Prefabricated Highway Edge Drains

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Prefabricated highway edge drains, made from a polymer drainage core wrapped in a geotextile filter, are being used regularly by transportation engineers for both new and retrofitted roadway systems. Although these drains are acknowledged to be low in cost and rapidly placed, their performance is still being evaluated by long-term monitoring. One reason for this challenge is that their design is essentially empirical. What is intended to be a rational design procedure is presented that can be used in a variety of geographic locations and for a variety of edge-drain products.

The need for proper drainage of paved roadway systems is being rapidly rediscovered by many transportation engineers. Cedergren has long championed this concept and continues to remind engineers of its importance (*1*). Once the pavement has been drained, however, its flow must be intercepted at the edge of the pavement, gravitationally transported parallel to the pavement, and then discharged at intervals of 100 to 500 ft into drainage swales or interceptor pipelines. The edge drains have traditionally been perforated pipes in a gravel envelope, with a sand layer providing a filter transition to the adjacent soil. In Pennsylvania this type of edge drain is currently bid at approximately $6 per linear foot.

In contrast to the edge drains made of perforated pipe and natural soil, prefabricated edge drains are made in the factory and consist of a polymer drainage core wrapped in a geotextile filter. Not only are the basic materials less expensive, but their installation is automated to the point where production rates are hundreds of feet per hour. This is reflected in their current bid prices of less than $2.50 per linear foot. Besides this obvious cost benefit, this edge drain has other desirable features: less excavated soil to be removed, less weight on the subgrade, no need for quarried materials, and potential use of reclaimed or recycled plastic materials, or both.

OVERVIEW

Since the introduction of prefabricated edge drains by the Monsanto Company in the early 1980s, a wide range of products has appeared and is being marketed to the user-owner community. Table 1 is a compilation of seven of these products produced by six different companies. Although the core polymers are currently made from polyethylene, the variations thereafter are considerable. The shape of the built-up cores is seen to be very different, and their thicknesses vary from 0.80 to 1.60 in. These various shapes and thicknesses result in very different compressive strengths and planar flow rates. The only common feature of the geotextile filter is that all appear to be made of nonwoven fabrics. Thereafter, the geotextiles vary as to polymer type (polypropylene or polyester), processing (needlepunched or melt-bonded), and post treatment (some are burnished).

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DESIGN CONSIDERATIONS

The various considerations in the design of a prefabricated edge drain focus separately on the drainage core and on the geotextile filter. From the schematic diagram in Figure 1 it can be seen that (a) the core must be capable of sustaining a certain amount of stress, and (b) it must convey a required flow rate. The geotextile must be capable of (c) passing this flow, (d) retaining the adjacent soil, and (e) sustaining the normal stress between core protrusion locations. These five aspects of design will be addressed sequentially.

Core Compressive Strength

A review of the technical literature (*2*, pp. 73–82) found that the maximum vertical stress due to a vibratory base plate compactor is 12 lbf/in^2. With a K_0 of 0.43, a horizontal pressure on the edge drain of 743 lbf/ft^2 is obtained. This pressure value, however, is far less than that in the situation shown in Figure 2. If a truck parks on the shoulder directly over the edge drain (clearly a worst-case situation), a Boussinesq analysis can be performed (*3,4*) that shows the horizontal stress to be 3,020 lbf/ft^2 (see Figure 2). Note that this value represents a factor of safety equal to 1. Some very compelling reasons for increasing this value are the following:

- Overweight vehicles,
- Impact loads (e.g., bumpy shoulders),
- Long-term creep loads (e.g., overnight parking or truck breakdowns),
- Stresses applied at various angles (see paper by Frobel elsewhere in this Record),
- Variation in edge-drain product strength,
- Effect of moisture on the product's performance,
- Effect of polymer aging on the product's performance, and
- Differences in product strength evaluation from the real-life situation by particular test methods.

