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Division of Water Pollution Control
Research Project 77-12 & 80-22

Installation and Evaluation of Permeable Pavement at Walden Pond State Reservation

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INSTALLATION AND EVALUATION

OF

PERMEABLE PAVEMENT

AT

WALDEN POND STATE RESERVATION

FINAL REPORT

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DIVISION OF WATER POLLUTION CONTROL

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ABSTRACT

Permeable pavements have been proposed for use in controlling surface runoff. A test installation was constructed at the Walden Pond State Reservation in Concord, Massachusetts to determine the feasibility of using permeable pavements in the New England environment. A brief history of the development and use of permeable pavements is provided. Background information, design methodology, and construction techniques are described for the data collection system and the parking area itself. All the following data are presented and discussed: mix gradation; pavement density; pavement, ground, and air temperatures; flow through permeable pavements; water quantity; water quality; and structural stability. This report also includes a survey of major existing permeable pavement installations located in the United States. The conclusions and recommendations cited in this report are based upon the results generated from the testing at Walden Pond.

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CHAPTER 1

INTRODUCTION

1.1 GENERAL

Current practices of land development have a significant effect on the hydrologic characteristics of the areas involved. Existing high density urban areas in many cases approach complete imperviousness, resulting in over 90 percent of the rainfall appearing as runoff and a negligible amount as infiltration (Tourbier, 1978). Suburbanization, characterized by the development of residential dwellings, shopping centers, and industrial parks, also cause a drastic increase in impervious areas with similar results. Recent concern for water resource protection in urban and suburban areas has resulted in a closer look at land development practices (Tourbier and Westmacott, 1974).

An increase in runoff has several consequences. The time of concentration is greatly reduced, which results in a larger peak flow, larger storm drainage facilities, potential flood damage to property, and stream bed and bank erosion. In areas with combined sewers, the large volumes of storm water entering the system often exceed the capacity of the system and are discharged via overflows into a receiving water. The nation wide significance of pollution by storm generated discharges was first identified in 1964 (U.S. Public Health Service, 1964). Many studies have been conducted to determine the composition of combined sewer overflow and to discern what effect this discharge has on the quality of the receiving water. Field and Struzeski (1972) present a substantial list of references dealing with this subject and state "Untreated overflows have proved to be a substantial pollution source in terms of impact on receiving water quality even though the percent of sanitary wastewater lost from the system by overflow is small, on the order of 3-5 % (p. 1393)." At the time an overflow occurs, a large residual sanitary pollution load is discharged over a short period of time. Known as the "first flush", this can be a shock load for the receiving water.

The storm runoff water itself is often a significant contributor to the overflow pollution load (American Public Works Association, 1969; Sartor and Boyd, 1972; Weibel, Anderson, and Woodward, 1964). Storm runoff picks up various contaminants including accumulated debris, animal droppings, eroded soil, tire and vehicular exhaust residue, air pollution fallout, deicing compounds, pesticides, fertilizers, decayed vegetation, heavy metals, and many unknown pollutants. Any reduction in the volume of storm runoff therefore can contribute to the protection of surface water supplies.

Independent of the effects of increased runoff, the lack of infiltration of storm water due to extensive land development is an important consideration. Under natural conditions a significant portion of the storm water would infiltrate the ground surface and eventually replenish ground water supplies. Ground water recharge is vital in order to maintain a constant yield from an aquifer.

Several methods have been investigated for reducing urban and suburban runoff (Tourbier and Westmacott, 1974). Everhart (1973) reported on a case study showing the many benefits to be gained by specifically designing to reduce the volume of storm runoff. One rather innovative approach to reducing storm runoff is the use of an open graded asphalt concrete (OGAC), commonly referred to as a porous pavement. The term permeable pavement is used in this report since it more accurately describes the primary feature of the pavement surface, that is its ability to pass water. By allowing storm water to pass through the pavement surface into the ground, a substantial reduction in runoff is achieved while allowing for ground water recharge. An economic and feasibility study on permeable pavement was performed by the Franklin Institute Research Laboratory (FIRL) in 1972 (Thelen et al., 1972). The following benefits of permeable pavement were cited by FIRL:

1. Eliminates combined sewer overflow

2. Reduces the cost of storm drainage systems

3. Provides for storm water retention
4. Allows for regeneration of ground water supplies
5. Eliminates the need for curbing
6. Improves safety by increasing skid resistance and enhancing pavement marking visibility
7. Allows the use of urban debris in the base course
8. Has comparable maintenance costs with those of conventional pavements
9. Relieves flash flooding
10. Preserves roadside vegetation
11. Preserves natural drainage patterns
12. Provides temperature and storm control if light colored pavement is used
13. Relieves pedestrian inconvenience by removing puddles.

FIRL concluded that permeable pavement is both technically sound and cost-effective for certain applications.

1.2 DEVELOPMENT OF PERMEABLE PAVEMENT

Permeable pavements (or OGAC) were initially developed for use as a wearing course on conventional dense pavements. These wearing courses, originally referred to as open-graded plant mix seal coats (OGPMSC), evolved from the conventional chip seal surface treatment used primarily to seal and maintain aged, but otherwise structurally sound pavements (Smith, Rice, and Spellman, 1974). Substantial use of OGPMSC's occurred in the western states during the late 1940's and early 1950's. Typical seal coats used at this time had a maximum particle size of approximately 3/8 in., a relatively high percentage of asphalt cement, and were placed in thicknesses ranging from 5/8 in. to 3/4 in. (Transportation Research Board, 1978). The excess asphalt cement was added to seal the dense pavement at the pavement-surface course interface. These mixes provided a highly skid resistant surface since water falling on the pavement would infiltrate the permeable surface and drain laterally out of the pavement at the sealed interface. In an attempt to increase the use of open-graded wearing courses, in 1974 the Federal Highway Administration recommended a two step construction process, and devised a more exact design procedure for the surface course, now designated open-graded asphalt friction courses (OGAFC). The existing surface had to be sealed separately with a tack coat, prime coat, or an overlay of dense-graded material, depending on its physical condition.

Since the 1930's 48 states, as well as the District of Columbia and Puerto Rico have constructed OGAFC pavements. Many states have developed their own mixes specific to their locations. California (Lammers, 1973), Virginia (Maupin, 1976), Maine (Lvettich, 1970), North Carolina (Mullen, 1973), and New Mexico (Williams, 1973) are but a few of the states including OGAFC's in their standard specifications. The Federal Aviation

Administration (White, 1976) has also developed such a mix. In addition, OGAC mixes have been investigated for use as a component in subgrade drainage systems (Tayabji and Barenberg, 1975). For an excellent report on the history and current use of OGAFC, see "Open-Graded Friction Courses for Highways," National Cooperative Highway Research Program, Synthesis of Highway Practice 49, Transportation Research Board, 1978.

In 1972 FIRL proposed the use of OGAC as the Main structural element in the pavement system rather than as an overlay. Aggregate gradations specified by the State of California, the Asphalt Institute, and a British Authority were investigated for their hydraulic and structural performance. A gradation specified by the State of California was deemed to be the optimum mix of those investigated. FIRL expressed the urgent need for field studies of permeable pavement to supplement their laboratory work. The first permeable pavement parking area in the United States if not the world was constucted in 1973 at the University of Delaware (Bachtle, 1974). This installation used the California aggregate gradation recommended by FIRL and was never formally monitored but was reported to be performing satisfactorily. The FIRL, under a subcontract to Rice University, concluded a field study at a permeable pavement parking area in The Woodlands, Texas (Hollinger and Haigh, 1976). It was concluded that (1) the physical behavior and durability of OGAC is comparable to that of dense asphalt concrete, (2) water which infiltrates through permeable pavement generally has a lower level of contaminants than runoff from a dense pavement, and (3) maintenance and construction requirements are more stringent for permeable pavement than for dense pavement. Permeable pavement has been utilized in several other parking areas in Delaware, Pennsylvania, and Florida. These sites were not test installations, therefore no report on their performance is available.

The performance of a permeable pavement installation in a climate experiencing severe cold has not previously been determined. Additional studies on the general performance of permeable pavement are required before this alternative design is accepted in engineering practice.

1.3 PROJECT OBJECTIVES

This study was designed as a research and demonstration project. The general objectives of the project were to design, install, monitor, and evaluate the performance of permeable pavements on an existing gravel parking lot (total area approximately 17,000 square yards) at the Walden Pond Reservation administered by the Massachusetts Department of Environmental Management.

Various designs of permeable pavement and monitoring system were installed in the parking lot. The performance of the pavement was monitored and evaluated in terms of:

1. Impact of Massachusetts weather

2. Water quantity

3. Water quality

4. Structural stability

An extensive survey of other existing permeable pavement installations was also conducted in order to enlarge the data base for evaluation.

PART 1 INSTALLATION OF PERMEABLE PAVEMENT

CHAPTER 2

DESIGN

2.1 SITE ANALYSIS

2.1.1 General

The field study site consisted of a 3.5 acre gravel parking area at the Walden Pond Reservation in Concord, Massachusetts, approximately 17 miles northwest of Boston. See Figure 2-1 for a general site location.

The parking area was a series of irregularly shaped connected sections. Figure 2-2 shows the plan of the parking area with the adopted numbering system. Each section had a width of approximately 70 ft, while lengths varied from 150 to 500 ft, except for section B which had both a width and a length of approximately 130 ft. The irregular layout of the area was ideal for separating the various mixes and structural systems utilized in the study.

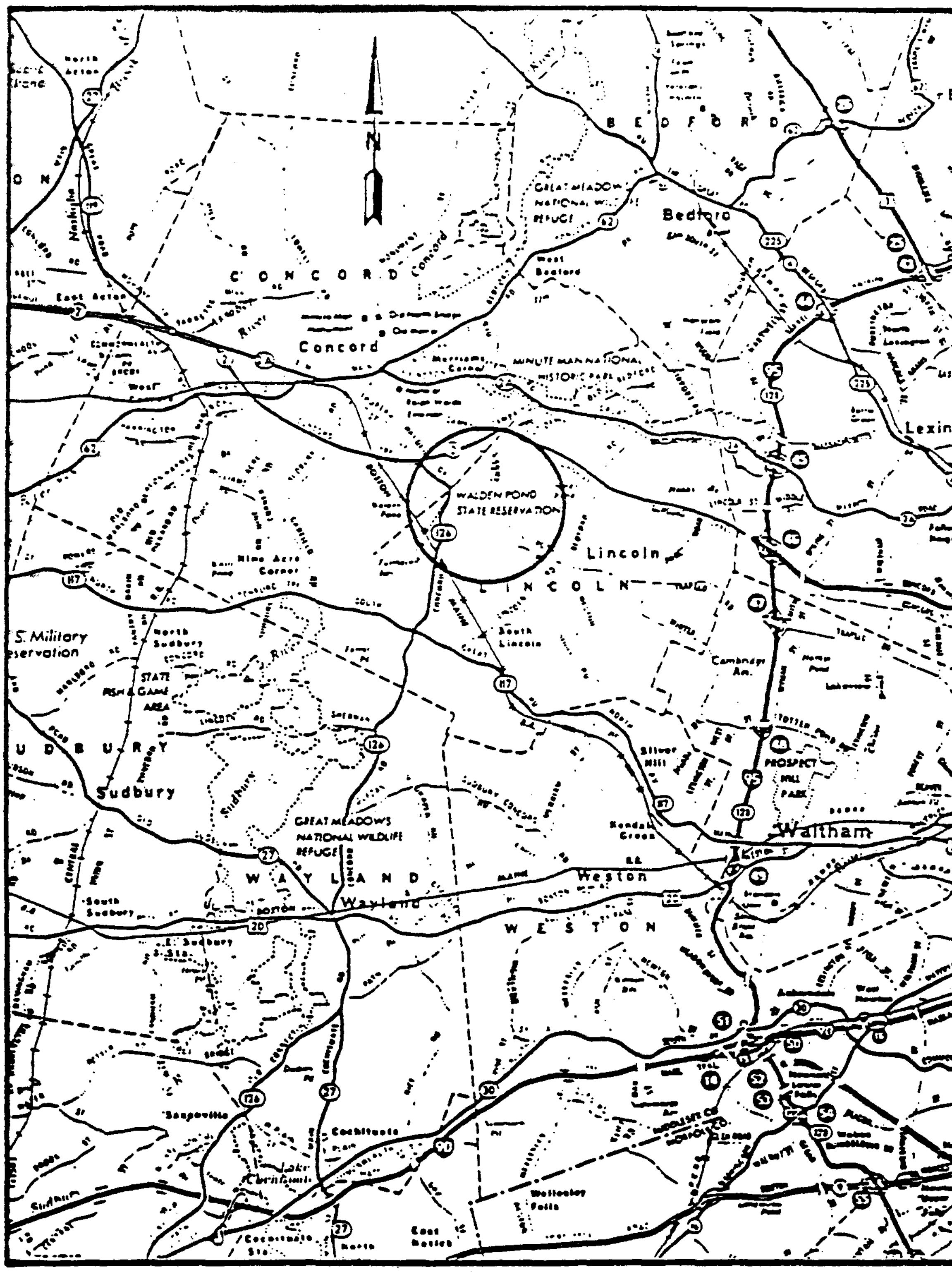


FIGURE 2-1 LOCUS PLAN

Topographically, the site was fairly uniform; elevations varied from approximately 202 ft to 210 ft. See Appendix N (Faulstich, 1979) for an approximate topographic plan of the site. Vegetation consisting of a mixture of deciduous and coniferous trees, with the former predominating, lined the periphery of the parking area. These trees provided shade and wind protection for certain sections of the site, especially section A and the entrance and exit roadways. Also these trees provided substantial leaf and needle cover during the autumn season.

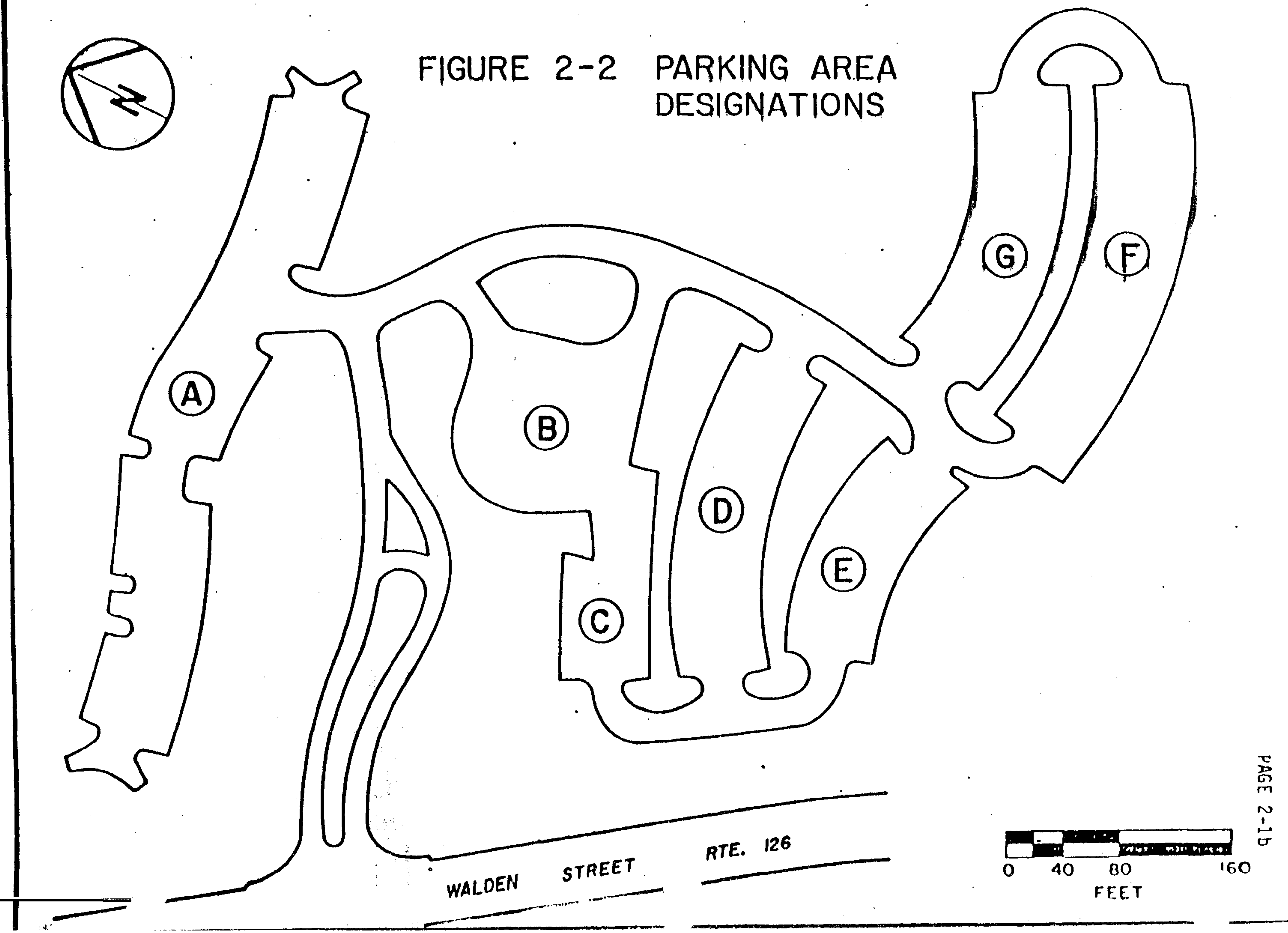
2.1.2 Soils Analysis

2.1.2.1 Virgin Soil

General information on the type of virgin soil situated in the Walden Pond area was obtained from the United States Soil Conservation Service, Department of Agriculture. These soil types are shown in Figure 2-3. The virgin soil in the locality of the parking area was of the Hinckley Series with slopes varying from 8 to 35 percent (See Figure 2-3, soil number 35, slope letter B). The Hinckley Series of soils is described by the Soil Conservation Service as follows:

> These are excessively drained soils developed in deep deposits of sands and gravel mainly from granite and gneiss. They are very sandy and gravely. They commonly have a gravely loamy sand surface soil and a sand and gravel subsoil

