FIELD AND LABORATORY STUDY ON THE USE OF GEOTEXTILES AND RELATED PRODUCTS TO BRIDGE THERMOKARSTS

FINAL REPORT

by

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16. Abstract A previously developed mathematical model was modified to calculate the reinforced material properties required to bridge thermokarsts of various sizes under various loading conditions. A field test was constructed consisting of 12 test sections. Trenches were excavated and filled with water which was allowed to freeze. The ice was covered with different geotextiles and geogrids and 30 inches of uncompacted alluvial sand and gravel. After the ice had melted, the surface was smoothed, compacted and trafficked with a loaded dump truck.

All sections withstood the thawing portion of the test as anticipated. The reinforcing materials sagged into the trenches as they picked up tension leaving surface depressions of 11 to 38 inches deep. The reinforcing materials over all of the widest test trenches failed in tension when the surface was smoothed. The reinforcing materials in two of the other sections failed in tension early in the trafficking portion of the test. The reinforcing materials in the remaining sections stretched or pulled out of the embankment until at least some of the load was carried on the bottom of the trench.

Laboratory tests on the reinforcing materials demonstrated that the effective stiffness of the geotextiles was several times lower than expected. When the proper material properties were used, the mathematical model appeared to accurately predict the test performance. From a theoretical standpoint there are readily available reinforcing materials that could be used to span voids on the order of 6 to 10 feet across. Tentative design and construction guidelines are given and a set of sample design curves are presented for preliminary design.

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ABSTRACT

The Research Section of the State of Alaska Department of Transportation and Public Facilities contracted with Shannon and Wilson, Inc. of Fairbanks to develop a design methodology for the use of geotextiles and related products (reinforcing materials) to bridge thermokarsts. The study included development of a mathematical model and the performance of a full scale load test to verify the model. This presentation is a summary of that study.

A mathematical model was developed which can be used to calculate the reinforcing material properties required to bridge thermokarsts of various sizes under various loading conditions. Use of the model indicates that fairly inexpensive and readily available materials could be used to span a void up to 10 feet across.

A test facility was constructed which consisted of 3 trenches crossing 4 wheel paths making a total of 12 test sections. The trenches were excavated to widths of 4.0, 5.5, and 6.8 feet and were filled with water which was allowed to freeze in the winter of 1983-84. In the spring of 1984, the trenches were covered with different types and combinations of geotextiles and geogrids and 30 inches of uncompacted alluvial sand and gravel. In the fall of 1984, after the ice had melted, the surface was smoothed, compacted and trafficked with a loaded dump truck.

All sections withstood the thawing portion of the test as anticipated. The reinforcing materials sagged into the trenches as they picked up tension leaving surface depressions on the order of 14, 24, and 35 inches deep over the narrow, medium, and wide trenches respectively.

The reinforcing materials in all of the 6.8 foot wide test sections failed in tension when the depressions were filled in the fall. The reinforcing materials in two sections failed in tension early in the trafficking portion of the test. The reinforcing materials in the

remaining sections stretched in a creep mode or pulled out of the embankment until at least some of the load was carried on the bottom of the trench.

Tests on the geotextiles for their tension-strain properties showed that the failure tension from long term wide width tensile tests was a small percentage of the value reported in the literature from grab tensile tests. This coupled with the creep properties, which were not reported in the literature, made the effective stiffness of the geotextiles several times lower than expected.

Friction tests between the soil and the reinforcing materials support the trends reported in the literature. Friction between silt and the reinforcing materials was slightly higher than the strength of the silt alone, friction between the geotextiles and the sand and gravel was significantly less than the strength of the sand and gravel alone, and friction between the geogrid and sand and gravel was slightly less than the strength of the sand and gravel alone.

When the proper material properties are used, the mathematical model appears to accurately predict the test performance. It is apparent that published geotextile properties are not always sufficiently accurate for use in stabilization design. It appears, from a theoretical standpoint, as though there are readily available reinforcing materials that could be used to span voids on the order of 6 to 10 feet across if the geotextiles have enough anchorage and if the orientation of the void and road is favorable. Woven geotextiles are directional in nature and therefore must be used with care in any environment where the orientation of the voids is not known, although there is some evidence that they may perform better on the bias when confined in the soil than they appear to when unconfined. Nonwoven geotextiles are generally not directional in nature but they are not generally stiff enough or strong enough to satisfy the needs. In addition, it would be difficult to span any void which is near the toe of a slope and runs parallel to the slope because of the need for anchorage on both sides of the void.