The compressive strength test for product evaluation should be on a section of edge drain of full width (usually 18 or 12 in.) by approximately 6 in. long cut so that a reproducible pattern of protrusions exists (*5*). Using this type of test procedure, the cumulative factor of safety for the uncertainties mentioned above should be at least 3. Thus the necessary

TABLE 1 PROPERTIES OF COMMERCIALLY AVAILABLE HIGHWAY EDGE DRAINS

No.	Company	Trademark	Core Characteristics			Geotextile Polymer & Type
			Material	Shape	Thickness (in.)	
1	A.C. F. Inc.	Drain-It	PE	double cuspated	1.00	PP nonwoven needle-punched
2	Advanced Drainage Systems	AdvanEDGE	HDPE	corrugated with columns	1.60	PP nonwoven melt bonded
3	American Wick Drain Co.	Akwadrain 125	HDPE	double cuspated	1.25	PP nonwoven needle-punched
4	Contech Construction Products Inc.	Stripdrain 100	HDPE	tapered column	1.00	PET nonwoven needle-punched
5	Monsanto	Hydraway 2000	PE	straight column	1.00	PP nonwoven needle-punched
6	Pro Drain Systems Inc.	PDS 20	HDPE	double cuspated	0.80	PP nonwoven needle-punched
		PDS 30	HDPE	tapered column	1.20	PP nonwoven needle-punched

From "Product Directory", Geotechnical Fabrics Report, Dec., 1989.

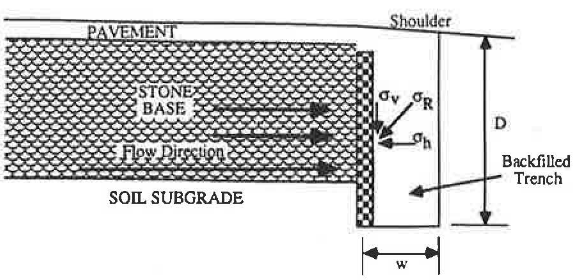

FIGURE 1 Generalized performance concept of a prefabricated highway edge drain.

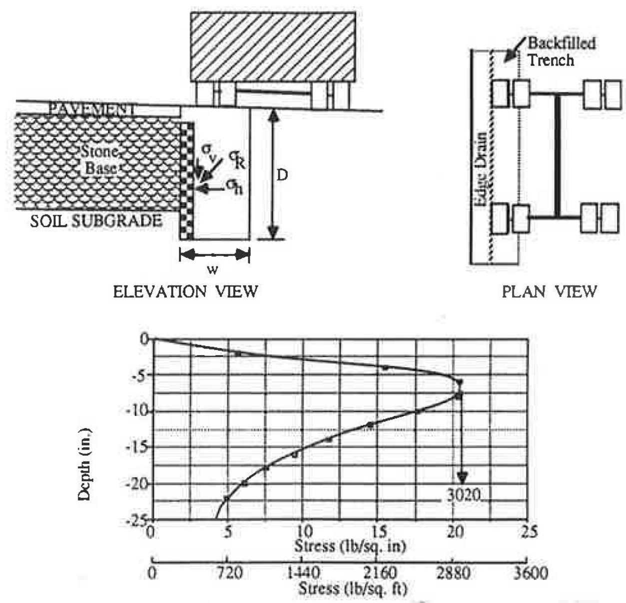

FIGURE 2 Worst-case design loading with resulting Boussinesq analysis for horizontal pressure on edge drain.

ultimate test strength from normally applied loads will be as follows:

$3.0 = \sigma_{test}/3{,}215$

$\sigma_{test} = 9{,}600\ \text{lbf/ft}^2\ (67\ \text{lbf/in.}^2)$

Core Required Flow Rate

To obtain the value of required flow rate for the transport of the water entering into the core, the FHWA (*6*) 1-hr–1-year frequency precipitation rate is used as the design precipitation rate. The design infiltration rate through the pavement used in the guidelines is from one-third to two-thirds of the design precipitation rate, because normally it is expected that less than 100 percent of the water falling on a pavement will enter through the pavement surface. In this regard, the guidelines suggest that the design precipitation rate (1-hr–1-year frequency rate) be multiplied by a coefficient between 0.50 and 0.67 for portland cement concrete pavements and 0.33 to 0.50 for asphalt concrete pavements [see work by Cedergren (*7*, pp. 104–142) for addition details].