FIGURE 2-2 PARKING AREA DESIGNATIONS



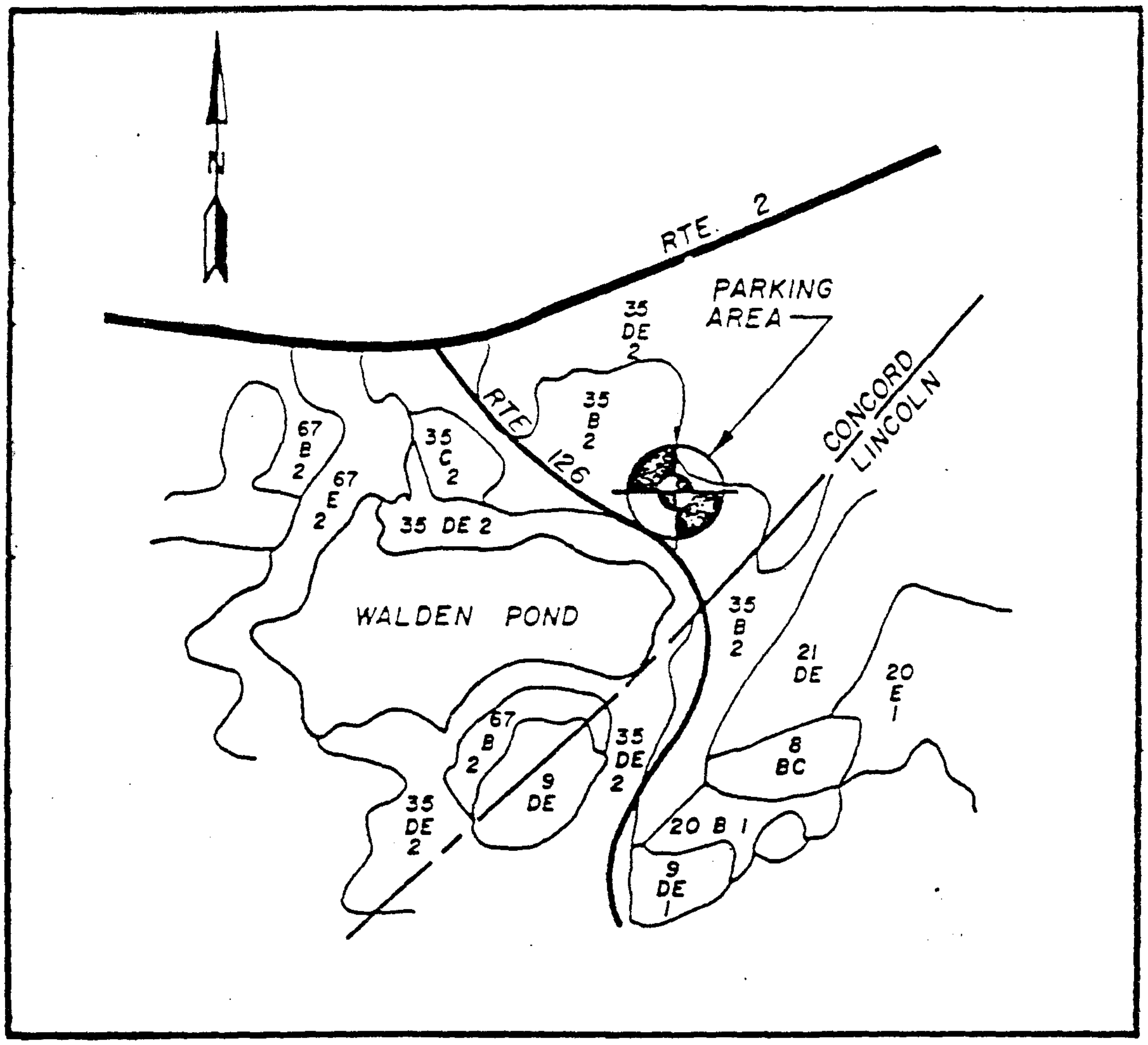


FIGURE 2-3 SOILS MAP

underlain by stratified sands and gravel. Hinckley soils are loose throughout and water moves rapidly through them. They are usually stone-free but may contain cobblestones and a few stones. In a few places the surface may be stony. They occur on level to very steep slopes.

Percolation tests were performed according to Massachusetts Department of Public Works (Mass. D.P.W.) specifications in order to roughly estimate the percolation rate of the virgin soil. An average of 1.5 minutes per inch drop was obtained from three determinations. This corresponds to a field permeability of 40 in./hr. A grain size distribution curve for the virgin soil is shown in Figure 2-4.

2.1.2.2 Gravel Fill

With the knowledge that a permeable pavement would be placed on the site in the near future, the specifications for the original construction of the gravel parking area in 1975 required at least three feet of non-heaving material in all sections. Through field testing certain sections of the virgin soil were found to be acceptable, while others were not. An excessively drained, non-heaving gravel borrow, Mass. D.P.W. M1.03.0 Type A and an excessively drained, non-heaving processed gravel, Mass. D.P.W. M1.03.1 were placed in depths sufficient to satisfy the job specifications. The gravel borrow was placed up to an elevation approximately 6 in. from finish grade. The

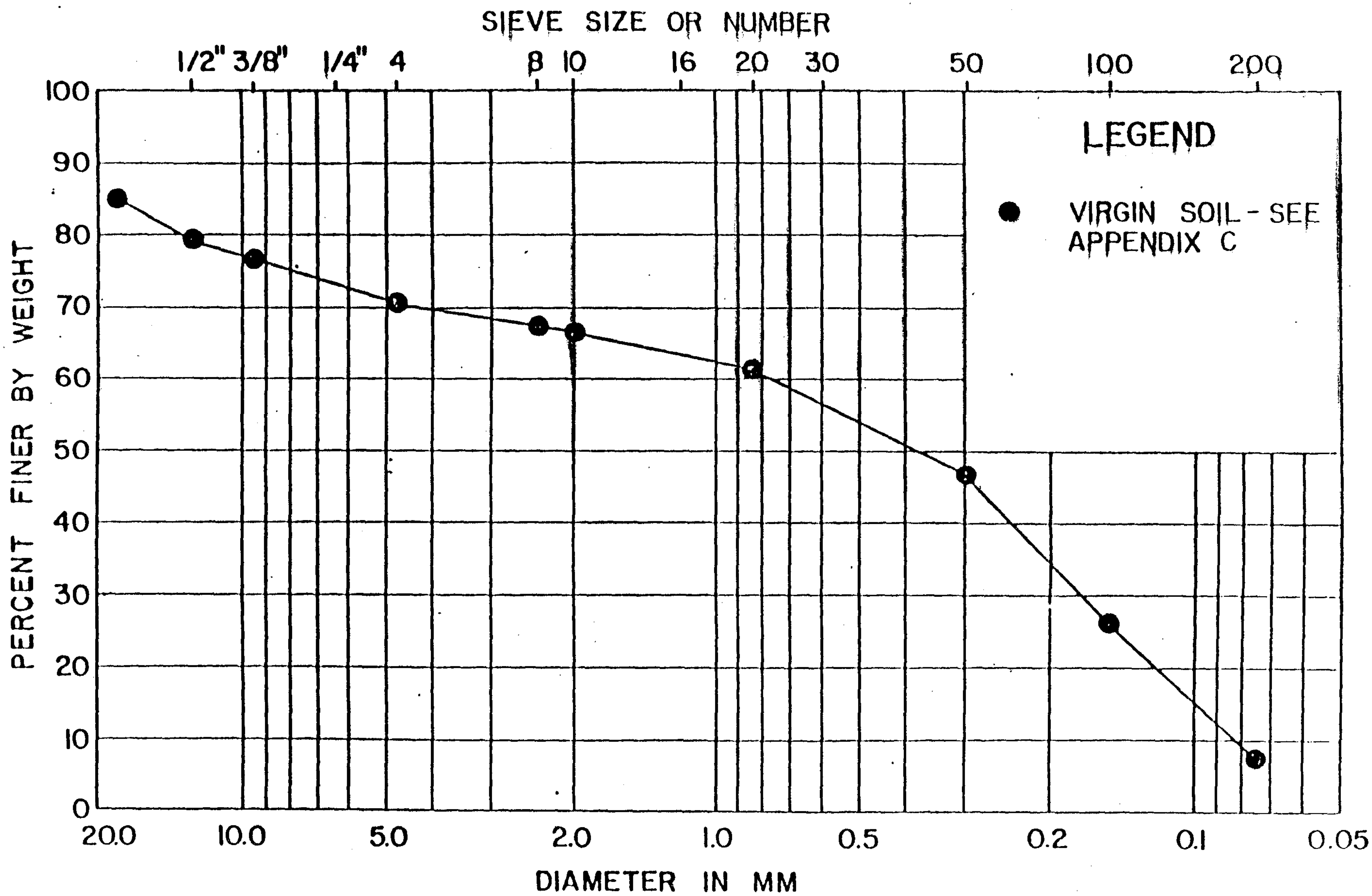


FIGURE 2-4 VIRGIN SOIL GRAIN SIZE DISTRIBUTION CURVE

processed gravel comprised the top 6 in. of the fill. Grain size distribution test results and a standard Proctor density test result for this fill obtained during the original construction of the parking area have been presented (see Appendix C, Faulstich, 1979) and are summarized in Figure 2-5.

In place density tests were performed on the gravel fill using a sand cone apparatus. The test locations are indicated in Figure 2-6. Moisture content determinations were performed on the soil removed during the density tests. Table 2-1 summarizes the density and moisture content data.

Three percolation tests were performed on the gravel fill according to Mass. D.P.W. specifications. An average value of 5.2 minutes per inch drop was obtained from these tests. This value was considerably larger than the average value obtained for the virgin soil. However, a percolation test result of 5 minutes per inch drop is equivalent to a permeability of 12 in./hr. Despite the fact that the percolation test is run under rather unrealistic conditions (a head of at least 12 in. and unrestricted lateral flow) the soil was assumed to be sufficiently permeable to imbibe all or most of the water applied to it.

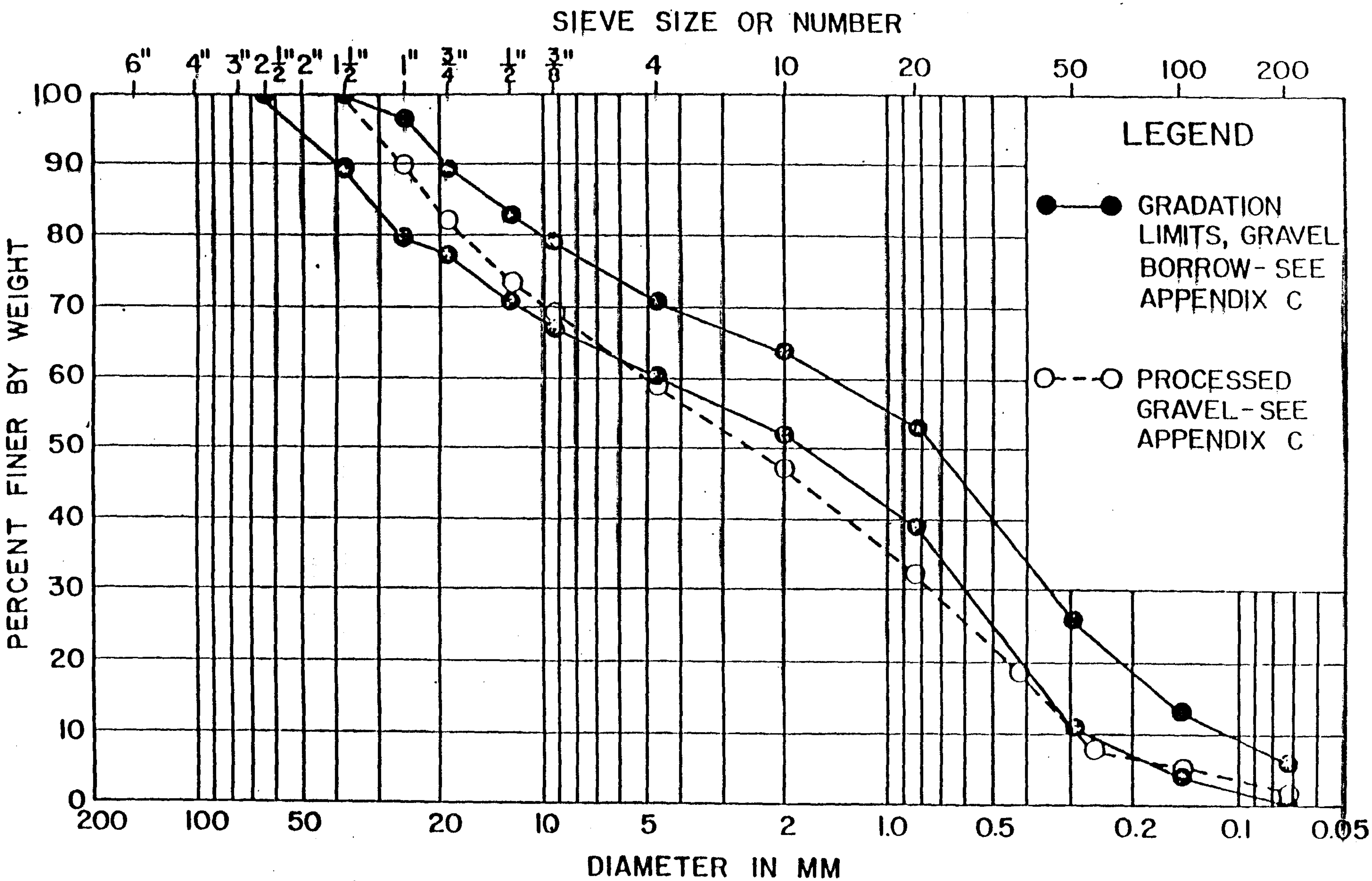


FIGURE 2-5 GRAVEL FILL GRAIN SIZE DISTRIBUTION CURVES

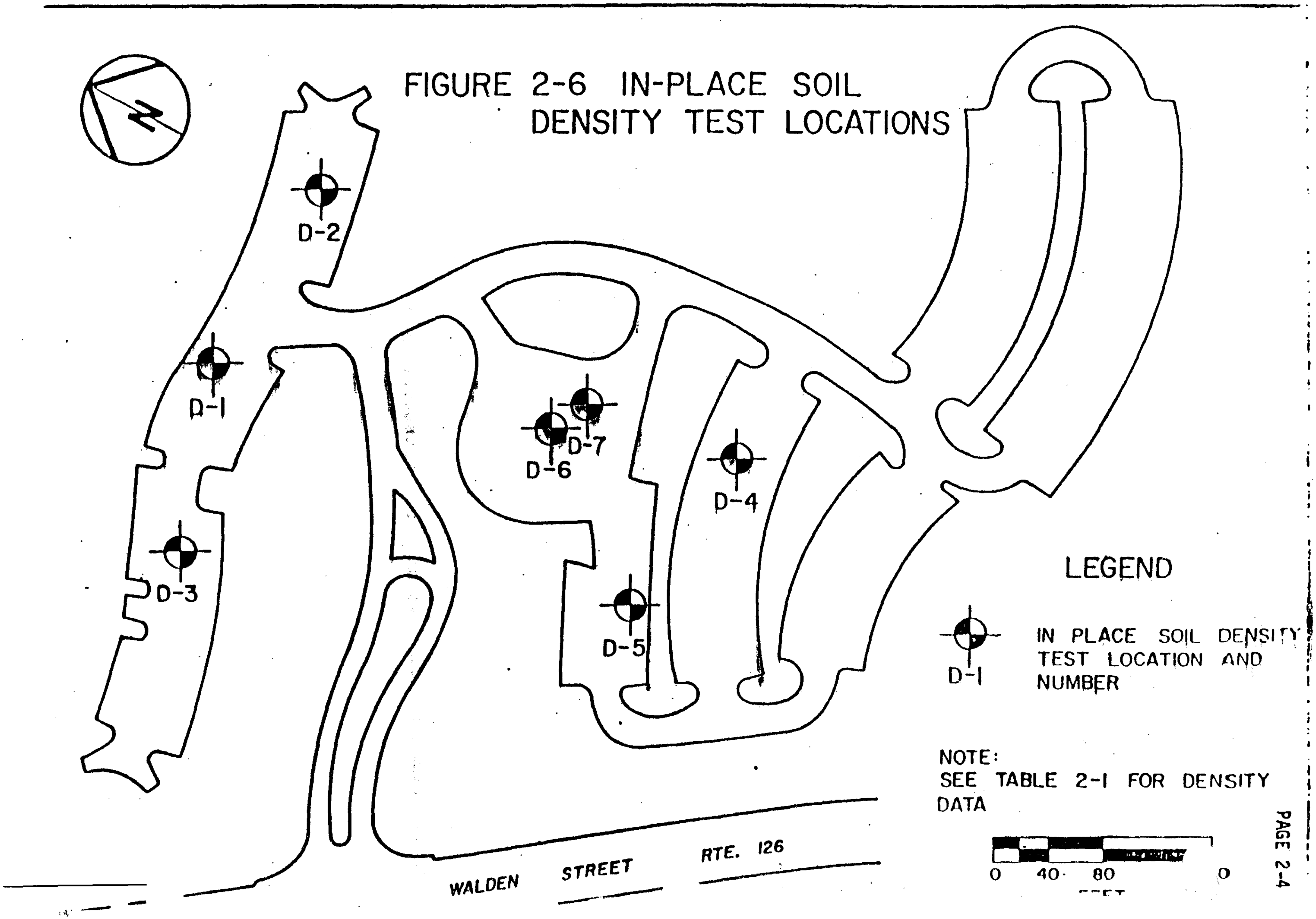


FIGURE 2-6 IN-PLACE SOIL DENSITY TEST LOCATIONS

TABLE 2-1

GRAVEL FILL DENSITY AND MOISTURE CONTENT DATA

Test No.	Wet Density pcf	Moisture Content %	Dry Density pcf
1	136.1	22.3	111.25
2	129.0	9.1	118.2
3	123.7	18.2	104.6
4	132.5	12.1	118.2
5	126.8	8.8	116.5
6	142.2	15.3	123.3
7	149.0	11.6	133.4
		Average =	117.9

2.1.3 Hydrologic and Climatic Characteristics

Precipitation data for the years 1973 through 1975 were obtained for a monitoring station in Bedford, Massachusetts, approximately 3.5 miles east of Walden Pond (National Oceanic and Atmospheric Administration, 1973-75). The data collected at this station was assumed to represent the climatic conditions at the Concord test site. Table 2-2 summarizes the precipitation data. Precipitation during the winter months of December through February was not considered since monitoring of the site was suspended during this period. This short term precipitation data was used merely to provide an indication of the magnitude of rain storms at the test site in order to properly size the water collection wells (See section 2.3.2). From this data it was determined that on the average, approximately 130 storms occured during the months of March through November, of which roughly 80 percent delivered a total precipitation of 0.5 in. or less, and roughly 92 percent delivered a total precipitation of one inch or less. Only one storm delivering 3 to 4 in. of precipitation was recorded during the months of March through November, for the years 1973 to 1975.