Tentative design and construction guidelines are given on the assumption that they not be used in production until a test has been conducted that proves that the concept will work. A set of sample design curves are presented for preliminary design and a computer program is presented for detailed design.

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FIELD AND LABORATORY STUDY ON THE USE OF GEOTEXTILES AND RELATED PRODUCTS TO BRIDGE THERMOKARSTS

1. INTRODUCTION

1.1 BACKGROUND

Roads in the north frequently cross permafrost terrain containing ice wedges. The thermal characteristics of the road surface cause the permafrost and the ice wedges to thaw beneath the road. The thawing ice may cause a sharp dip in the road surface which is hazardous to motorists. The dip may form at the rate of several inches per week during the summer and early fall requiring slow driving and frequent roadway surface leveling throughout this entire period. Thawing and settlement typically continue for many years until all the ice has melted, thermal equilibrium is reached, or the road fill bridges the void left by the thawing ice.

The current design technique to alleviate the problem is to use an increased height of fill with or without an insulation layer. This procedure slows the rate of thawing, usually smooths out the dips on the surface, and increases the potential for bridging. Long term maintenance is still required even if very large fill thicknesses are used. A better solution would be highly desirable, particularly in areas where large quantities of good quality fill material are difficult or expensive to obtain.

Geotextiles and related products are frequently used to increase the stability of roadbeds underlain by soft subgrades. Laboratory model studies by Kinney (1979), Collins, et. al. (1980), Giroud et. al. (1982), Kinney and Barenberg (1980), Binquet and Lee (1975), and others have shown dramatic increases in stability under certain conditions. Field test installations by Webster and Watkins (1977), Steward (1977) and others also show very significant increases in stability when a

geotextile is used over a soft subgrade. In general the field and laboratory data indicate that stiffer reinforcing materials usually produce greater stabilizing effects, and that the benefit is generally greatest when a thin aggregate layer is used over a very soft subgrade. Many of the manufacturers' brochures and other publications present design procedures which make use of these trends.

Several theoretical design approaches have been published which establish that a significant portion of the stabilizing effect comes about through the "drum effect" where part of the roadbed and wheel load is carried by the reinforcing material instead of being transmitted directly to the soft subgrade below. Kinney (1979), Kinney and Barenberg (1979), Leflaive (1977), Bourdeau et. al. (1982), Kinney and Barenberg (1982), and Giroud (1982) have presented fairly detailed descriptions of the mechanisms involved in the transfer of stress from vertical stress on the subgrade directly under the load, to a combination of vertical and horizontal stress on the subgrade outside the normal zone of vertical stress influence. Additional benefit is gained through an increase in confining pressure in the granular fill. This increases the modulus of the granular layer thereby allowing it to spread the load out more effectively (Kinney 1979). These mechanisms have been extended in this paper to cover the situation where the reinforcing material supports the entire roadbed and wheel load across a total void caused by a thawing ice wedge.

1.2 OBJECTIVE

The objective of this study was to determine if the reinforcement technique can be efficiently and economically used in new construction where thawing ice wedges are expected or in the reconstruction of existing roads where the problem has occurred. In old construction it may, in some instances, be necessary to only reinforce the area directly affected, whereas in other areas, it may be necessary to reinforce long reaches of roadway. The design and construction techniques developed from this study are established such that they can be performed on site

specific areas as they are anticipated in design or encountered in the field. This study includes a determination of the best materials available for tensile reinforcement from those currently manufactured by the industry and reasonably available in Alaska.

1.3 SCOPE OF STUDY

A laboratory study was performed to define the tension-strain properties of the materials used. The existing analytical models were refined. A full scale field test was designed, constructed, and instrumented which consisted of a simulated ice wedge covered by several reinforced embankment sections. The embankments were monitored over one thawing period and the test results were combined with the theoretical developments to establish a design scheme.

The work plan followed the sequence listed below:

1. A literature search was performed to determine the results of any current work in the area of study.

2. Various reinforcement materials were obtained and laboratory tests were performed to determine their tension-strain characteristics.

3. The existing analytical model was refined.

4. A suitable test site was selected.

5. A full scale field test was designed, constructed, and instrumented. The test was designed so that an objective evaluation could be made of the data obtained.