The 1-hr–1-year frequency precipitation rates give a maximum value of 1.2 in./hr at a location in the southeast corner of Pennsylvania near Philadelphia. Using this information, one can now calculate the required flow rate to the edge drain. Assumed in the calculation is a two-lane (each 12 ft wide) asphalt concrete pavement that has an infiltration coefficient of 0.42 with an edge drain on one side; W = 24 ft. Also taken in the design are drainage outlets at 300-ft centers with a release factor of water in the stone base course of ⅓.

The required formula is

$$q_{\text{design}} = c i_R (WL) f_R$$

where

q_{design} = design flow rate per foot of pavement length, that is, between drainage outlets;
c = pavement infiltration coefficient;
i_R = rainfall intensity;
W = width of pavement;
L = length between drainage outlets; and
f_R = release factor for water in the stone base.

Substituting the foregoing values into this formula gives

$$q_{\text{design}} = (0.42)(1.2)(W \times L)(1/3)(1/12)(1/60)(7.48)$$
$$= 0.00175WL$$

$q_{design} = 0.00175WL$ gal/min/ft width/ft outlet spacing

and for $W = 24$ ft and $L = 300$ ft, $q_{design} = 12.6$ gal/min for the designed outlet spacing.

In a similar manner, a design table can be generated in which the major variables are the pavement drainage width (W), the spacing between drainage outlets (L), and the release factor (f_R). Table 2 has been generated in accordance with these variables. Here it is seen that as pavement width, outlet spacing, and release factors increase, the design flow rate increases proportionately.

The foregoing analysis is essential in order to properly size edge-drain cores to handle their design flow rates. In the procedure, the major uncertainty is the release factor for the retention time that the water is in the stone base. It is believed that this value varies as follows:

- Well-graded base courses, in service ≥ 5 years: $f_R = 1/4$,
- Well-graded base courses, in service < 5 years: $f_R = 1/3$,
- Open-graded base courses, in service ≥ 5 years: $f_R = 1/3$,
- Open-graded base courses, in service < 5 years: $f_R = 1/2$.

This range of release values was used in preparing Table 2.

As with all design-by-function concepts, a factor-of-safety process is used to arrive at a required flow rate for the edge drain from the factor of safety = 1 design value.

$$FS = q_{test}/q_{design}$$

Using a factor of safety of 1.2 against uncertainties in the design, product variability, intrusion of the geotextile filter, and variations of the test with respect to the actual situation results in a required flow rate for the example problem of the following:

TABLE 2 DESIGN TABLE OF REQUIRED FLOW RATES FOR HIGHWAY EDGE DRAINS UNDER CONDITIONS OF VARYING PAYMENT WIDTH, OUTLET SPACING AND RELEASE FACTORS

Pavement Drainage Width W (ft.)	Outlet Drainage Spacing L (ft.)	Design Flow Rate (gal/min-ft.) for Release Factors of 0.25	0.33	0.50
6	200	1.6	2.1	3.1
6	400	3.1	4.1	6.3
6	600	4.7	6.2	9.4
6	800	6.3	8.3	12.6
6	1000	7.9	10.4	15.7
12	200	3.1	4.1	6.3
12	400	6.3	8.3	12.6
12	600	9.4	12.4	18.8
12	800	12.6	16.6	25.1
12	1000	15.7	20.7	31.4
18	200	4.7	6.2	9.4
18	400	9.4	12.4	18.8
18	600	14.1	18.7	28.3
18	800	18.8	24.9	37.7
18	1000	23.6	31.1	47.1
24	200	6.3	8.3	12.6
24	400	12.6	16.6	25.1
24	600	18.8	24.9	37.7
24	800	25.1	33.2	50.3
24	1000	31.4	41.5	62.8