The mean annual air temperature and the average annual total precipitation taken from a 24 year record at Hanscom Field, Bedford Massachusetts are 48.5° F and 43.7 in. respectively (Eaton, 1978). The design freezing index for Concord, Massachusetts is 750° F-days based on the coldest year in a 30 year period and the mean freezing index for the 30 years is 250° F-days (Department of the Army, 1965).

Surface runoff from the parking area was either directed to grassed swales which led to pea stone dry wells, or was allowed to flow directly off the parking area into adjacent woodlands. Drainage calculations were performed to determine the effectiveness of the existing method of handling the surface runoff from a paved surface. A design storm of 10 year return frequency was used and, even when allowing for 95 percent runoff data. Precipitation during the winter months of December through February was not considered since monitoring of the site was suspended during this period. This short term precipitation data was used merely to provide an indication of the magnitude of rain storms at the test site in order to properly size the water collection wells (See section 2.3.2). From this data it was determined that on the average, approximately 130 storms occured during the months of March through November, of which roughly 80 percent delivered a total precipitation of 0.5 in. or less, and roughly 92 percent delivered a total precipitation of one inch or less. Only one storm delivering 3 to 4 in. of precipitation was recorded during the months of March through November, for the years 1973 to 1975.

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TABLE 2-2

PRECIPITATION DATA FOR BEDFORD, MASS., 1973-1975

Precipitation in.	1973-1975 Average No. of Storms			Average Yearly Total	% of Average Yearly Total	Cumulative % of Average Yearly Total
	March-May	June-Aug	Sept-Nov			
Trace Amount	10.0	7.7	9.7	27.4	20.8	20.8
0.01-0.05	10.3	10.0	10.0	30.3	23.0	43.8
0.05-0.10	5.7	3.3	3.3	12.3	9.3	53.1
0.10-0.15	3.3	3.3	3.7	10.3	7.8	60.9
0.15-0.20	1.3	2.7	1.7	5.7	4.3	65.2
0.20-0.30	5.0	2.3	2.7	10.0	7.6	72.8
0.30-0.40	2.3	1.0	2.7	6.0	4.5	77.3
0.40-0.50	3.0	1.0	2.0	6.0	4.5	81.8
0.50-0.60	1.0	1.7	0.7	3.4	2.6	84.4
0.60-0.70	0.7	1.0	0.7	2.4	1.8	86.2
0.70-0.80	1.0	1.0	0.7	2.7	2.0	88.2
0.80-0.90	1.0	0.7	0.7	2.4	1.8	90.0
0.90-1.00	1.7	1.0	0.3	3.0	2.3	92.3
1.00-2.00	2.0	2.7	4.3	9.0	6.8	99.1
2.00-3.00	0.0	0.0	0.7	0.7	0.5	99.6
3.00-4.00	0.0	0.3	0.0	0.3	0.2	99.8
				132.0		

from the paved surfaces, the existing system was determined to be marginally sufficient. Thus, no changes in the drainage system were made.

2.2 PAVEMENT SYSTEM DESIGN

2.2.1 General

A pavement system is defined as a structure consisting of superimposed layers of selected and processed materials whose primary function is to distribute the applied vehicular loads to the subgrade (O'Flaherty, 1967). Pavements can be either flexible or rigid; a flexible pavement is generally considered to be any pavement other than a concrete one. The OGAC pavement investigated in this study is classified as a flexible pavement.

Sargious (1975) lists five variables that should be considered in designing and constructing any pavement:

1. Load Variables
   Load variables result from various traffic types.

2. Regional Variables
   Regional variables include the types of existing soils, and environmental characteristics such as temperature, frost, rainfall, and storms.

3. Structural Variables
   Structural variables consist of different types of subgrades, subbases, and asphalt concrete surfaces.

4. Performance Variables

   Safety, the serviceability rating, and durability are the three performance variables.

5. Cost Variables

   The cost variables associated with pavement design and construction are construction costs and annual maintenance costs.

The subgrade strength is the main factor controlling the design of a flexible pavement. Pressure is transmitted from the surface to the subgrade through lateral distribution of the applied load with depth. If the subgrade deflects, so does the flexible pavement above it. Consequently, the basic design criterion for flexible pavements is the pavement depth required to distribute the applied surface load to the subgrade without overstressing the subgrade. If overstressing occurs, excessive subgrade deformation may take place. This deflection may exceed the limit beyond which structural damage occurs to the pavement (O'Flaherty, 1967).

One of the main concerns in the design of conventional dense flexible pavement systems is subgrade water. Cedergren and Godfrey (1974) state that most damage to pavements is caused by inadequate drainage of free water in the structural section of the pavement. Traffic loads on saturated structural sections can cause pumping, faulting, and other problems if the water is not free to escape. The source of subgrade water may be infiltration through cracked pavement surfaces or capillary rise by groundwater. The primary approach to eliminating water in pavements has been to seal the surface to prevent infiltration, or to provide sufficient drainage for ground water if seepage occurs (Tayabji et al., 1975). Very often drainage systems are designed by existing standards rather than being rationally designed and, as such, are often hydraulically inadequate. Another commonly utilized method of combating the drainage problem is to design a pavement strong enough to stand up over a weak, wet subbase. Historically this method has not shown to be very successful since very small quantities of water can saturate the densegraded, heavily compacted base courses commonly used today.

The design of a permeable pavement system is very different from the design of a dense pavement system. Water is allowed to freely enter the base course and, in order for the pavement system to remain structurally sound, provisions for adaquate drainage of the subgrade water must be made. The base course for a permeable pavement system ideally consists of crushed stone or a gravel of sufficient permeability. The base course acts both to distribute the applied loads to the subgrade and to store infiltrated water. The subgrade of a permeable pavement system must have an adequate load bearing capacity both wet and dry as well as a permeability sufficient to pass the infiltrated water in a reasonable length of time. Further, it must be non-frost susceptible to prevent capillary rise and frost heaving. FIRL has concluded that a subgrade soil likely to meet these requirements would be a sandy silt some distance above the water table. Its coefficient of permeability should be greater than 0.01 ft/day. Obviously, then, permeable pavement is site specific or requires replacement of unacceptable material with a highly permeable natural or processed fill.

2.2.2 Base Course

Two general types of base courses were utilized in this study, the existing gravel fill and an open-graded crushed stone. The characteristics of the gravel fill are presented in section 2.1.2.2. Three versions of a crushed stone base similar to that placed at the University of Delaware's permeable pavement installation were used. The first version consisted of a 10.5 in. layer of 3/4 in. to 2 in. stone overlaid by a 1.5 in layer of 3/8 in. stone. The second version consisted of two layers of 3/4 in. to 2 in. stone, one 4 in. thick and the other 5 in. thick, each overlaid with a 1.5 in. layer of 3/8 in. stone. The third version consisted of a 5 in. layer of 3/4 in. to 2 in. stone overlaid by a 1.5 in. layer of 3/8 in. stone. These open-graded bases provided a considerable storage reservoir for infiltrating water. The 3/8 in. stone courses were placed to stabilize the layers of 3/4 in. to 2 in. stone. According to section 3.05 of the job specifications (Commonwealth of Massachusetts, 1977):

> Crushed stone shall consist of durable crushed rrock consisting of the angular fragments obtained by breaking and crushing solid or shattered natural rock, and free from a

detrimental quantity of thin, flat, elongated or other objectionable pieces. The crushed stone shall be free from clay, loam or deleterious material, not more than 0.5 percent of satisfactory material passing a No. 200 sieve shall be allowed to adhere to the crushed stone and shall be uniformly graded to the requirements for the following type of crushed stone:

A. Type p

Square Opening Sieve	Percent Passing
1/2 in.	100
3/8 in.	0-5

B. Type q

Square Opening Sieve	Percent Passing
2 1/2 in.	100
2 in.	95-100
1 1/2 in.	35-70
1 in.	0-15
3/4 in.	0-5

Typical laboratory permeabilities of 140,000 ft/day for an aggregate size range of 1 to 1 1/2 in. and 38,000 ft/day for an aggregate size range of 3/8 to 3/4 in. have been reported by Cedergren (1974). See Figure 2-7 for the base course cross sectional plans.

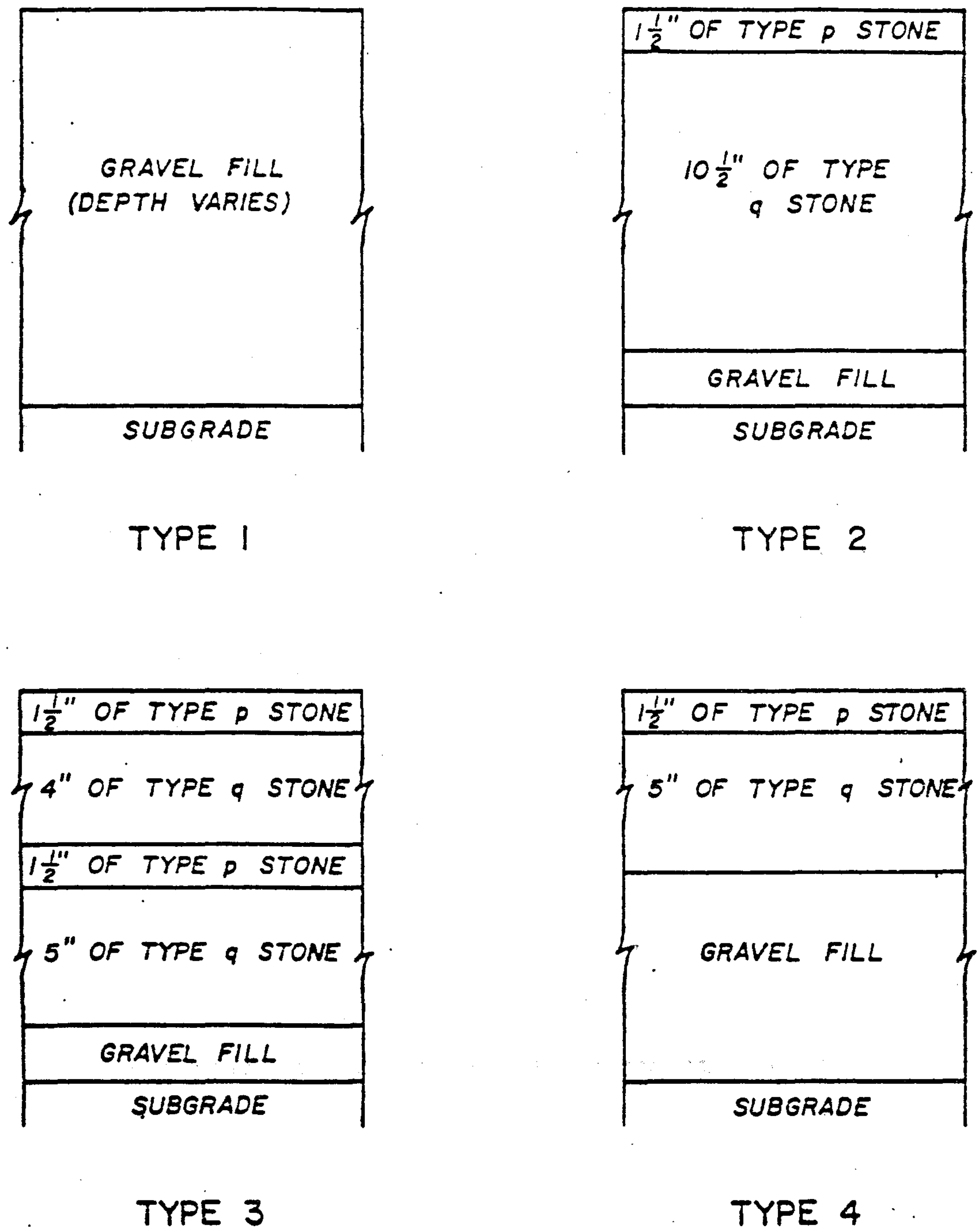


FIGURE 2-7 BASE COURSE TYPES

2.2.3 Asphalt Concrete

Asphalt concrete is composed of an aggregate combination bound together with an asphalt cement. The aggregate combination is the most influential factor in determining the mix characteristics. By varying the relative amounts of coarse aggregate (retained on a No. 8 sieve), fine aggregate (passing a No. 8 sieve), and mineral dust (passing a No. 200 sieve), mixtures ranging from very dense to very permeable can be obtained. The asphalt cement content also exhibits an influence on the mix characteristics, to a lesser degree however.

Four asphalt concrete mix classes were specified for the field site: Three open-graded mix classes, J, K, and L, and one dense-graded mix class, I. Mix class J was the "California mix" recommended by FIRL. Mix class K was an open-graded mix developed by Warren Brothers, Inc., the construction contractor. Mix class L was a modification of mix class J; more fine aggregate was included. Mix class I was the standard dense mix specified by the State of Massachusetts. The composition limits for each mix class are presented in Table 2-3. These four mix classes theoretically provided various degrees of permeability ranging from very dense (mix class I) to very permeable (mix class J). The job mix formulas and asphalt contents for each mix class were determined by the contractor and approved by the State of Massachusetts. They are presented with the allowable tolerances in Table 2-4. The asphalt cement used for all mixes was an Exxon AC-20 viscosity graded asphalt.

TABLE 2-3

MIX CLASS COMPOSITION LIMITS

Percent by Weight Passing Square-Opening Sieves					
Sieve	Mix Class I Binder Course	Mix Class I Top Course	J	K	L
7/8 in.	100				
3/4 in.	80-100				
1/2 in.	55-80	100	100	100	100
3/8 in.		80-100	90-100	100	80-100
No. 4	28-50	50-76	35-50	25-40	35-50
No. 8			15-32	0-10	20-32
No. 10	18-35	35-50			
No. 16			0-15	0-5	5-20
No. 20		22-35			
No. 40	7-18	14-27			
No. 80		6-18			
No. 200	1-5	2-7	0-3	0-3	0-3
Asphalt %	4.5-5.5	5.5-7.0	5.0-7.0	3.5-5.0	3.5-5.0

TABLE 2-4

JOB MIX FORMULAS AND ALLOWABLE TOLERANCES

Sieve	Mix Class I				Mix Class J		Mix Class K		Mix Class L	
	Binder Course JM	Max. Tol.	Top Course JM[a]	Max. Tol.[b]	JM	Max. Tol.	JM	Max. Tol.	JM	Max. Tol.
	Percent by Weight Passing Square-Opening Sieves									
7/8 in.	100	7.0%								
3/4 in.	93	7.0%								
1/2 in.	68	7.0%	100	7.0%	100	5.0%	100	5.0%	100	5.0%
3/8 in.			93	7.0%	90	5.0%	100	5.0%	87	5.0%
No. 4	37	7.0%	63	7.0%	40	5.0%	35	5.0%	44	5.0%
No. 8					17	3.0%	5	5.0%	26	5.0%
No. 10	26	4.0%	40	4.0%						
No. 16					12	3.0%	2	5.0%	19	5.0%
No. 20			31	4.0%						
No. 40	13	4.0%	20	4.0%						
No. 80			12	4.0%						
No. 200	2	2.0%	4	2.0%	1	0.5%	1	0.5%	1	0.5%
Asphalt %	5.0	0.4%	6.4	0.4%	5.0	0.3%	4.5	0.3%	4.5	0.3%

[a]Job mix formula

[b]Maximum Tolerance ±

Asphalt concrete thickness was varied to determine the influence of thickness on permeability and stability. Thicknesses of 1.5 in., 2.5 in., and 4 in. were used for mix class J and thicknesses of 2.5 in. and 4 in. were used for mix class I. Both mix class K and mix class L were placed solely in a depth of 2.5 inches. One test section was left unpaved to act as a control. Two sections of mix class J, one 4 in. thick and the other 2.5 in. thick, were placed in two lifts. These were located in section A (See Figure 2-2). The asphalt concrete in all other sections was placed in a single lift except for mix class I which consisted of a binder and a top wearing course.