6. The test embankment was monitored over one thawing period.

7. This report was prepared detailing the results of the study. This report contains the following information.

- A detailed summary and objective analyses of the test results.

- Implementation recommendations based upon the test results.

- Recommended design criteria, construction techniques, and construction specifications which identify and emphasize any areas of special significance for a design or construction engineer.

- Recommendations which outline areas requiring further study of tensile reinforcement of road embankments.

2. THEORETICAL MODEL

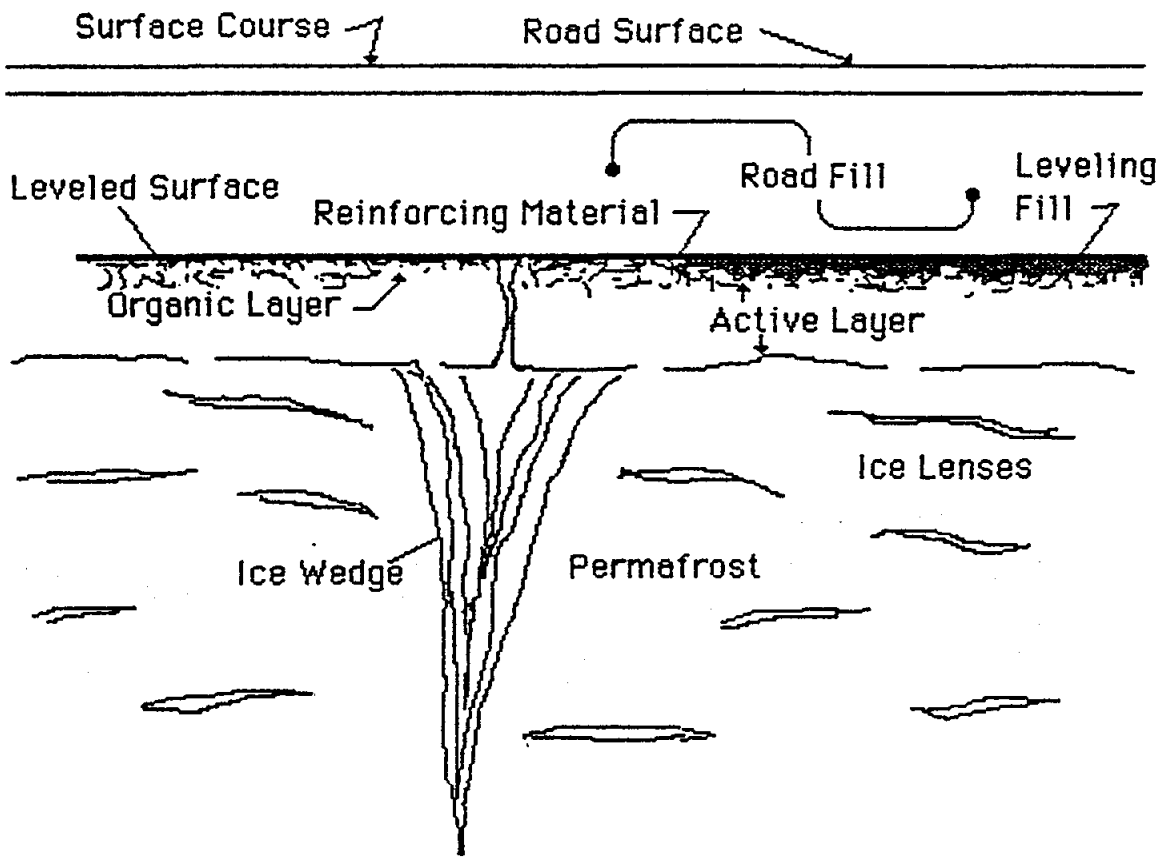
2.1 GENERAL CONCEPTS

The conditions immediately after construction of a road are shown schematically on Figure 1. Frequently the organic layer is left in place to avoid disturbing the existing active layer during construction. The organic layer provides some insulation and helps spread the load on the active layer which is frequently very soft, at least initially.

Figures 2 and 3 show schematically what the road section might look like after a few thawing seasons with and without a reinforcing material if no maintenance were performed. Thawing ice rich permafrost will cause differential settlement. The reinforcement is not expected to significantly affect the unevenness caused by thawing ice rich permafrost. If the subsurface explorations indicate that this unevenness will be significant, the road should not be paved until the time between necessary surface levelings is long enough to warrant paving.