Note: Table results are based on maximum rainfall intensity in Pennsylvania (1.2 in./hr.) and average asphalt pavement surface conditions (infiltration coefficient of 0.42)

$$1.2 = q_{test}/12.6$$
$$q_{test} = 15 \text{ gal/min-ft}$$

The commonly used laboratory test for highway edge drains is ASTM D-4716 [Constant Head Hydraulic Transmissivity (In-Plane Flow) of Geotextiles and Geotextile Related Products]. Recommended conditions for performing the test are as follows:

- 1,500 lbf/ft^2 normal pressure,
- Core material placed between solid plates,
- 15 min of dwell time pressure,
- 15 min of test time for flow, and
- Hydraulic gradient of 0.10.

Geotextile Filter Flow Capacity

The geotextile must be able to accept the flow coming from the stone base course beneath the pavement system. The design is precisely the same as for the core except that it is not cumulative along the length of the pavement; that is, each running foot of geotextile is needed for only the associated running foot of edge drain. The other consideration is that some of the geotextile will be blocked from accepting flow because of the bonding or blocking of the core's protrusions. This consideration is easily handled by a geometric factor, f_B, which is product specific.

With the same design example as that used in the foregoing section on core flow rate—a 24-ft-wide road in Pennsylvania—

$$q_{design} = 0.00175W \text{ gal/min-ft}$$

and for $W = 24$ ft,

$$q_{design} = 0.042 \text{ gal/min-ft}$$
$$= 0.00562 \text{ ft}^3\text{/min-ft}$$

Using Darcy's law with an edge drain 1.5 ft high by 1.0 ft long and a core blockage factor f_B as described above,

$$q = kiA$$

$$\frac{q_{design}}{A} = k\frac{\Delta h_{ave}}{t}$$

$$\frac{0.00562}{1 \times 1 \times (1 - f_B)} = k\frac{0.75}{t}$$

$$\frac{k}{t} = \Psi_{design} - \frac{0.00749}{(1 - f_B)}$$

$$\Psi_{design} = \frac{0.00749}{(1 - f_B)} \text{ (min}^{-1}\text{)}$$

where

Ψ = permittivity,
k = coefficient of permeability,
t = thickness, and
f_B = blockage factor (20 to 95 percent).

For $f_B = 0.95$ (worst case),

$$\Psi_{\text{design}} = \frac{0.00749}{(1 - 0.95)}$$
$$= 0.150\ (\text{min}^{-1})$$

and

$$k_{\text{design}} = \Psi_{\text{design}} t$$

For a geotextile thickness of 0.060 in. (=0.0050 ft),

$$k_{\text{design}} = (0.150)(5.0 \times 10^{-3})$$
$$= 0.75 \times 10^{-3}\ \text{ft/min}$$
$$k_{\text{design}} = 0.38 \times 10^{-3}\ \text{cm/sec}$$

Using a worst-case cumulative factor of safety of 10 for clogging, blinding, and other considerations (*4*), $k_{\text{reqd}} \geq 0.0038$ cm/sec. This required permeability is satisfied by many geotextiles. (Values for geotextiles commonly used in filtration applications range from 0.1 to 0.001 cm/sec.) This statement can also be extended for any f_B-value of existing commercial products.

Geotextile Opening Size (0_{95})

The opening size of a geotextile is characterized by its apparent opening size (AOS), but in calculations it is preferable to work with the 0_{95} value corresponding to the nearest AOS sieve size. A review of soil retention criteria in the literature finds that all of them are for retention of fine sands and larger particle sizes (*4*). It is believed that these are not the soils that are troublesome for highway edge drains. Loss of soil through the geotextile and into the drainage core becomes a problem with the fine-soil fraction consisting of fine silts and dispersive clays. For example, the gradation of soil found inside the core of a completely clogged prefabricated edge drain in central Pennsylvania consisted of the following:

Sieve No.	*Percent Fines*
100 (0.25 mm)	99
200 (0.074 mm)	95
400 (0.037 mm)	85
Clay size (0.002 mm)	15