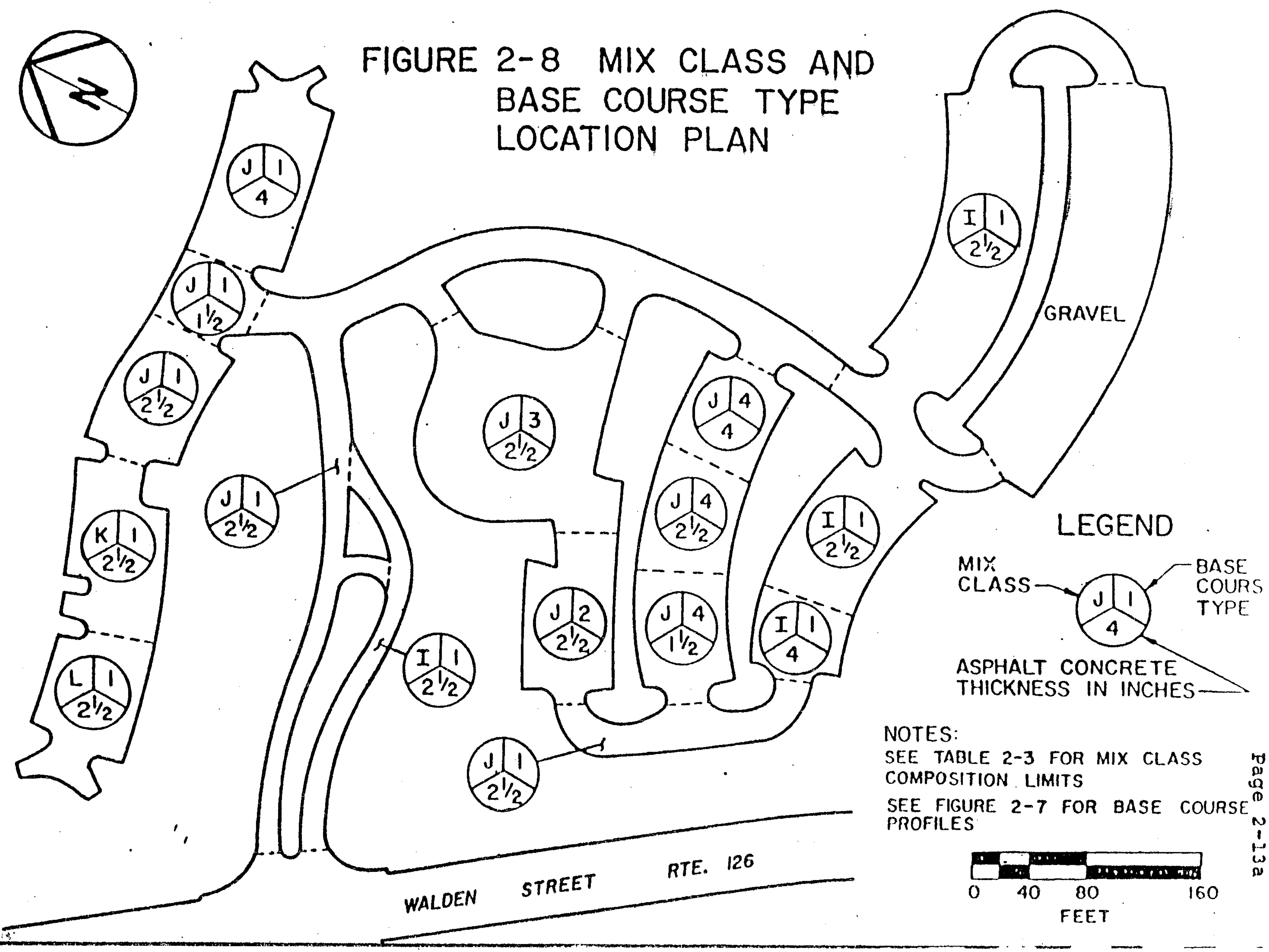
The placement locations for each mix class are shown in Figure 2-8. Also shown in Figure 2-8 are the base course types and asphalt concrete thickness.

2.3 MONITORING SYSTEM

2.3.1 General

The parameters monitored at the field site were water quality and quantity, depth of penetration of freezing temperatures in the soil, air temperature, and pavement system stability. Of these, water quality and quantity, depth of penetration of freezing temperatures in the soil, and air temperature required permanent monitoring equipment at the test site. Pavement system stability was tested periodically by mobile testing equipment. A simplified schematic of the

FIGURE 2-8 MIX CLASS AND BASE COURSE TYPE LOCATION PLAN



monitoring system is shown in Figure 2-9. The system is described in the following sections.

2.3.2 Water Collection Wells

Water collection wells were required in order to determine the following hydrologic components: total precipitation, surface runoff, and infiltration through the pavement surface. By collecting and measuring these quantities, a mass balance could be performed to determine the relative percentage of each component:

$$\text{precipitation} = \text{runoff} + \text{infiltration} + \text{losses}.$$

In addition, these wells stored water for analysis of its quality. Poly-vinyl chloride (PVC) schedule 40 tubing was chosen as the construction material for the water collection wells owing to its workability, availability, and low cost.

Design curves showing the tributary area required to fill several sizes of collection wells for various total rainfall depths were prepared and are shown in Figure 2-10. One hundred percent runoff was assumed in preparing these curves. An inside diameter of 4 in. was chosen for all water collection wells because of the availability of the necessary fittings. Tradeoffs between well length, drainage area size, and maximum collectable precipitation were made using the design curves. A maximum design storm of approximately 0.3 in. was selected. This allowed the collection, without an overflowing of the

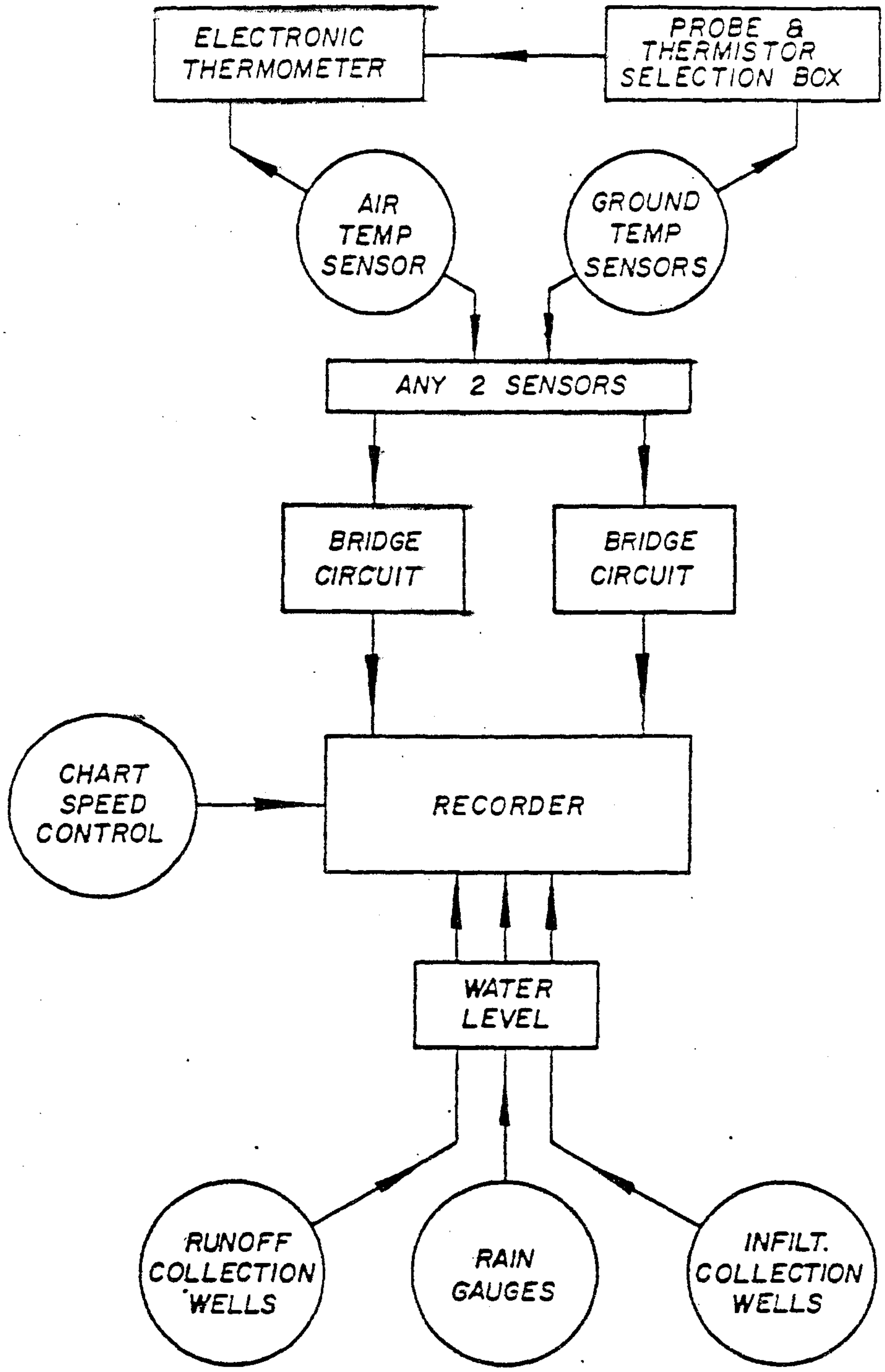


FIGURE 2-9 MONITORING SYSTEM SCHEMATIC

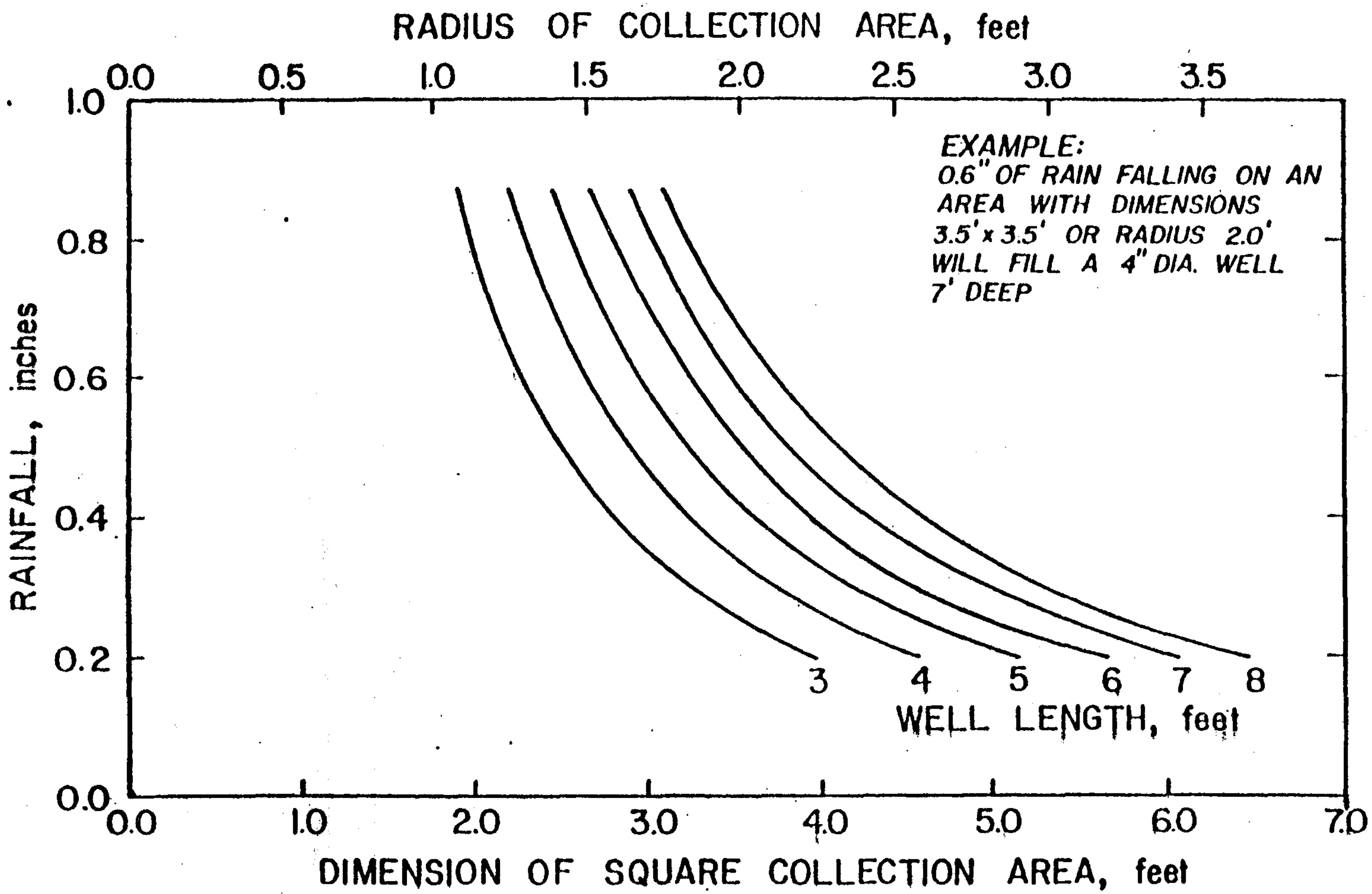


FIGURE 2-10 WATER COLLECTION WELL DESIGN CURVES

wells, of approximately 75 percent of the storms occuring during the monitoring period of the year (See Table 2-2). Well depths were limited to about 7 ft. to facilitate removal of the water collected in the wells and to minimize construction problems. By considering appropriate coefficients of runoff for the various mixes used, well depths and drainage areas were calculated.

2.3.2.1 Runoff Collection Wells

The runoff collection wells consisted simply of a PVC tube sealed at the bottom and fitted with a perforated screw-on cap. These wells were placed at the low point in a defined drainage area (See Figures 2-11, 2-12, B-12, and B-13) (Faulstich, 1979). A drainage area of 4 by 4 ft was selected for the runoff wells located in the dense pavement areas while a drainage area of 8 by 8 ft was chosen for the runoff wells located in the permeable pavement areas. Depths sufficient for collecting the design storm were calculated for each well.

2.3.2.2 Infiltration Collection wells

The infiltration collection wells consisted of a PVC tube sealed at the bottom and fitted with a solid removable cap. A short distance from the top of the well, a plastic liner was connected by a coupling to the PVC tube. Holes were drilled in the tube at the level at which the liner joined the tube. These wells were placed with the plastic liners buried and sloped towards the well (See Figures 2-13, 2-14, and B-8 through B-11)

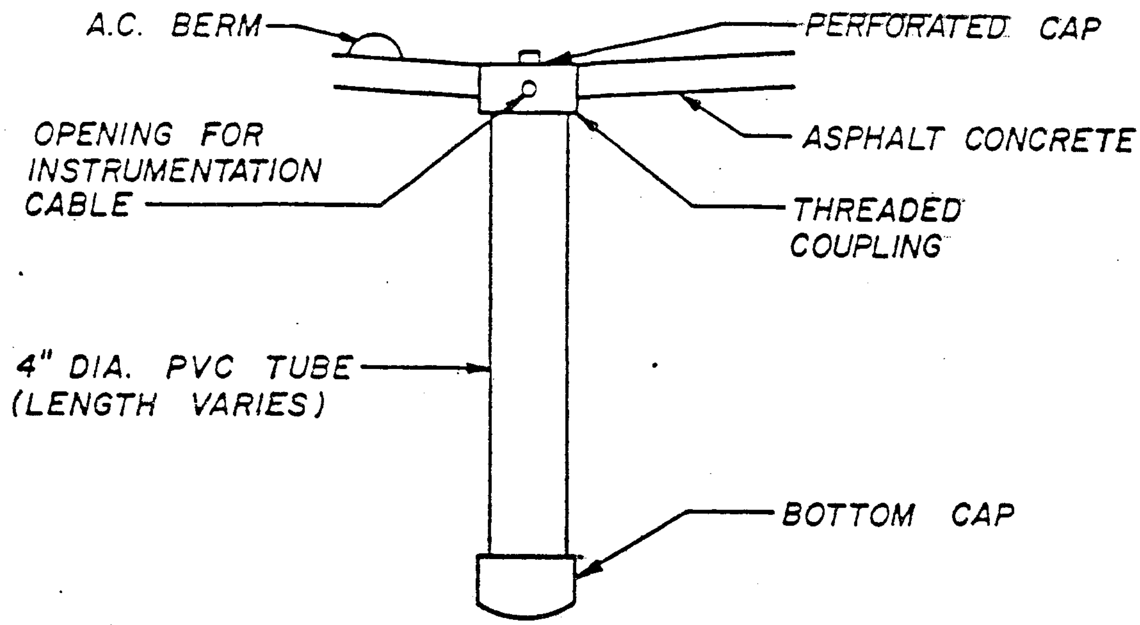
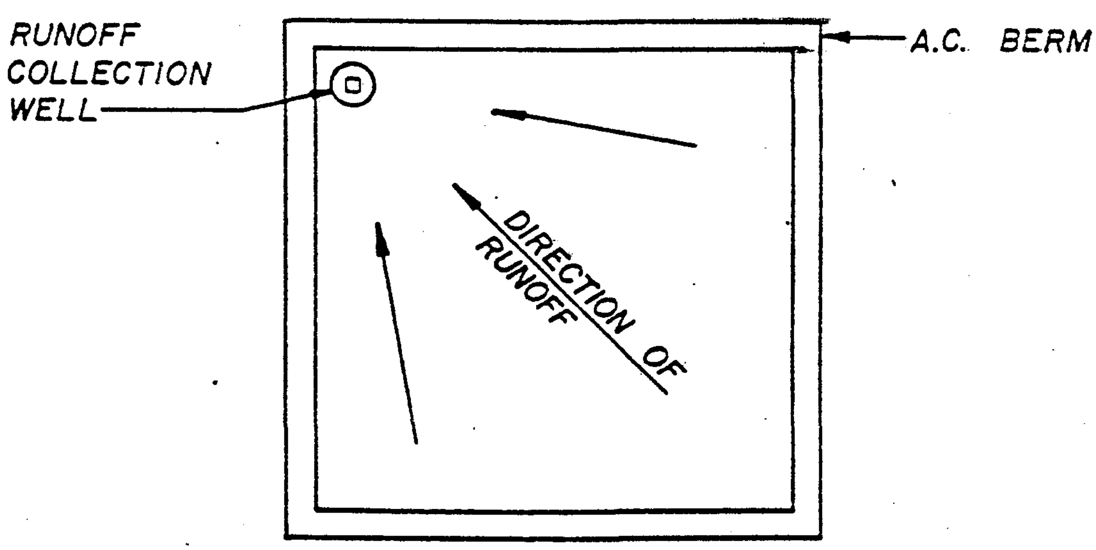