The behavior of the road surface over the ice wedge is expected to be greatly different with and without reinforcement. Without reinforcement, the material in the original active layer should settle abruptly over the wedge. The shape of the depression in the road will be determined by the width and shape of the ice wedge, the thickness of the active layer, the materials in the active layer, the thickness of the road section, the materials in the road section, and the traffic loading. The width of the distressed area will probably be wider than the ice wedge under most conditions, however, if conditions are such that bridging occurs, the distressed portion of the road surface may be narrower than the ice wedge. Bridging is most likely to occur when the active layer is stiff, the road embankment is well compacted, and the traffic loading is light.

In a properly designed reinforced section, the reinforcing material should be able to span the void left by the ice when it thaws as shown



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TYPICAL PROFILE AFTER CONSTRUCTION BUT PRIOR TO THAWING

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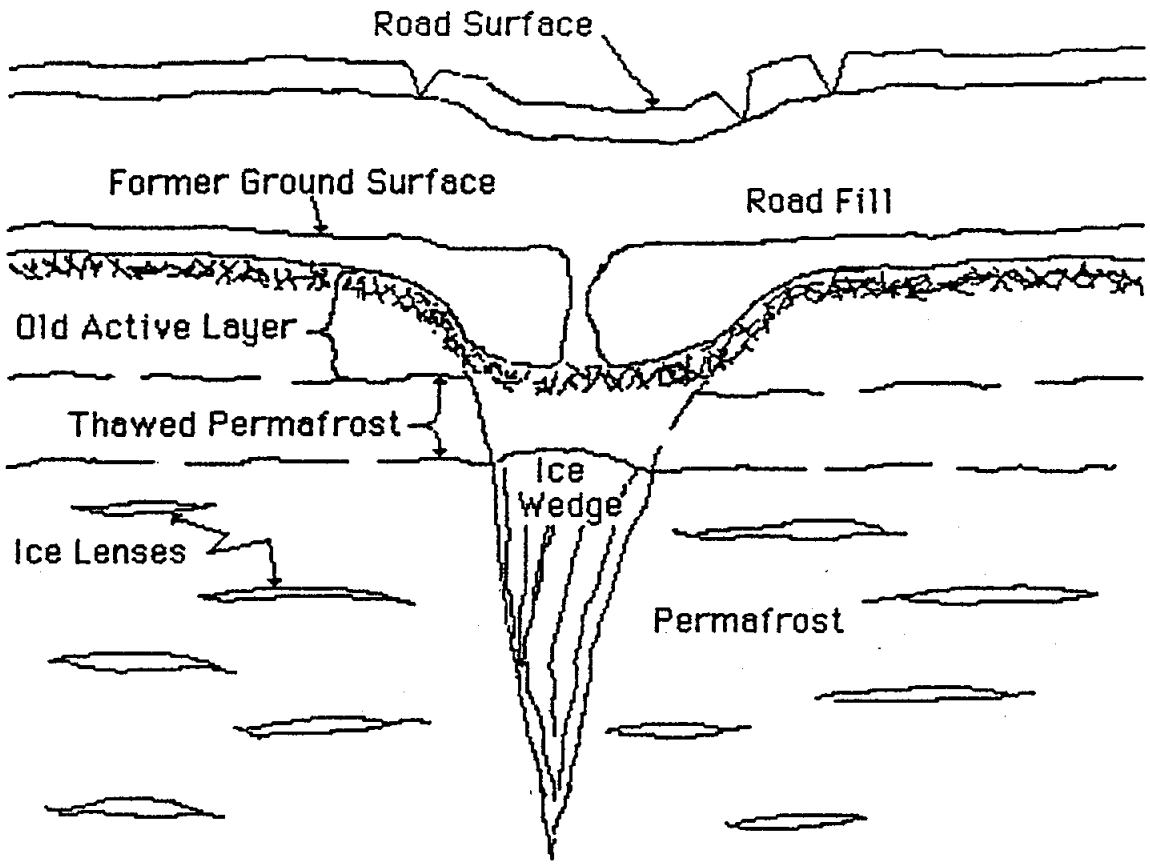
Fig. 1



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TYPICAL PROFILE AFTER THAWING WITHOUT REINFORCING MATERIAL OR MAINTENANCE