Whenever backfilling against the geotextile is not tight, such soil loss can occur. Thus those geotextiles with low 0_{95} values become very desirable for edge-drain filters. Note, however, that most geotextiles are in the No. 40 to No. 100 AOS ranges, which is illustrated in the following list of types of soil retained on different AOS sieve sizes:

AOS (sieve no.)	*0_{95} (mm)*	*Type of Soil Retained*
40	0.42	Lower medium sand
60	0.25	Upper fine sand
70	0.21	Middle fine sand
100	0.15	Lower fine sand
200	0.074	Upper-range silt
400	0.037	Middle-range silt

Thus, to prevent silts from moving through the geotextile, one must have a No. 200 or No. 400 AOS sieve size. Furthermore, dispersive clays would require still finer opening sizes, which are simply not available.

On the basis of the foregoing discussion and limitations, an AOS sieve size of 100 or higher is recommended, which corresponds to an opening size of 0.15 mm or less. It should be recognized, however, that very few commercially lightweight geotextiles have such a small opening size and a different, or heavier, geotextile than currently being supplied may be required.

Geotextile Strength

The geotextile filter has a secondary, but still very important, role in that it must support the backfill soil from one core protrusion to the next. In other words, it cannot collapse into the core and block the flow. The distance between protrusions is typically 0.5 to 2.0 in., with the core protrusion itself having a diameter of 0.1 to 1.0 in. Note that two strength phenomena are occurring simultaneously. First, the backfill soil is putting stress on the geotextile, thereby pushing it into the core, and the geotextile is going into some type of complicated mode of tension. Second, this action produces a puncture stress in the geotextile around the protruding core tip. Initial laboratory test results of hydrostatically stressing the geotextile against the core show that the puncture mode is more critical than the tensile mode and the problem formulation is developed accordingly. The design puncture strength is simply

$$P_{\text{design}} = \sigma_R(A_T - A_S)$$

where

P_{design} = design puncture strength,
σ_R = maximum stress imposed on the geotextile by the backfill,
A_T = geotextile area between centers of adjacent core protrusions, and
A_S = geotextile area over the individual core protrusion supports.

Using a maximum stress value of σ_R = 3,200 lbf/ft² (or 22.2 lbf/in.²) from earlier in the paper and a variety of A_T- and A_S-values for the current range of commercially available edge-drain products gives results shown in Table 3. Here design puncture strengths are seen to vary from 4.4 to 83.7 lb, depending on the geometric configuration of the edge drain.

Because many existing edge drains have 1.0-in. protrusion spacings and 0.3-in. protrusion diameters, these values will be used to select the design puncture strength, 19.1 lb (Table 3). Now the required puncture strength value uses a factor of safety for design uncertainties, load variations, product variation, long-term considerations, and so on, as follows:

$$FS = P_{\text{reqd}}/P_{\text{design}}$$
$$3.0 = P_{\text{reqd}}/19.1$$
$$P_{\text{reqd}} = 57.3\ \text{lb (use 60 lb)}$$

P_{reqd} can be taken directly from the results of ASTM D-3787 (Puncture Strength of Geotextiles), which uses a 5⁄16-in. plunger (0.31 in.) and models the edge-drain situation from this paper quite nicely; that is, scale effects are believed to be minimal.

TABLE 3 DESIGN PUNCTURE STRENGTH VALUES FOR GEOTEXTILES ON EDGE DRAINS

Spacing (in.)	Total Area (sq. in.)	Support Diameter (in.)	Support Area (sq. in.)	Design Puncture Strength (lbs.)
0.5	0.25	0.1	0.01	5.0
0.5	0.25	0.2	0.04	4.4
1.0	1.00	0.1	0.01	20.8
1.0	1.00	0.2	0.04	20.1
1.0	1.00	0.3	0.09	19.1
1.0	1.00	0.4	0.16	17.6
1.5	2.25	0.1	0.01	47.0
1.5	2.25	0.2	0.04	46.3
1.5	2.25	0.3	0.09	45.3
1.5	2.25	0.4	0.16	43.8
1.5	2.25	0.5	0.25	41.9
1.5	2.25	0.6	0.36	39.9
1.5	2.25	0.7	0.49	36.9
2.0	4.00	0.1	0.01	83.7
2.0	4.00	0.2	0.04	83.1
2.0	4.00	0.3	0.09	82.0
2.0	4.00	0.4	0.16	80.5
2.0	4.00	0.5	0.25	78.6
2.0	4.00	0.6	0.36	76.3
2.0	4.00	0.7	0.49	73.6
2.0	4.00	0.8	0.64	70.5
2.0	4.00	0.9	0.81	66.9
2.0	4.00	1.0	1.00	62.9