FIGURE 2-11 TYPICAL RUNOFF COLLECTION WELL



NOTE: SIZE OF COLLECTION AREA VARIES

FIGURE 2-12 TYPICAL RUNOFF COLLECTION WELL DRAINAGE AREA

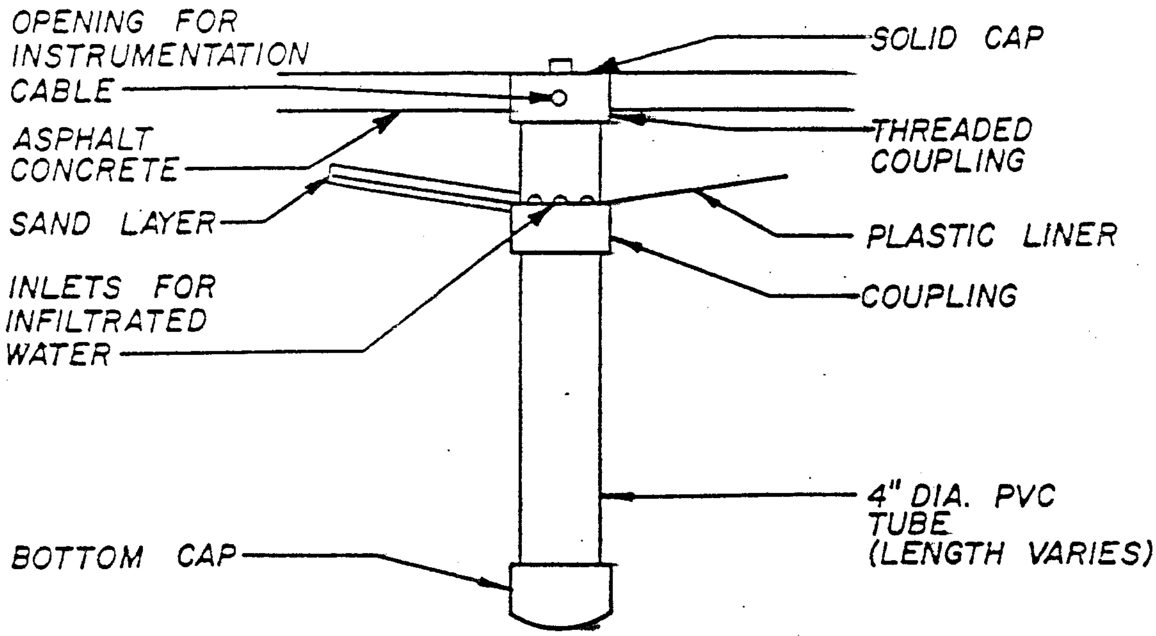
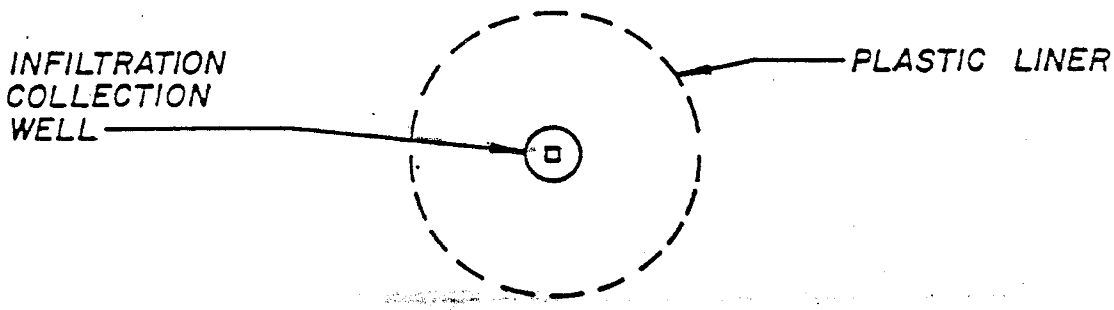


FIGURE 2-13 TYPICAL INFILTRATION COLLECTION WELL



NOTE: SIZE OF COLLECTION AREA VARIES

FIGURE 2-14 TYPICAL INFILTRATION COLLECTION WELL DRAINAGE AREA

(Faulstich, 1979). Water which infiltrated the pavement surface percolated through the base course until it reached the impervious plastic liner. From there water ran laterally into the well. Sizing of the liner diameter and of the well depth were done in the same manner that was used in the design of the runoff collection wells.

2.3.2.3 Rain Gauges

Two rain gauges were also constructed of PVC tubing. One of these gauges had a 1.5 in. inside diameter, and the other gauge had a 3 in. inside diameter. Both gauges were fitted with a 10.25 in. diameter collection funnel. A typical rain gauge is shown in Figure 2-15. The actual rain gauges are shown in Figure B-14 (Faulstich, 1979).

2.3.2.4 Rainfall Sensor

A rainfall sensor was constructed to change the recorder chart speed (See section 2.3.4.3) from 2 in./hr during dry periods to 9 in./hr during a rainstorm. This was designed to spread out the recorded data on the chart paper for ease in data reduction. The sensor consisted of a 10.25 in. diameter funnel fitted with an adjustable valve at the outlet. Two electrodes were fastened to the funnel and connected to a relay package. The valve was adjusted to restrict the flow of water from the funnel enough so that a fairly substantial rainfall would be retained in the funnel long enough to short out the electrodes.

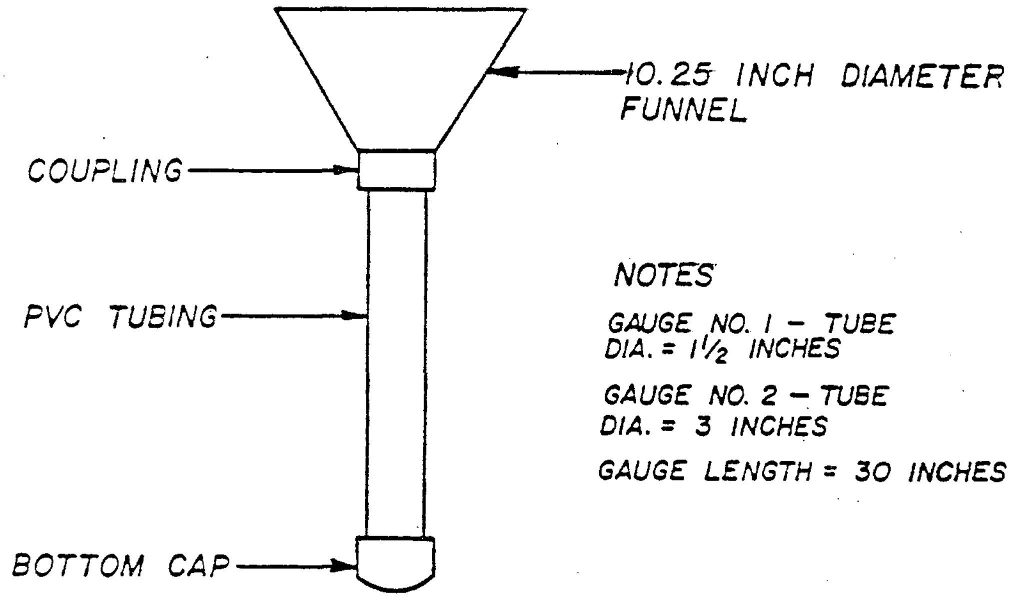


FIGURE 2-15 TYPICAL RAIN GAUGE

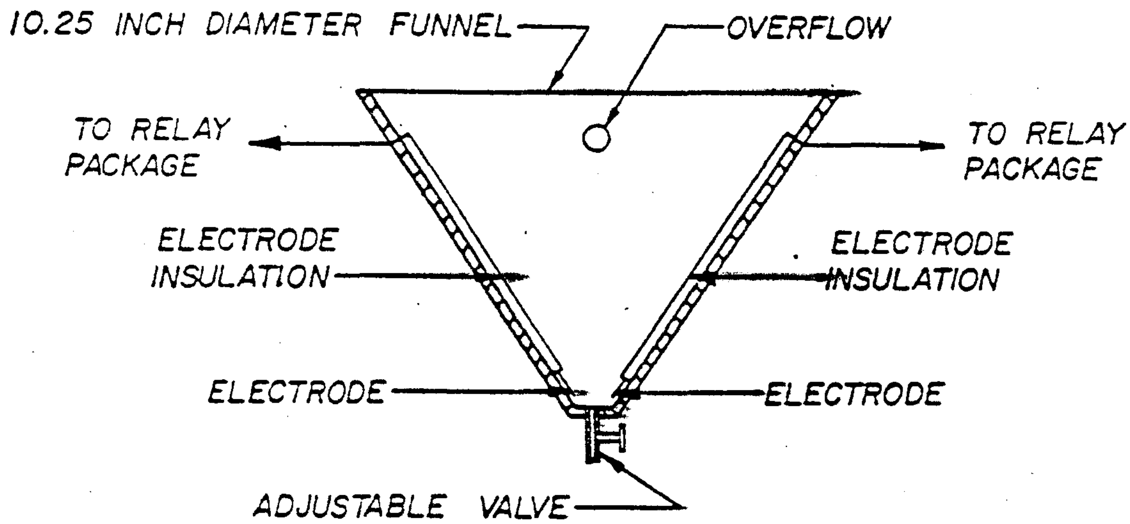


FIGURE 2-16 RAINFALL SENSOR

When water entered the funnel and shorted out the electrodes, thus completing the circuit, the relay was energized and started the timer motor. After a 3 minute delay, if the electrodes were still shorted out, another relay was energized. This relay was normally closed and in this position set the chart drive plug at 2 in./hr. When energized the relay set the chart drive plug at 9 in./hr. The system stayed in the high speed mode until the rainfall sensor no longer detected water. The relay would then return to the normally closed position, shifting the recorder chart speed back to 2 in./hr. See Figure 2-16 for a diagram of the rainfall sensor.

2.3.2.5 Water Collection Well Locations

All of the water collection wells were placed in the corners of the parking areas to provide some protection from potential damage from automobiles. Four water collection wells were placed in mix class J. Of these, three were infiltration collection wells and one was a runoff collection well. Since mix class J was expected to be very permeable, sufficient data for a water balance was anticipated from the infiltration collection wells. Two water collection wells were placed in mix class K: one runoff collection well and one infiltration well. Two runoff collection wells were placed in mix class I because it was a dense mix. Two water collection wells were installed in the gravel section. Here again one runoff collection well and one infiltration well were used. No water collection wells were installed in mix class L. The rain gauges were located in a clearing adjacent to the parking area. The layout of the water collection wells at the test site is shown in Figure 2-17.

2.3.3 Temperature Monitoring Devices

Initially four temperature probes were installed to monitor ground temperatures beneath the pavement. Each probe consisted of 10 temperature sensors mounted at 6 in. intervals on a 3/4 in. by 1 in. by 66 in. plastic bar (see Figure B-7, Faulstich, 1979). The assembled probes were buried with the uppermost sensor approximately at the existing ground level. This allowed the ground temperatures to be measured down to a depth of 5 feet. A typical temperature probe assembly is shown in Figure 2-18. In addition to the probes for measuring ground temperatures, a temperature sensor for measuring air temperature was mounted at a height of approximately 4.5 ft above ground level. This sensor was shielded from both the wind and the sun.

The temperature sensors were YSI No. 401 general purpose probes. They consisted of a thermistor temperature sensing element housed in a probe and attached to a plasticized vinyl jacketed shielded lead wire. The thermistor resistance varied non-linearly with temperature.

One temperature probe assembly was installed in each of the mix classes except for mix class L. A probe was also installed in the gravel section. The layout of the temperature probes at the test site is shown in Figure 2-17.

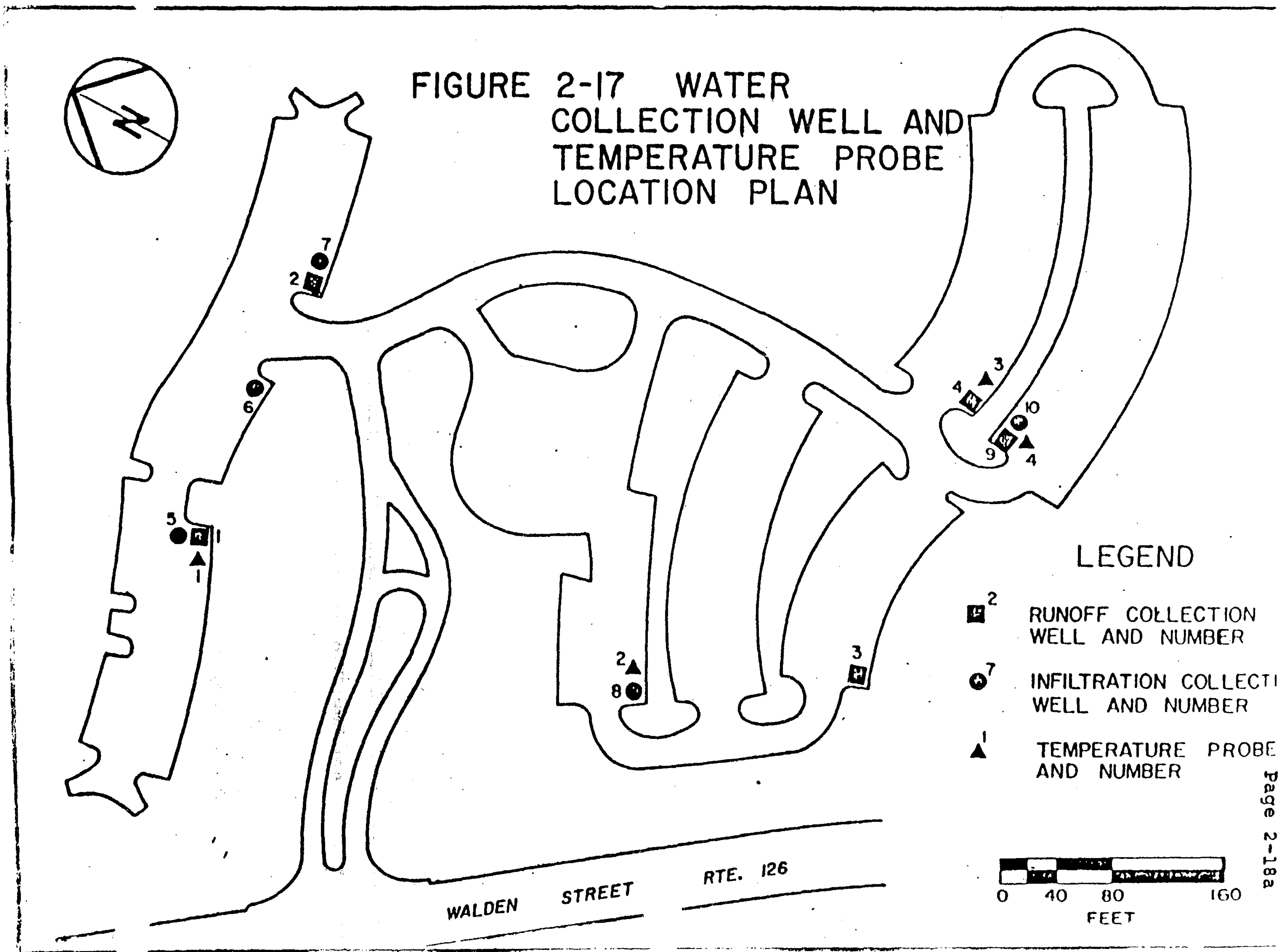


FIGURE 2-17 WATER COLLECTION WELL AND TEMPERATURE PROBE LOCATION PLAN

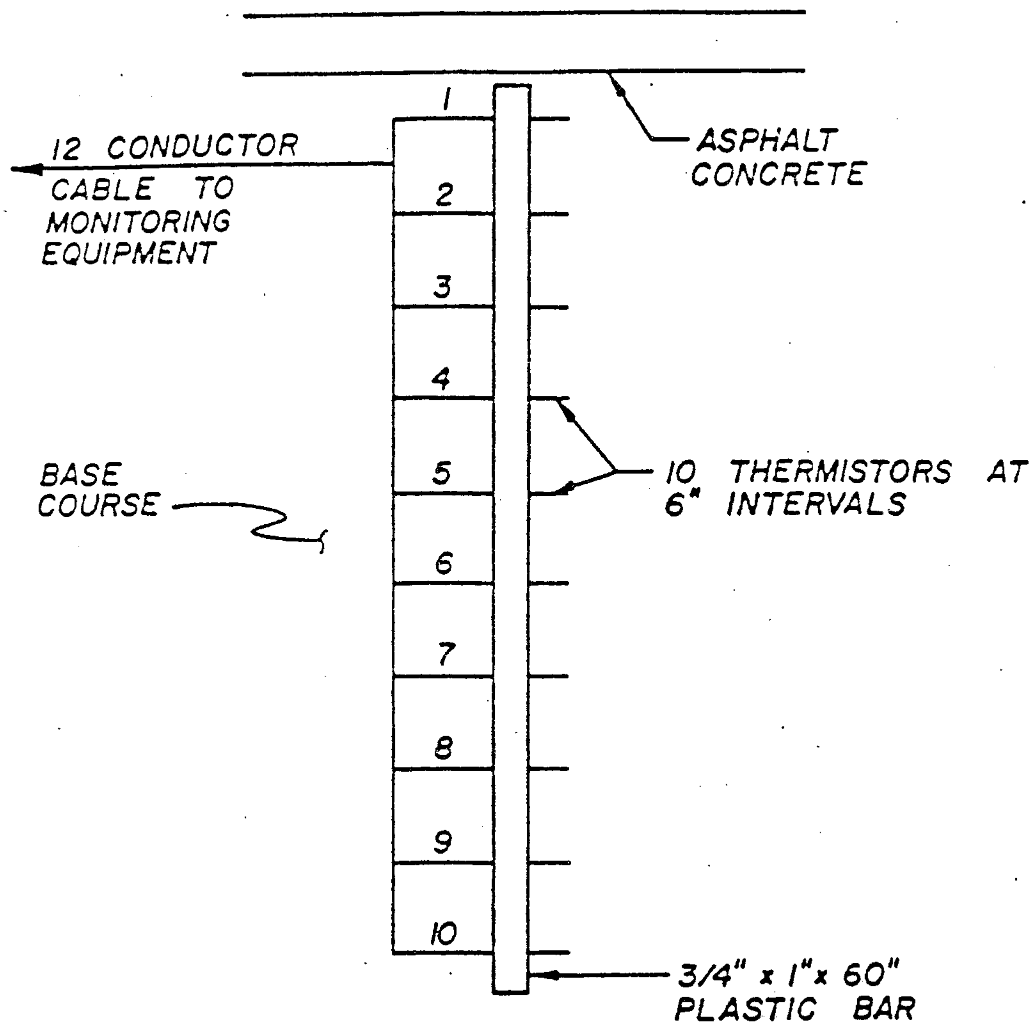


FIGURE 2-18 TYPICAL TEMPERATURE PROBE

2.3.4 Instrumentation

The instrumentation installed at the test site consisted of: (1) the equipment used to detect water levels in the water collection wells, (2) the equipment used to display both ground and air temperatures, and (3) the equipment used to continuously record the data. Instrumentation not installed directly on-site were situated in an instrumentation shed adjacent to the parking area. This equipment is shown in Figure B-16 (Faulstich, 1979).

