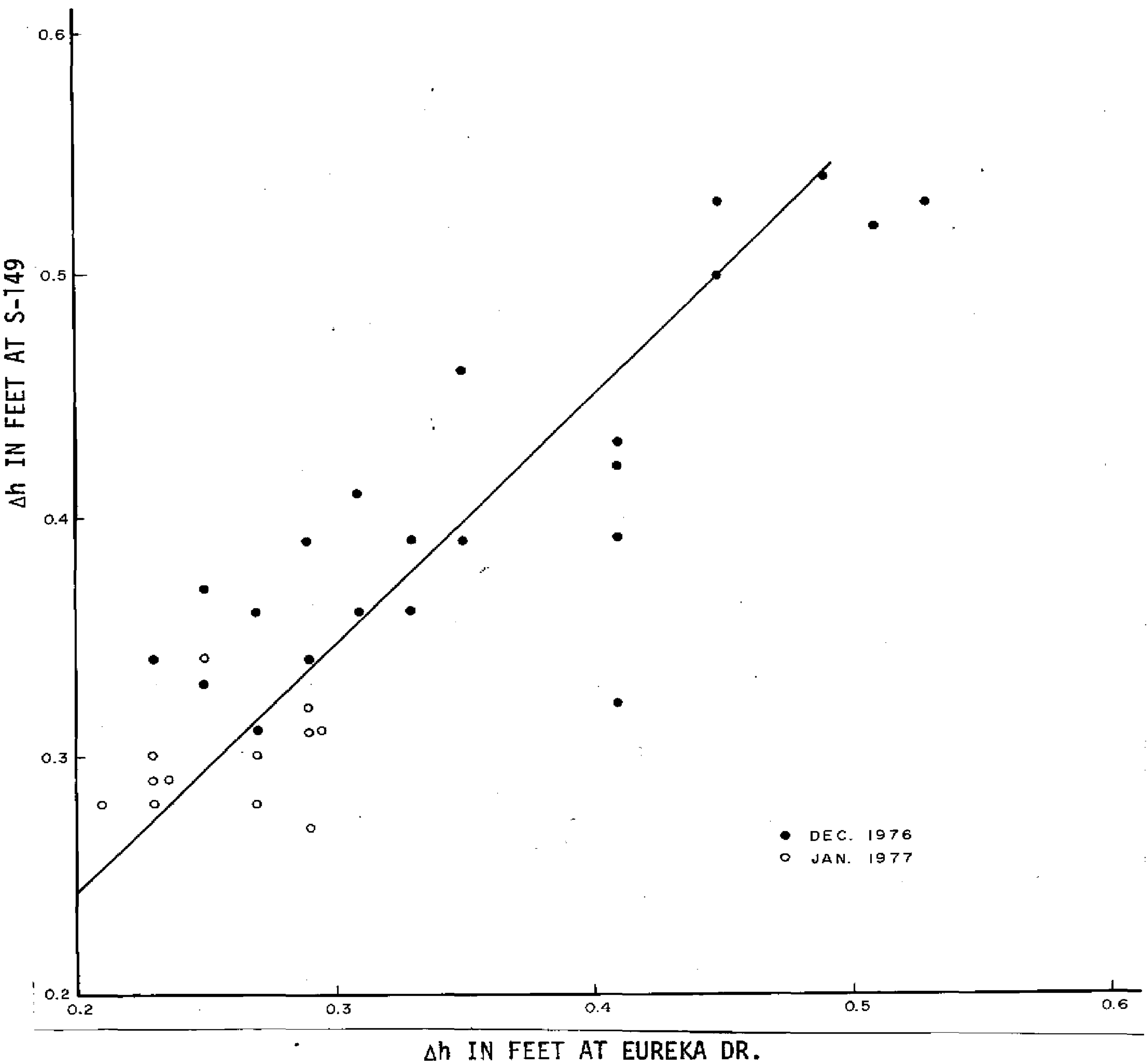
*Distance in feet upstream of S-149
**DYE Velocity measurements