Other geotextile mechanical properties such as grab strength, burst strength, and tear strength are obviously important but not easily determined by a specific design method. Thus values recommended for nonwoven geotextiles are controlled by the 60-lb required puncture strength just calculated. These are taken from the recently (July 1989) adopted survivability table of Task Force 25, Joint Committee of AASHTO, Associated General Contractors (AGC), and American Road and Transportation Builders' Association (ARTBA):

Degree of Surviv-ability	*Puncture Strength (lb)*		*Grab Strength (lb)*		*Trapezoidal Tear Strength (lb)*	
	Woven	*Non-woven*	*Woven*	*Non-woven*	*Woven*	*Non-woven*
Medium	70	40	180	115	70	40
High	100	75	270	180	100	75

Thus a nonwoven geotextile with a 60-lb required puncture strength has a high survivability rating, a grab strength requirement of 180 lb, and a trapezoidal tear strength requirement of 75 lb. (Note that the burst value was purposely omitted in this recent version by Task Force 25.)

COMPARATIVE TESTING OF PRODUCTS

Regarding the drainage core, there is no available ASTM method specifically for prefabricated edge drains. Therefore, it was decided to develop a Geosynthetic Research Institute (GRI) test protocol for evaluating edge-drain core strength. The major elements of the test are as follows:

- The test specimen is full width (usually 18 or 12 in.) by 6 in. long and is cut so that a reproducible pattern of protrusions exists.
- The test specimen is evaluated without its geotextile covering.
- The upper and bottom load platens are 18 by 6 in., and the test specimen is placed in the center of the load platens. Note that the stress is calculated on the basis of the specimen size. Thus the imposed stress on the edge-drain core is the force in pounds exerted by the compression test machine divided by the area of the specimen and is then converted to any desired unit, for example, pound-force per square foot, kilograms per square meter or kilopascals.
- The loading rate of the compression testing machine was 0.04 in./min (≈1 mm/min).
- For these tests the compressive load was applied perpendicular to the test specimen and not at an angle.
- The test results for the nine edge-drain cores are given in Figure 3.
- The maximum stress attained by the core is considered to be its strength. It is usually a well-defined peak at which point the protrusions begin to deform noticeably and then to "telescope" or "bulge."
- No particular concern is given to the strain at failure, because these small deformations are not believed to be of major concern in a highway edge-drain application.
- A summary of these findings will be given at the end of the paper.

The flow rate test for edge-drain cores that most manufacturers and testing laboratories use is ASTM D-4716 [Constant Head Hydraulic Transmissivity (In-Plane Flow) of Geotextiles and Geotextile Related Products]. It is not a particularly good simulation of the edge-drain situation, primarily because the core lies horizontally and flows full instead of being positioned vertically and only flowing in the lower flow zone; recall Figure 1. A more accurate test has been developed by Dempsey (*8*), but it requires more than 30 ft of floor space, which is a major impediment to its widespread adoption. Thus ASTM D-4716 was used.