2.3.4.1 Water Level Indicators

Electronic liquid level measuring elements and transmitters manufactured by Amiprodux, Inc., were used in all of the water collection wells and in the two rain gauges. The elements were galvanized to prevent corrosion. Acrylic disks were attached to the bottom of the elements to displace water in a slight "dead space" at the end of the element. Figure B-15 (Faulstich, 1979) shows a typical water level indicator. Power was supplied to the transmitters by central power units which were also manufactured by Amiprodux, Inc.. The outputs from each central power unit were connected to the strip chart recorder (See section 2.3.4.3).

2.3.4.2 Electronic Thermometer

An adjustable range temperature meter, YSI model 42, was used to display the temperatures sensed by the thermistors. The meter actually sensed thermistor resistance and provided an instantaneous meter readout of the corresponding temperature. The leads from the thermistor were connected to a switching panel assembly from which each thermistor could be connected to the temperature meter. Thus, the meter was for attended use only.

In order to obtain a continuous reading of temperature, two bridge circuits were assembled which provided a voltage output corresponding to the thermistor resistance and matching the strip chart recorder input voltage range. These circuits allowed for a continuous record of any two thermistor readings.

2.3.4.3 Strip Chart Recorder

A Westronics M11E 24 point strip chart recorder was used to continuously record the water level and temperature data. When monitoring water quantity during the spring, summer, and fall, 12 points on the recorder were utilized. A print interval of 30 seconds between successive points was selected. The resulting time for a complete cycle of all 12 points was 6 minutes. Recorder chart speed was controlled by a chart drive plug which could be set for a chart speed from 1 in./hr to 120 in./hr. Normal recorder chart speed was set at 2 in./hr.

CHAPTER 3

CONSTRUCTION

3.1 GENERAL

The construction contract was awarded to the Warren Brothers Company, Inc., a nation-wide contractor with a division in Cambridge, Massachusetts. Construction of the parking area began in August of 1977. A field engineer from the Massachusetts Department of Environmental Management was responsible for overseeing the construction. Representatives from Northeastern University were also present inspecting at all times, and were directly responsible for laying and testing the instrumentation cable and for the installation of the water collection wells and temperature probes. Construction involved the following steps:

1. Instrumentation cable installation
2. Base course preparation
3. Water collection well and temperature probe installation
4. Asphalt concrete placement.

3.2 INSTRUMENTATION CABLE INSTALLATION

Two types of cables were required for the monitoring system. These were a three conductor cable for the water level indicators and a twelve conductor cable for the temperature probes. Cable trenches were excavated to a depth of 10 in. below existing ground level and to a sufficient width (approximately 18 in.) (see Figure B-1, Faulstich, 1979). The cables were shielded by PVC conduit wherever the ground surface was to be paved (See Figure B-2, Faulstich, 1979). All cables were tested for continuity immediately after installation (prior to backfilling), after backfilling, and after grading (prior to paving). The cables were also tested for high voltage leakage after backfilling and after grading.

3.3 BASE COURSE PREPARATION

Preparation of the various base courses used at the test site consisted of placement of the crushed stone base and final grading of the gravel base. The areas in which a crushed stone base was to be placed were excavated to a depth equal to the thickness of the specified crushed stone base and were graded. The crushed stone was placed and graded by a "Jersey Box" and a standard grader (See Figures B-4 and B-5, Faulstich, 1979) and then rolled. All other areas not requiring crushed stone base were final graded and rolled.

3.4 WATER COLLECTION WELL AND TEMPERATURE PROBE INSTALLATION

Excavation for the water collection wells and temperature probes was acomplished by a power auger, a backhoe, and by hand. The water collection wells were installed such that their tops projected above the existing ground level a distance equal to the thickness of the asphalt concrete to be placed in that location. Wells 3 and 8 were unintentionally set slightly low in the ground in relation to the final surface elevation. Well 3, being a runoff collection well, was not seriously affected by this. Well 8, being an infiltration collection well and as such not having berms surrounding it, was seriously affected since it tended to drain a large portion of section D. Well 8 was later converted to a runoff collection well (See section 3.5.5). Sand was placed above and below the plastic liners of the infiltration collection wells to protect the liners from stones. A small amount of type p stone was placed around the inlets in the infiltration collection wells to prevent sand and silt from washing in. The berms surrounding the runoff collection well in the gravel section (well 9) were constructed of 2 in. by 8 in. boards set in the ground.

The temperature probes were installed such that the uppermost sensor of each was approximately 2 in. below existing ground level. The connections between the temperature sensor leads and the instrumentation cables were sealed with silicon rubber in junction boxes buried adjacent to the probes.

3.5 ASPHALT CONCRETE PLACEMENT

3.5.1 General

Conventional equipment and construction methods were used for the placement of the asphalt concrete. A Blaw-Knox 500 paving machine was used to lay the asphalt concrete and a Galion 10 ton static roller was employed for compaction of the asphalt concrete (See Figures B-17, B-18, and B-21, Faulstich, 1979). Asphalt concrete in the areas surrounding the water collection wells was placed and compacted by hand (See Figure B-19, Faulstich, 1979). The berms surrounding the runoff collection wells were sealed with an asphalt-based sealant to prevent intrusion of runoff water from other sections of the parking areas. Unheated 10 wheel trucks were used to transport the asphalt concrete from the batch plant to the project site. No truck traffic was allowed on the OGAC during construction in an attempt to prevent possible clogging of the surface with soil from the truck tires.

Three individuals were responsible for quality control of the asphalt concrete, the field engineer, a represenitive of Northeastern University, and a technician from the Warren Brothers Company, Inc.. The field engineer, representing the Massachusetts Department of Environmental Management, was responsible for overseeing the paving procedures and especially for controlling the asphalt concrete thicknesses. Arrival and placement asphalt concrete temperatures were monitored by the Northeastern University representative. Asphalt concrete densities were monitored by the technician from the Warren Brothers Company, Inc. using a nuclear density-moisture meter.

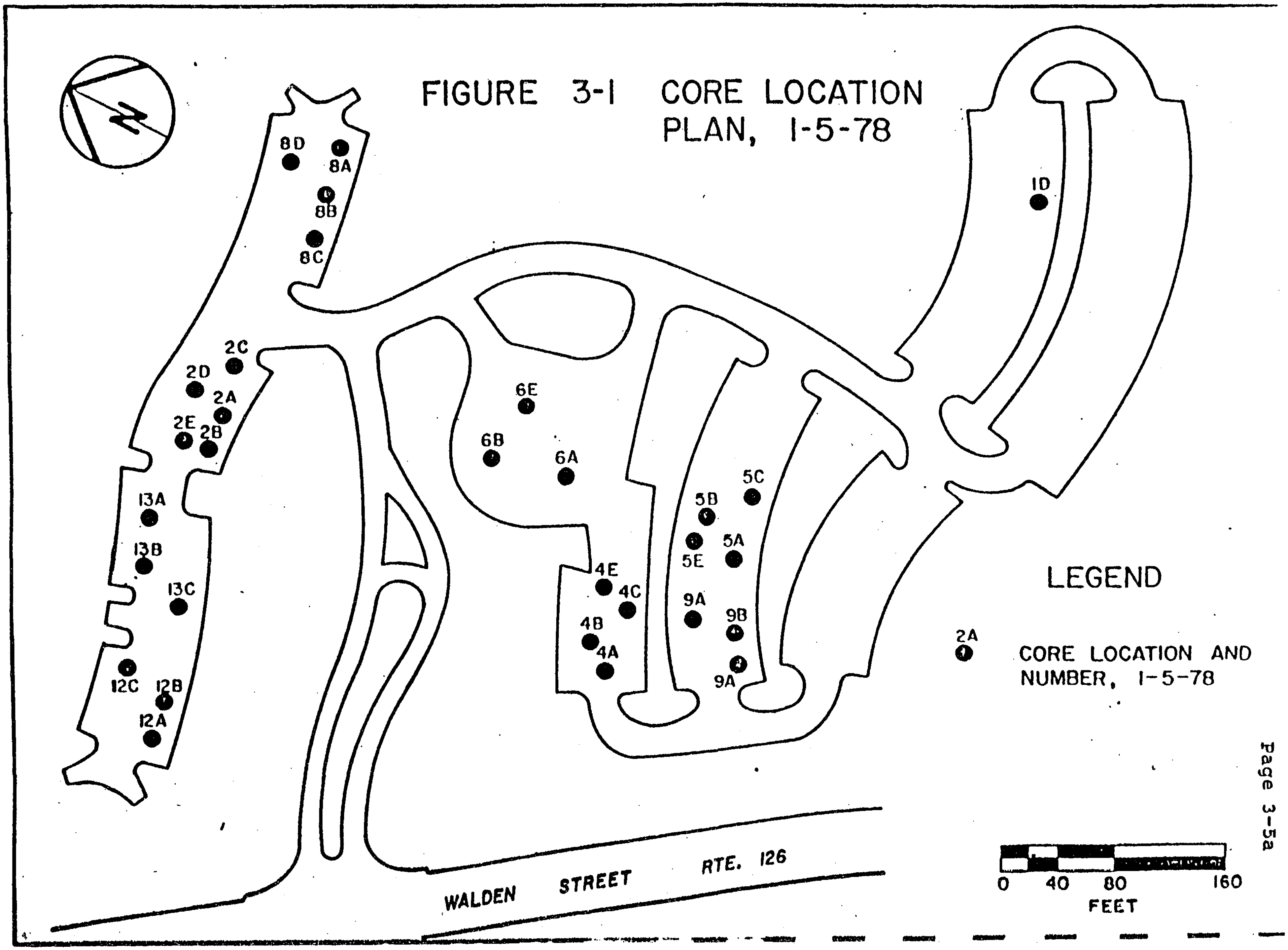
3.5.2 Initial Placement of the Asphalt Concrete

The initial placement of the asphalt concrete occured in October of 1977. Average air temperatures ranged from 54° F (12° C) to 68° F (20° C) and the average ground surface temperature was approximately 52° F (11° C) during the construction period. The specified placement temperature range for the asphalt concrete was 225° F to 250° F (107° C to 121° C). Actual placement temperatures varied considerably from this range (See Appendix E, Faulstich, 1979).

3.5.3 Problems

It was apparent from rough field testing that mix J was not as permeable as expected. Mix L also did not appear to be very permeable, however, this was expected. Mix K, on the other hand, appeared to be sufficiently permeable. Core samples were obtained in January of 1978 from mixes J, K, and L.. These core samples were tested for gradation by Briggs Engineering and Testing Company, Inc., an independent testing laboratory and the Mass. D.P.W. (See Appendix F, Faulstich, 1979). Several grab samples were also analyzed by the Mass. D.P.W.. These results are presented in Appendix F as samples G1, G3, G4, G5, G6, G7, and G8. Permeability tests on the core samples were performed by Northeastern University. The locations from which the core samples were taken are shown in Figure 3-1. The letter in each

FIGURE 3-1 CORE LOCATION PLAN, 1-5-78



core number indicates to which authority the cores went for testing. All "A" samples were tested for gradation by Briggs Engineering and Testing Company, Inc.. All "B", "C", and "D" samples were tested for permeability or density by Northeastern University. All "E" samples were tested for gradation by the Mass. D.P.W.. The gradation analyses confirmed that all three mixes did not meet the gradation specifications set forth in the job mix formulas (See Appendix F, Faulstich, 1979). From Appendix F it can be seen that each core sample gradation in general had a larger percent finer for each sieve than that specified by the job mix formulas. In other words each mix had a smaller amount retained on each sieve than specified, thus making each mix more dense than it was designed to be. Laboratory permeability tests on mix K confirmed the conclusion of the field testing that mix K was very permeable. The core sample gradation results for this mix were compared to the job mix formula for mix J. This plot appears in Figure 3-2 and shows that mix K as placed really conformed to mix J specifications. Since mix K as placed was very permeable, it was concluded that the job mix formula for mix class J was ideal from a permeability viewpoint. The Warren Brothers Company, Inc. was informed that the mixes did not meet the job specifications and therefore would not be accepted. After a 6 month delay during which several 8 in. square asphalt concrete samples were obtained and tested for gradation by the Warren Brothers Company, Inc., all confirming earlier results, a compromise on the reconstruction of the parking area was reached. Approximately two-thirds of mix J was removed and

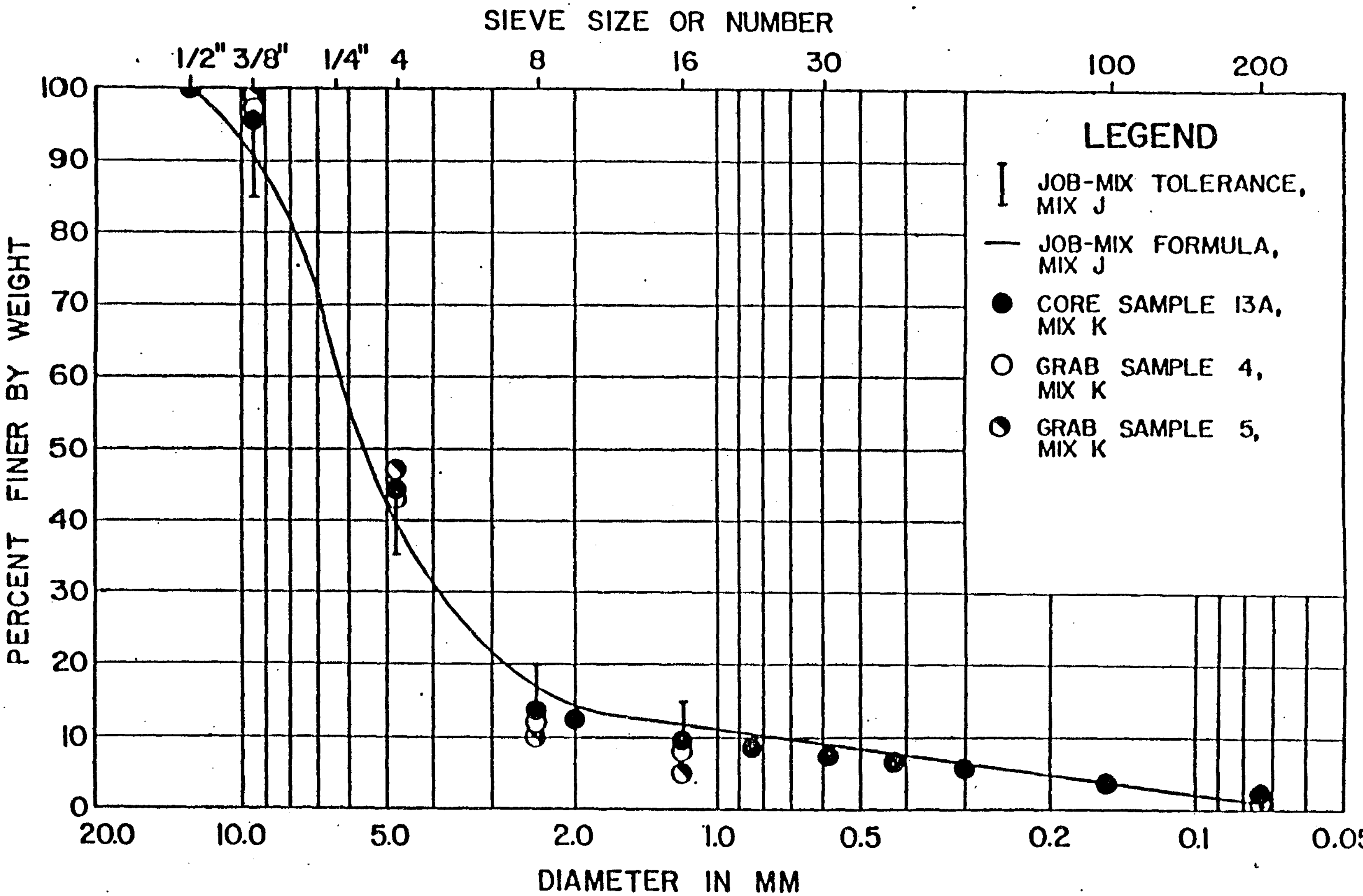


FIGURE 3-2 MIX K EXTRACTION RESULTS VS. MIX J JOB MIX FORMULA

replaced. Mix K, being very permeable despite not complying to the specifications, and mix L, judged to be of little interest, were not replaced. The sections of the parking area which were reconstructed are shown in Figure 3-3.