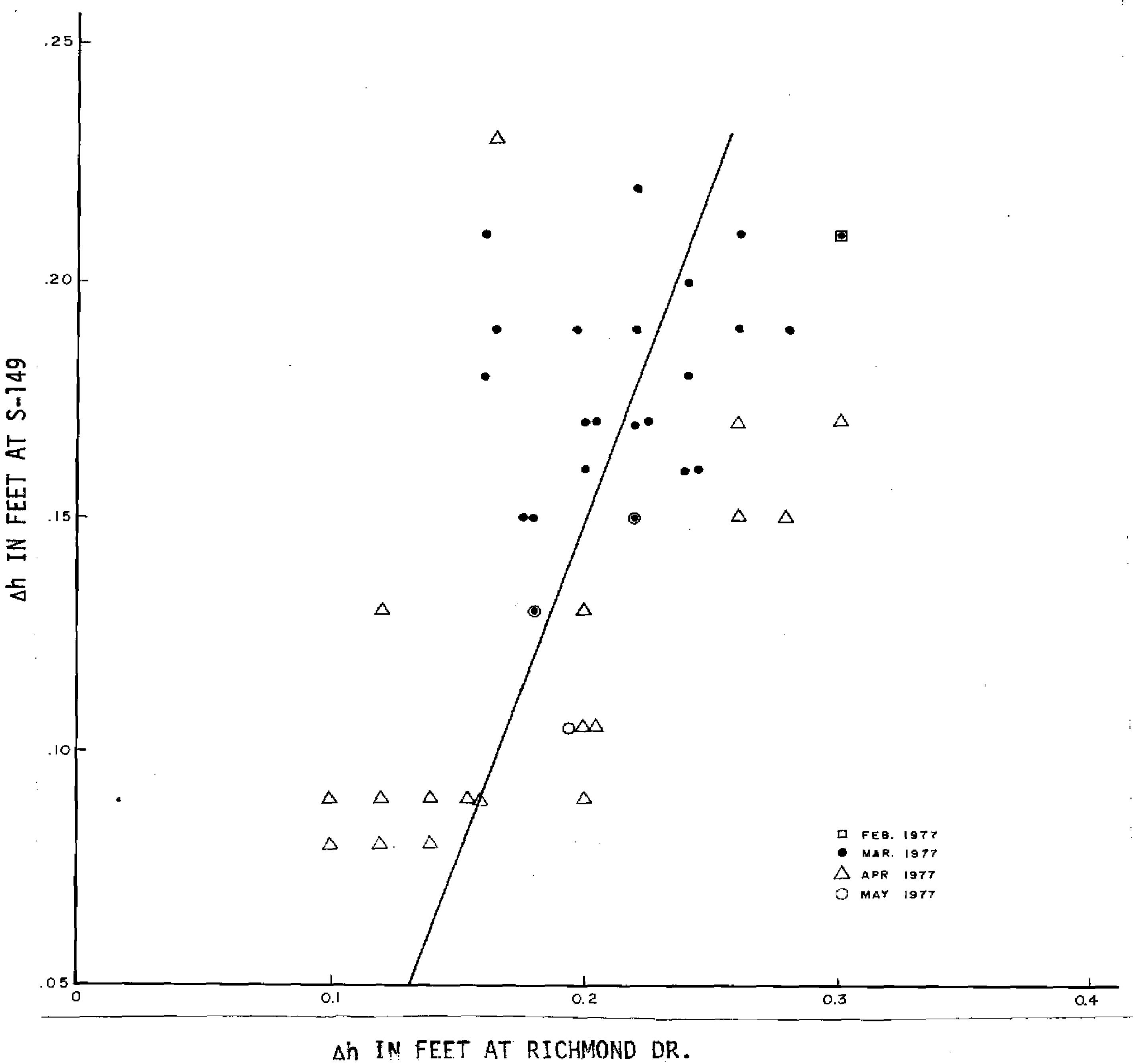
Regression analysis of Δh data during data collection period No. 1.



Regression analysis of Δh data during data collection period No. 2.



Regression analysis of Δh data during data collection period No. 3.



Regression analysis of Δh data during data collection period No. 4