The salient features of this version of the test method as it applied to edge-drain flow testing are as follows:

- The core size is 18 in. wide by 12 in. long.
- The core is evaluated without its geotextile covering.
- The core is positioned between solid plates on the top and bottom; thus no intrusion occurs.
- The value of normal stress is applied via an air bag at an initial value of 5.0 lbf/in.^{2} and then maintained for 15 min.
- Tap water is sent through the length of the core at different values of hydraulic head, starting at a high value of hydraulic gradient and successively proceeding downward in steps to the limit of the measuring system. The hydraulic gradients evaluated in this project were at the following values: 0.5 (when sufficient water was available), 0.25, 0.125, 0.062, 0.031, 0.016, and 0.0078.
- At each interval of hydraulic gradient the flow rate in gallons per minute is measured and then converted to gallons per minute-foot by multiplying the flow rate by 18/12, since the specimen size is 18 in. wide.
- When this set of data has been obtained, the normal pressure is then increased to 10 lbf/in.^{2} and the entire process is repeated. An entire series of normal pressures is then evaluated. The normal pressures used in this study were as follows: 5, 10, 15, 20, 25, and 30 lbf/in.^{2} (the limit of the air bladder system in this test method).

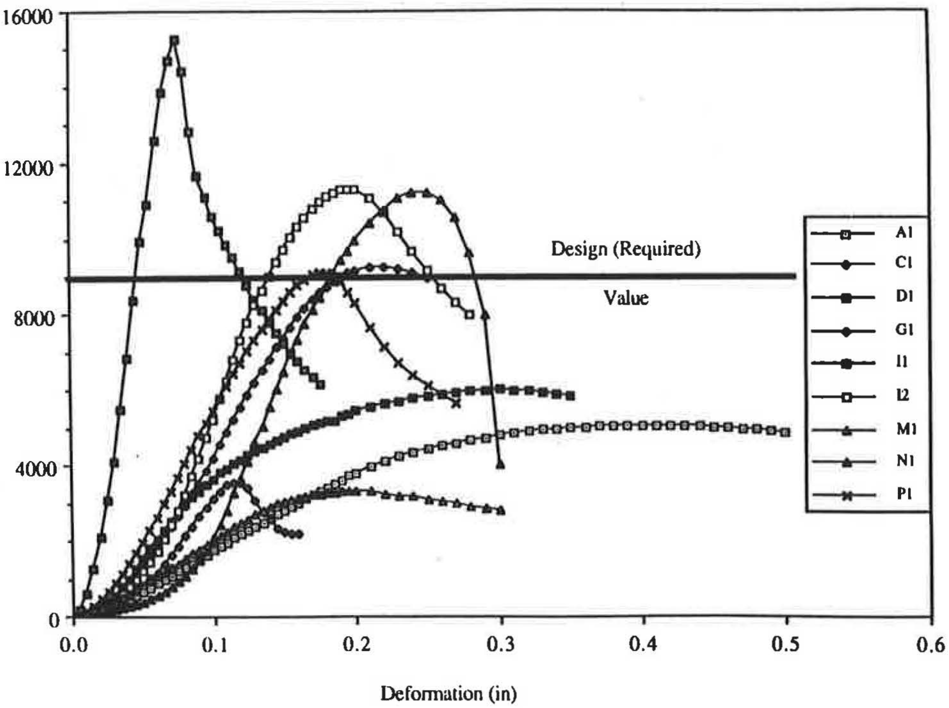

FIGURE 3 Compressive stress versus deformation of various geocomposite highway edge drains.

- The resulting data from all of the foregoing tests are then plotted as normal pressure versus flow per unit width for each hydraulic gradient value.
- The results of this entire sequence of testing have been given by Hwu (*9*) for each of the products. These findings are the essential characteristic curves for flow rate for the various geocomposite edge drains evaluated.
- The flow rate at 1,500 lbf/ft^2 pressure and a hydraulic gradient of 0.10 is then selected for comparison with the required, or design, value (Figure 4).
- A summary of these findings will be given at the end of the paper.

Because the properties of permeability, opening size, and strength of the geotextile filter are readily available from manufacturers' literature, they can be directly compared with the required values as per the previous design recommendations.