3.5.4 Reconstruction

Reconstruction of the parking area took place in August of 1978. The specified sections of unacceptable asphalt concrete were chipped out and the base course was regraded. Additional type p stone was placed to replace that lost in the removal of the asphalt concrete. The construction equipment and methods were similar to those used in the initial placement of the asphalt concrete. The mixing, placement, and compaction temperatures were lowered slightly from those used initially. Asphalt concrete arrival temperatures averaged 275° F (135° C) and placement temperatures averaged approximately 240° F (116° C). In order to prevent possible overcompaction, rolling was initiated after the asphalt concrete had cooled slightly. In general the asphalt concrete was rolled at a temperature of 180° F (82° C) or less (See Appendix E, Faulstich, 1979). An inspector was situated at the batch plant to insure that a mix meeting the job specifications was produced. The gradation analysis results listed in Table 3-1 were obtained at the batch plant. Due to misunderstanding, the aggregate gradation labeled "Loads 1-7" in Table 3-1 was placed at the project site despite being rejected by the inspector. After adjustments were made, the gradation labeled "Loads 8-" in Table 3-1 was approved by

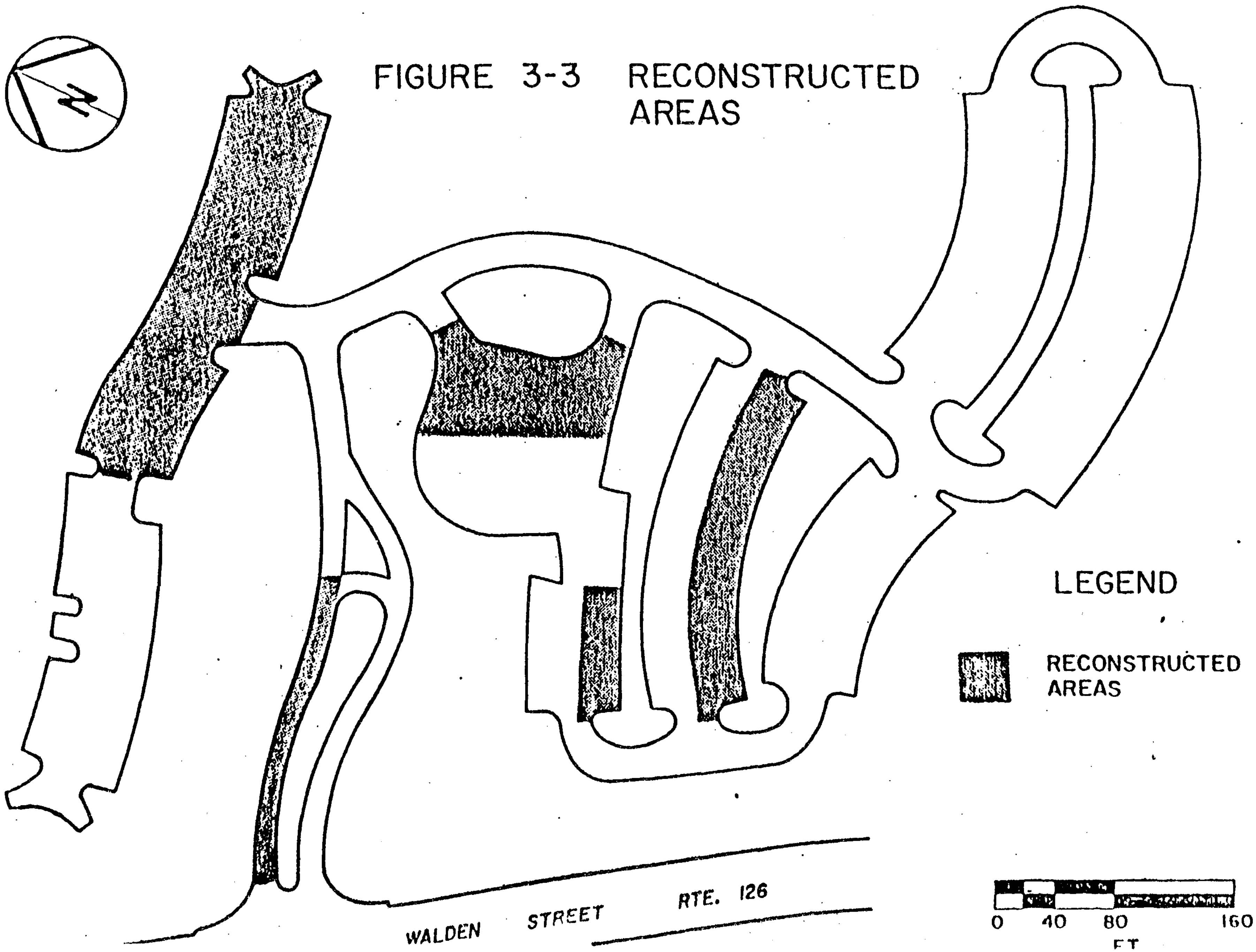


FIGURE 3-3 RECONSTRUCTED AREAS
LEGEND
RECONSTRUCTED AREAS
WALDEN STREET RTE. 126
0 40 80 160
FT

Table 3-1 was approved by the inspector. Consequently, two different mixes, one within the specifications for mix class J, and one outside of the specifications, were placed at the site during the reconstruction.

3.5.5 Modifications To The Original Design

Reconstruction of the parking area provided an opportunity for modifying the instrumentation system. An additional temperature probe, probe 5, was installed in the exit roadway. This probe provided ground temperature data to supplement that obtained from probe 2. Nine thermistors instead of the usual 10 were installed on probe 5, thus limiting the monitoring depth to approximately 4.5 feet.

TABLE 3-1

EXTRACTION RESULTS FROM BATCH PLANT, 8/78

Percent By Weight Passing Square-Opening Sieves

Sieve	Loads 1-7	Adjustments[a]		Loads 8-	JM[b]
1/2 in.	100	100	100	100	100
3/8 in.	90.4	93	89	92	90-95
No. 4	47	45	41	38	35-45
No. 8	19	23	15	19	15-20
No. 16	14	14	13	10	9-15
No. 200	0.7	1.5	0.6	0.4	0.5-1.5
Asphalt %	4.7	5.2	5.2	4.8	4.7-5.3

a. Not Placed

b. Job mix Formula

Water collection wells 6 and 8 were converted from infiltration collection wells to runoff collection wells by cutting the liner, sealing the inlets, installing a perforated cap, and placing a berm around the well. This conversion was done because these wells were located in low spots in the parking area. Previously, during periods of heavy precipitation, runoff from the other sections of the parking area drained to these low spots and eventually entered the infiltration collection well resulting in erroneous data.

Four infiltration collection wells, numbers 11, 12, 13, and 14, approximately 18 in. deep were installed in section A to collect additional water for water quality analysis. These wells were not used for water quantity purposes.

3.6 AS-BUILT PLANS AND SPECIFICATIONS

An as-built plan showing the mixes placed at the site is presented in Figure 3-4. The mix designations are defined as follows:

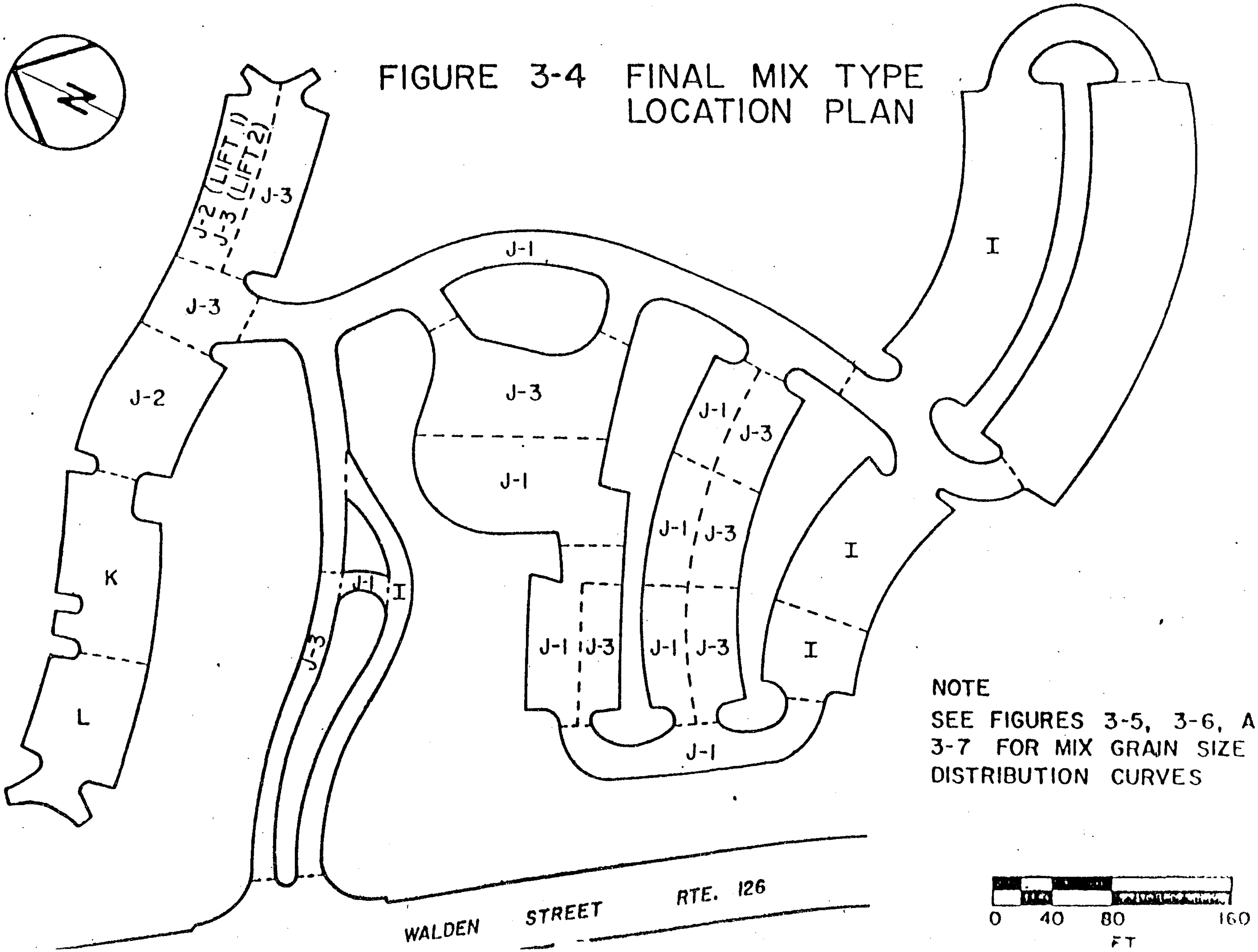
mix J-1: Original mix J placed in October of 1977 Did not meet mix class J specifications.

mix J-2: Mix having aggregate gradation labeled "Loads 1-7" in Table 3-1. Did not meet mix class J specifications, however it was closer to the specifications than mix J-1. Placed in August of 1978.

mix J-3: Mix having aggragate gradation labeled "Loads 8-" in Table 3-1. Did meet mix class J specifications. Placed in August of 1978.

mix K: Original mix K placed in October of 1977. Did not meet class K specifications but it did meet mix class J specifications.

FIGURE 3-4 FINAL MIX TYPE LOCATION PLAN



NOTE
SEE FIGURES 3-5, 3-6, AND 3-7 FOR MIX GRAIN SIZE DISTRIBUTION CURVES

mix L: Original mix L placed in October of 1977. Did not meet mix class L specifications.

mix I: Original mix I placed in October of 1977. Did meet mix class I specifications.

Gradation curves for these mixes are shown in Figures 3-5 through 3-7. Only the gradation results of core samples taken from areas of mix J-1 that were not reconstructed were used in determining the mean gradation curve for mix J-1 shown in Figure 3-6.

An as-built location plan for the water collection wells and temperature probes is shown in Figure 3-8. As-built specifications for the water collection wells and temperature probes are listed in Tables 3-2 and 3-3, respectively.