SUMMARY AND CONCLUSIONS

A design methodology for prefabricated edge-drain core and geotextile filter properties has been the focus of this paper. The designs have been developed with no specific product in mind but are regionalized to weather conditions in southeastern Pennsylvania. These are as follows:

Requirement	*Method*	*Value*
Core strength	GRI GG4	≥ 9,600 lbf/in.^{2}
Core flow rate	ASTM D4716	≥ 15 gal/min-ft (at 1,500 lbf/ft^2 and 0.10 gradient)
Geotextile permeability	ASTM D4491	≥ 0.001 cm/sec
Geotextile AOS	ASTM D4751	≥ No. 100 sieve (0.15 mm or less)
Geotextile puncture strength	ASTM D3787	≥ 75 lb
Geotextile grab tensile strength	ASTM D4632	≥ 180 lb
Geotextile trapezoidal tear strength	ASTM D4533	≥ 75 lb

Nine commercially available products were evaluated against the above-listed requirements. It was found that five products met the core strength requirement. Seven of the nine cores met the flow rate requirement. Clearly, product testing is required to see if the required design values are being met by the various products.

For the geotextile filters used on the respective cores it was found that all

- Are more than adequate in their permeability,
- Are too open and will allow soil loss into the edge-drain core,
- Have too low a puncture strength,
- Have too low a grab tensile strength, and
- Have too low a trapezoidal tear strength.

Thus they all fail to meet the recommended values on the basis of their large opening sizes and low strength properties. This difficulty, however, is readily overcome by using a somewhat heavier geotextile, which will result in a tighter void

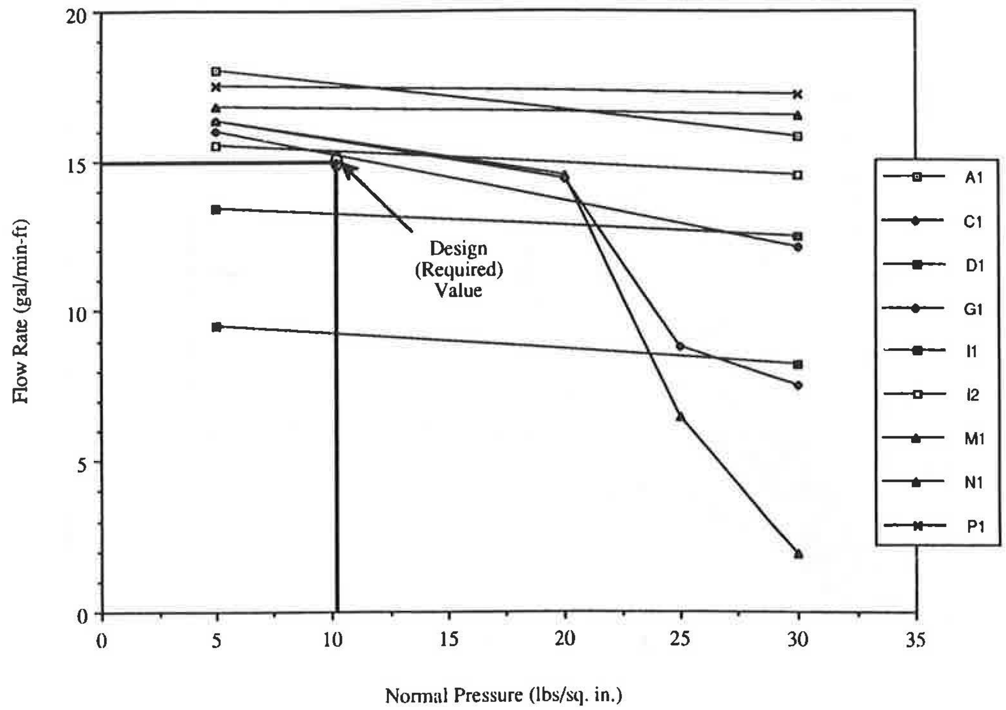

FIGURE 4 Flow rate behavior of geocomposite edge drains at hydraulic gradient of 0.10.

structure with higher strength values yet still adequate permeability.

ACKNOWLEDGMENTS

This study was funded by the Pennsylvania Department of Transportation, with Umakant Dash as the Project Technical Officer. The authors' sincere appreciation is extended to the Department, to Dr. Dash, and to other PennDOT personnel involved.

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