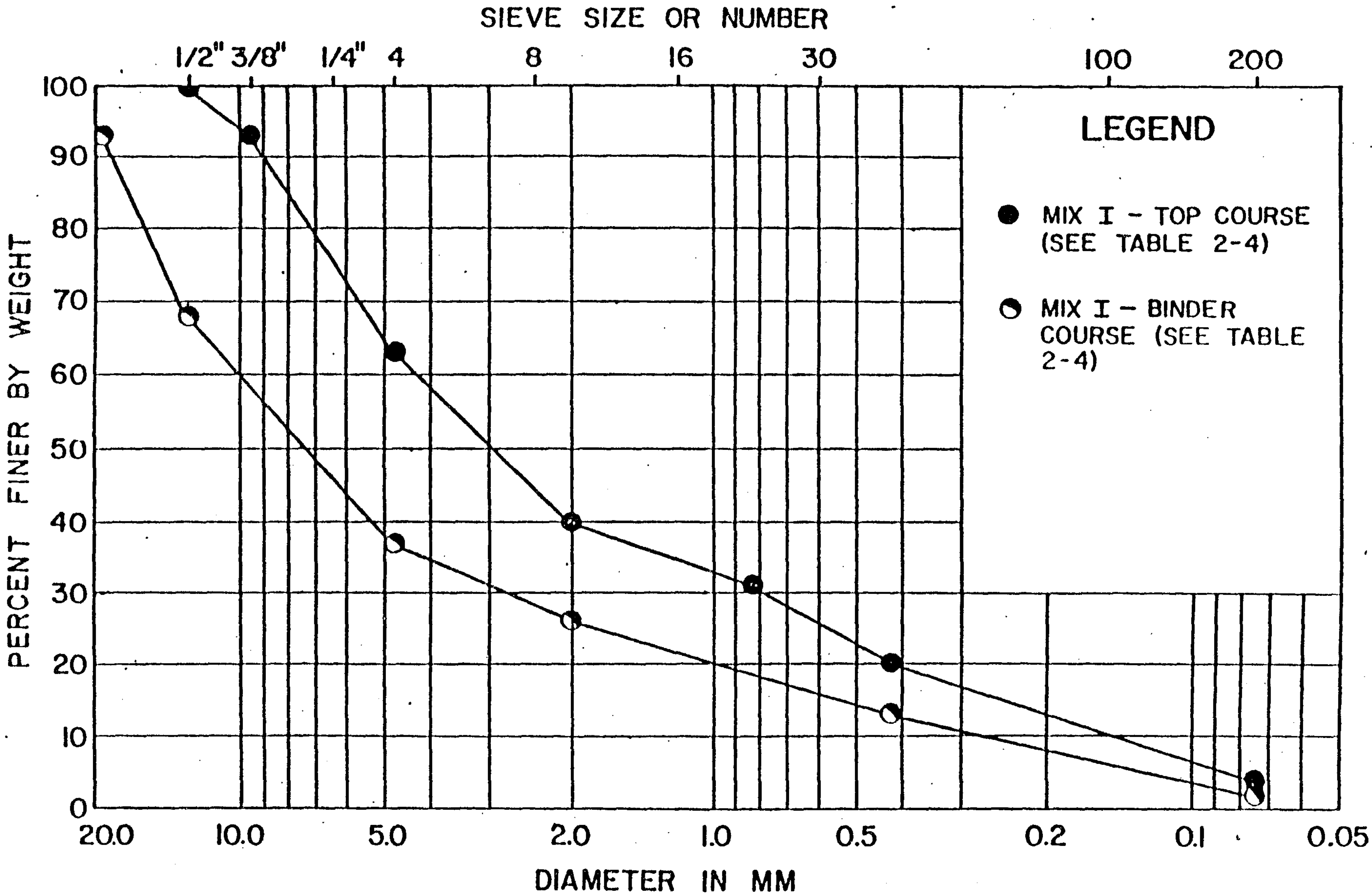


FIGURE 3-5 GRADATION CURVES FOR MIX I AS PLACED

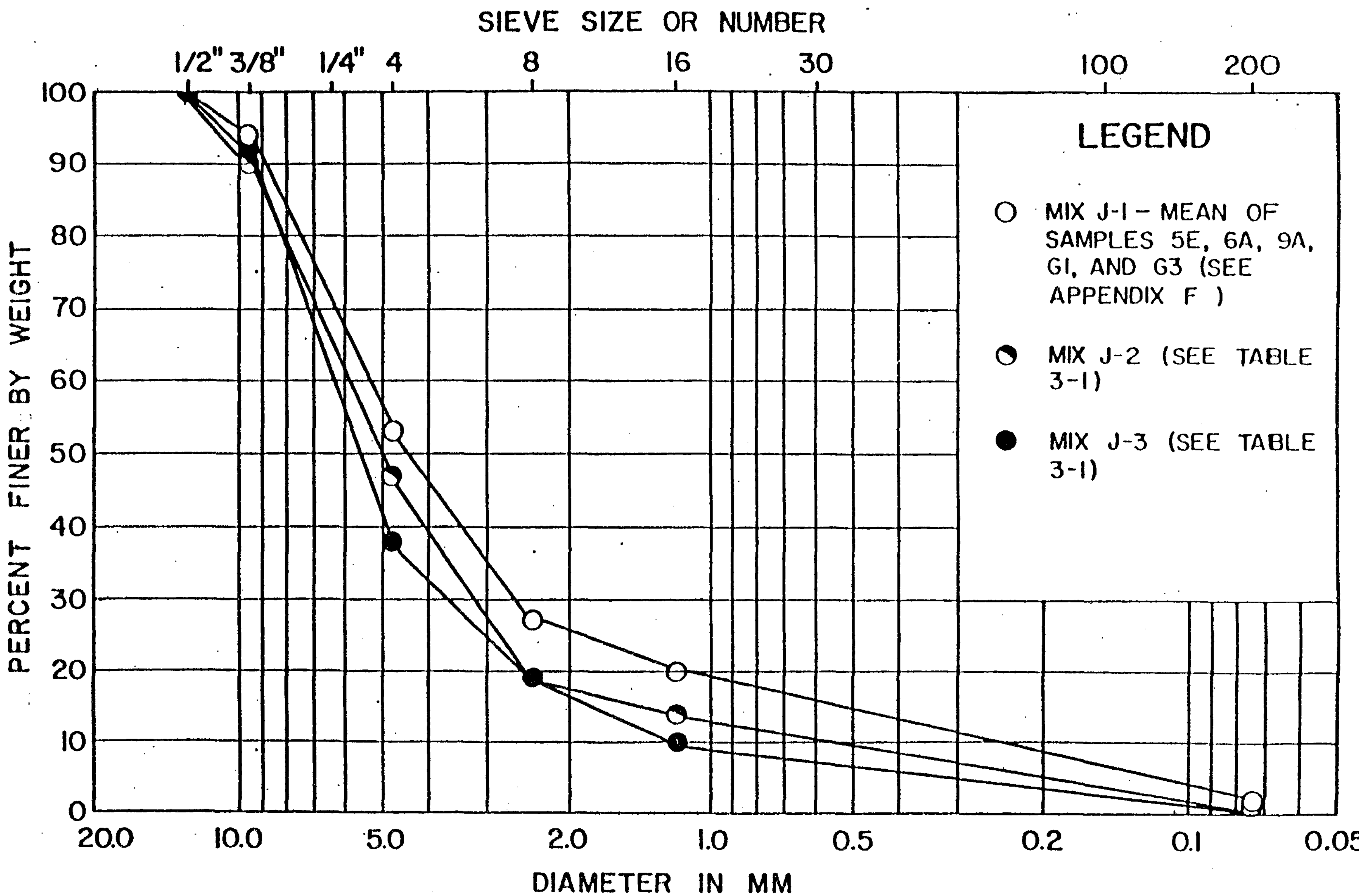


FIGURE 3-6 GRADATION CURVES FOR MIXES J-1, J-2, AND J-3 AS PLAC

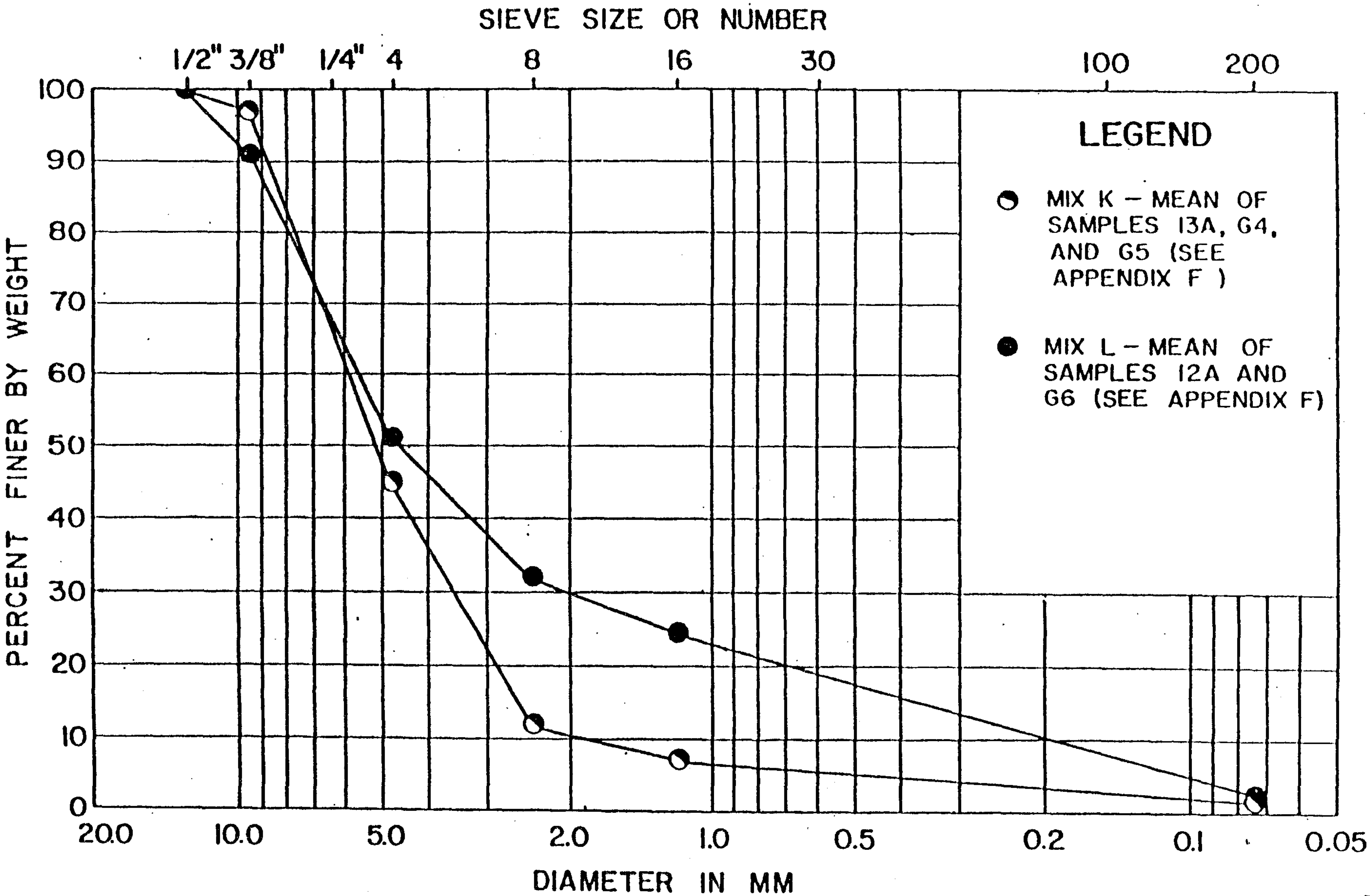


FIGURE 3-7 GRADATION CURVES FOR MIXES K AND L AS PLACED

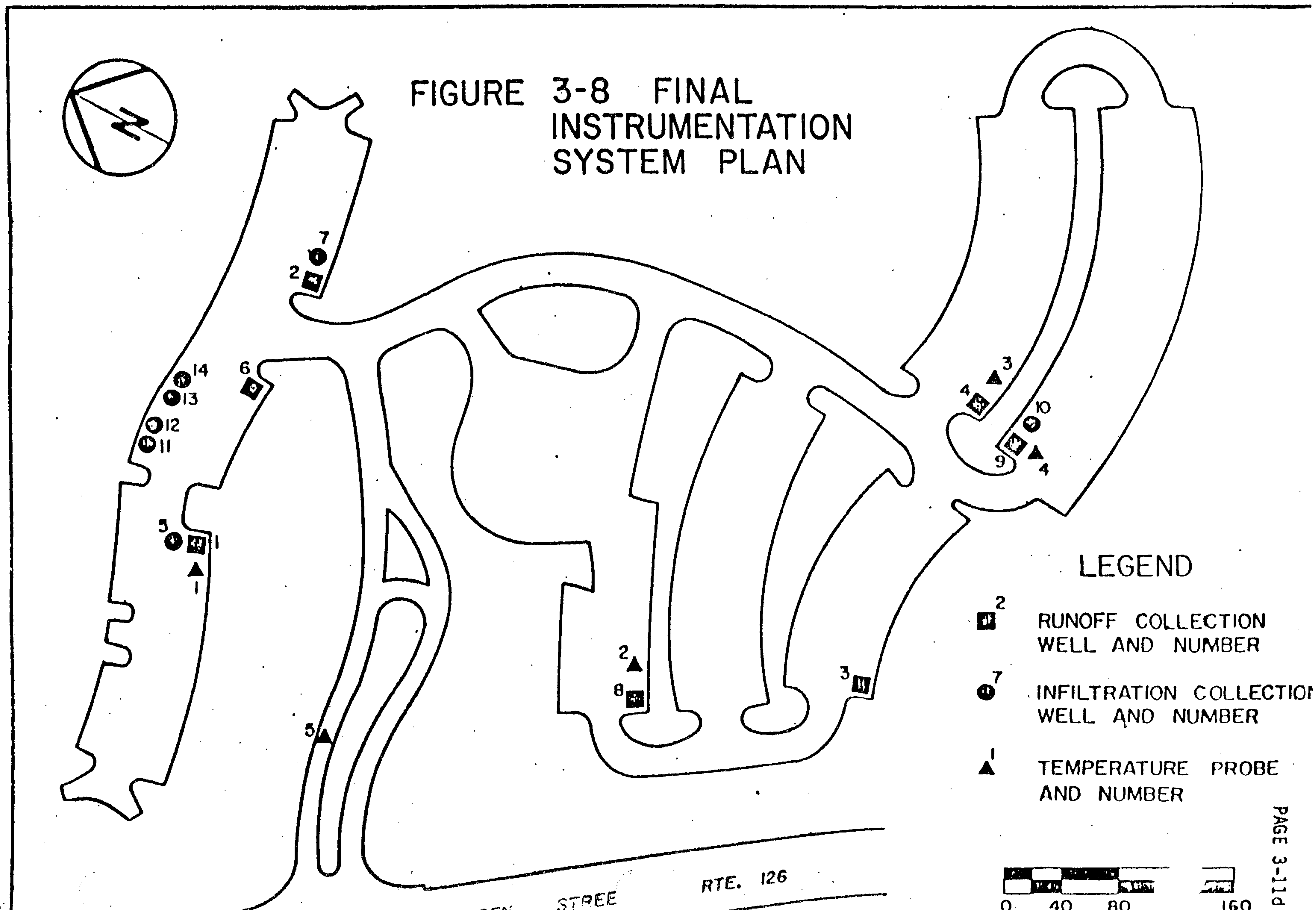


FIGURE 3-8 FINAL INSTRUMENTATION SYSTEM PLAN

TABLE 3-2

AS-BUILT SPECIFICATIONS FOR WATER COLLECTION WELLS

Water Collection Well	Well Type[a]	Approximate Capacity liters	Collection Area sq ft	Mix Type
1	RO	10	64	K
2	RO	10	64	J-3
3	RO	16	16	I
4	RO	10	16	I
5	I	11	19.7	K
6	RO	11	36	J-2
7	I	13	12.6	J-3
8	RO	11	100	J-3
9	RO	10	64	--
10	I	10	7.1	--
11	RG	3.5	0.7	--
12	RG	1.0	0.7	--
13	I	3.5	1.4	J-2
14	I	3.5	1.4	J-2
15	I	3.5	1.4	J-2
16	I	3.5	1.4	J-2

[a]RO = Runoff, I = Infiltration, and RG = Rain Gauge

TABLE 3-3

AS-BUILT SPECIFICATIONS FOR TEMPERATURE PROBES

Temperature Probe	Length ft	Number of Thermistors	Mix Type	Base Course Type
1	5	10	K	1
2	5	10	J-3	2
3	5	10	I	1
4	5	10	- -	1
5	4.5	9	J-3	1

3.7 OBSERVATIONS

As evidenced by the problems encountered in obtaining the specified mix composition, proper quality control is an essential component in the construction of a permeable pavement. Standard paving procedures and equipment were successfully used in constructing the pavement. A compaction temperature of approximately 180° F seemed to be the optimum temperature for the OGAC used at the test site. The laborers raking and spreading the asphalt concrete by hand were asked to evaluate the material's workability as compared to conventional asphalt concrete. The general agreement was that there was no difference in the workability of the two mix types. One worker felt that the quality of a cold joint formed with the OGAC was inferior to that of a cold joint formed with a conventional mix (See Figure B-20, Faulstich, 1979).

PART II MONITORING AND EVALUATION OF PERMEABLE PAVEMENT

CHAPTER 4

ASPHALT CONCRETE DENSITY

4.1 GENERAL

Current engineering practice utilizes asphalt concrete density as an indicator of compaction. However, the terms density and compaction are not synonymous. From a structural viewpoint, compaction of an asphalt concrete is much more important than its density. Since the degree of compaction of an asphalt concrete cannot be directly measured, density is used as an indicator of compaction. Specific roller types and weights and rolling temperatures are cited by states in their standard specifications. These compaction specifications have been shown through experience to result in acceptable compaction, measured as density. Zube (1967) states that "proper and adequate compaction is most essential in constructing a stable an compaction of asphalt concrete to be:

1. Temperature of the mix at the various phases of compaction, with emphasis on an adequately high temperature at the time of initial rolling

2. Air temperature as it affects mixing temperature

3. Gradation and shape characteristics of the aggregate

4. Asphalt cement content of the mix

5. Weight of the rollers and the number of passes.

In 1952 a research study was conducted on the physical characteristics of newly constructed asphalt concrete pavements (Graham et al., 1965). A portion of this study was devoted to establishing the inherent variability in the density of highway pavement surfaces. The conclusions of this study were:

1. Density does not vary significantly in the longitudinal direction (direction of paver)

2. The initial density varies significantly across the lane, being greatest in the center and least near the edge

3. Traffic increases the density and decreases the variability.

Asphalt concrete density data were obtained during both construction periods at the Walden Pond permeable pavement instalation for two purposes. First these measurements were taken to document the relative densities of the mixes placed, and second they were taken to indicate the applicability of utilizing a nuclear density meter to measure the density of OGAC.

4.2 ASPHALT CONCRETE DENSITY DETERMINATION METHODS

Two methods were utilized to determine the asphalt concrete density: a nuclear density meter and the Standard Method of Test for Specific Gravity of Compressed Bituminous Mixtures, ASTM Designation D 1188. The standard laboratory test was performed on core samples taken from the test site. The results of these tests were used as a check on the nuclear density meter data obtained during construction.

4.2.1 Nuclear Density Meter

A nuclear density-moisture asphalt meter, model C-75, manufactured by the Seaman Nuclear Corporation of Milwaukee, Wisconsin, was used for the field density determinations (See Figure B-24, Faulstich, 1979). A technician from Warren Brothers Company, Inc. operated the meter. This meter used an air gap procedure for the measurement of surface density by gamma ray backscatter. The principle involved is that the denser the material, the more radiation it will absorb. Since the meter reads the backscatter, or reflected radiation, the lower the meter reading the higher the density. The meter reading of the backscatter is actually a function of the material's density and its chemical content. The air gap method of calibration was developed to isolate the effect of the chemical composition, resulting in a corrected meter reading that is a function of the material density only. For a complete description of the air gap method see " Air Gap Procedure for the Measurement of Surface Density by Gamma Ray Backscatter Technique," Highway Research Board Circular No. 44.

Nuclear density meters offer several advantages over conventional density measurement methods. These advantages include speed and ease of measurement, reproducibility, and nondestructiveness (Dunn and McDougal, 1970). According to the Seaman Nuclear Corporation, correlations within one pcf of the conventional specific gravity test performed on core samples are typical for nuclear density meters on the dense asphalt concrete. However, it was reported that nuclear density meter results do not correlate well with conventional results on open-graded mixes having a significant amount of surface air voids (Brown, 1966). The Seaman Nuclear Corporation recommends the use of native fines without asphalt cement to prevent radiation leakage along the surface of an open-graded mix. This recommendation was not followed in the actual density measurements.

4.2.2 Specific Gravity Test

Bulk specific gravity tests were performed by Northeastern

University according to ASTM designation D 1188 on core samples obtained from the test site (See section 3.5.3). One specific gravity determination on a core sample was made by the Mass. D.P.W. according to the American Association of State Highway and Transportation Officials (AASHTO) designation T 166. The results obtained from these tests were compared to the results obtained in the field by the nuclear density meter.

4.3 DENSITY DATA

Field density data was collected during both construction periods (October of 1977 and August of 1978). The asphalt concrete was rolled until a smooth surface (free from ridges) was obtained. The roller passes necessary to reach this condition were not counted, and therfore connot be considered uniform for every test area. Laboratory specific gravity determinations were made on samples taken from mixes placed in October of 1977 only.

4.3.1 Field Density Data

The field asphalt concrete density data collected at the field site during both construction periods is presented in appendix G (Faulstich, 1979). These data are summarized in Table 4-1. Mix J-1 placed in October of 1977 had average densities ranging from 132.7 pcf to 139.4 pcf. No evaluation of the effect of the base course type or asphalt concrete thickness could be formulated since the compactive effort applied to each test section was not uniform. No finish density data was

TABLE 4-1

FINISH ASPHALT CONCRETE DENSITY DATA SUMMARY

Date	Mix Type	Mix Thickness in.	Base Course Type	Average Density pcf	Standard Deviation
10/77	J-1	1.5	1	136.4	1.3
	J-1	2.5	1	132.7	5.1
	J-1	4.0	1	134.8	3.5
	J-1	2.5	2	134.4	2.4
	J-1	2.5	3	134.4	2.3
	J-1	1.5	4	135.6	4.6
	J-1	2.5	4	139.2	4.0
	J-1	4.0	4	139.4	3.2
8/78	J-2	2.5	1	130.1	2.4
	J-3/J-2	4.0	1	128.8	1.5
	J-3	4.0	1	130.1	2.4
	J-3	1.5	1	125.2	0.8
	J-3	2.5	2	124.8	3.2
	J-3	2.5	3	124.4	3.9
	J-3	1.5	4	120.4	1.0
	J-3	2.5	4	117.9	2.0
	J-3	4.0	4	124.6	1.2

collected for mixes K, L, and I. Mixes J-2 and J-3, placed in August of 1978 had average densities ranging from 117.9 pcf to 130.1 pcf. The average density range of these mixes was approximately 10 percent lower than the average density range of mix J-1. Mixes J-2 and J-3 were allowed to cool slightly before rolling, thus resulting in lower final densities than would have occured if the asphalt concrete had been rolled soon after placement as was the case with mix J-1. Also, the fact that mixes J-2 and J-3 were more open-graded than mix J-1 could have contributed to the density difference.

4.3.2 Laboratory Specific Gravity Data

The specific gravity data obtained through laboratory testing of asphalt concrete core samples are presented in Table 4-2. From Table 4-2 it is apparent that mix J-1 is less dense than both mixes L and I, as expected. Mix I is in the range expected for a conventional dense mix: 140-145 pcf. Mix L was to have an aggregate gradation similar to a dense mix. The density data obtained through laboratory testing verifies the similarity between mix L and mix I.

TABLE 4-2

LABORATORY BULK SPECIFIC GRAVITY TEST RESULTS

Core No.	Mix Type	Mix Thickness in.	Base Course type	Specific Gravity	Density pcf
1D	I	2.5	1	2.32	144.5
2D	J-1	2.5	1	2.19	136.3
8B	J-1	4.0	1	2.18	135.7
12B	L	2.5	1	2.26	141.2
Mass. D.P.W.	J-1	2.5	1	2.28	142.3

4.3.3 Correlation of Laboratory and Field Density Data

The density data for the two core samples of mix J-1 are listed in Table 4-3 with the average field density for the test section from which the core samples were taken.

TABLE 4-3

LABORATORY AND FIELD ASPHALT CONCRETE DENSITY DATA

Mix Type	Mix Thickness in.	Base Course Type	Core Density pcf	Field Density pcf	Standard Deviation (Field)
J-1	2.5	1	136.3	132.7	5.1
J-1	4.0	1	135.7	134.8	3.5

Based on this limited data, it appears that the nuclear density meter slightly underestimates the density of the OGAC, most likely because of radiation leakage at the irregular pavement surface. However, this slight underestimation is not considered to be significant.

4.4 CONCLUSIONS

Because mix J-1 did not meet the gradation specifications for mix class J it is really of no concern other than for indicating the applicability of the nuclear density meter. A meaningful evaluation of the applicability of using nuclear density meters to measure the density of OGAC would require a statistical analysis of numerous core sample densities and field asphalt concrete densities. In general, it can be safely stated that the densities of the open-graded mixes J-2 and J-3 are significantly lower than the density of a conventional dense mix such as mix I. This was expected because of the relatively high void content of mixes J-2 and J-3. As mentioned in section 4.1, the degree of compaction, rather than the magnitude of the density is most important for good pavement performance.

CHAPTER 5

GROUND AND AIR TEMPERATURE

5.1 GENERAL

Ground temperature indicated by the thermistors installed at the test site were recorded weekly or biweekly from 11/77 to 5/79. The air temperature at the observation time was also recorded. Other weather data consisting of the maximum and minimum daily air temperatures and the depth of snow on the ground were obtained from a weather station in Bedford, Massachusetts. This station, operated by Weather Services Corporation, is approximately three miles northwest of Walden Pond and is listed in Local Climatological Data as "the station, Bedford."

The main objective in monitoring air and ground temperatures was to determine the temperature distribution beneath the various mixes during the winter months and the influence of several factors including air temperature on that distribution. Specifically, the depth of frost penetration in the various test sections was of main concern. Sargious (1975,