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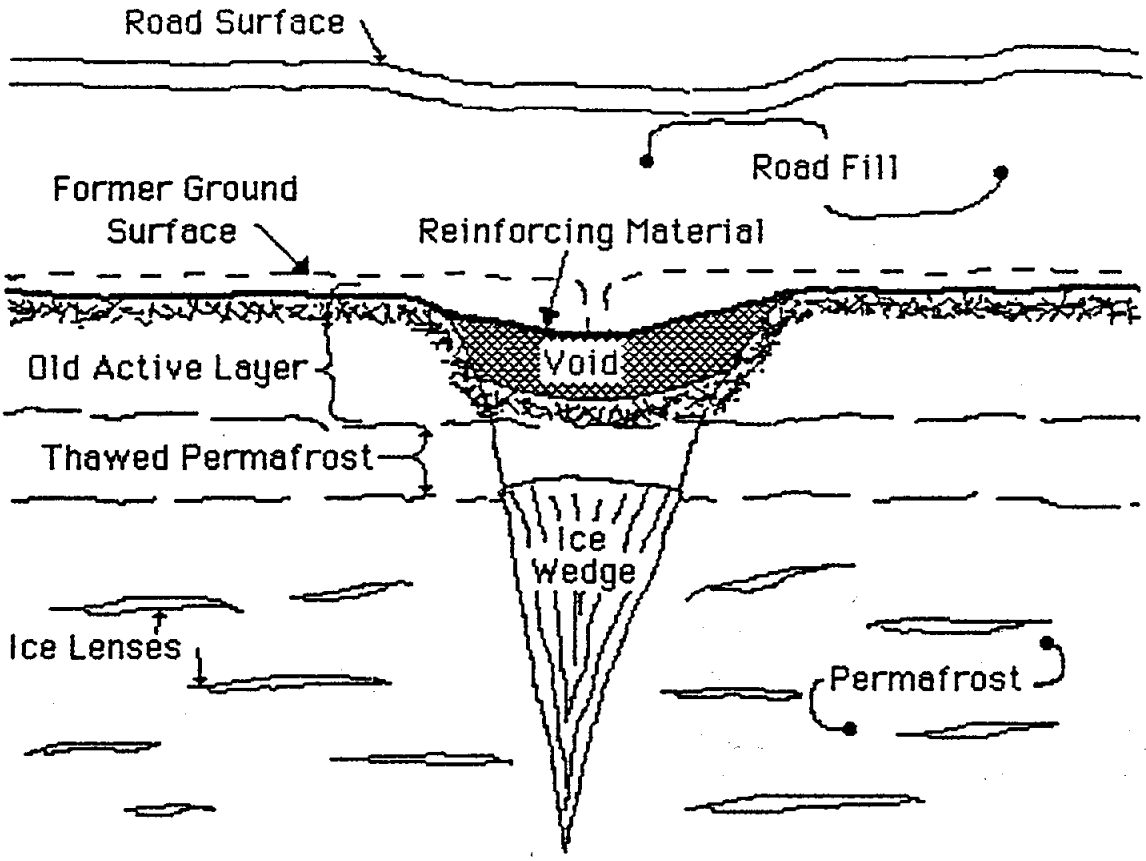
Fig. 2



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TYPICAL PROFILE
AFTER THAWING WITH A
REINFORCING MATERIAL
BUT WITHOUT MAINTENANCE

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Fig. 3

on Figure 3. This will work if the reinforcement is strong enough, it is properly anchored, and the thawed permafrost at the edge of the void is sufficiently stiff and strong to support the road section without collapsing into the void. The largest unknown in the analysis is the ability of the thawed permafrost at the edges of the void to carry the required load. There are also significant unknowns involved in the shear strength between the reinforcement materials and the soil and the tension-strain characteristics of geotextiles and related products under long term and dynamic loading.

The reinforcing materials support the road embankment by a combination of hoop tension and lateral restraint of the aggregate in the embankment allowing it to bridge the void. The bridging effect has been discussed by Kinney (1979) and others; however to date there are no comprehensive analysis techniques to quantify the effect. Soil bridging appears to be a minor factor in this application of geotextiles and related products and it is conservative to neglect it. The following analysis is based on hoop tension alone.

In order to develop the required uplift pressure to support the road embankment and the wheel load, the reinforcing material must undergo downward movement and must develop tension from its embedment outside the void. The displacement, which will also be experienced by the road surface, may amount to several feet depending on the width of the void and the properties of the reinforcing materials. The deformation will take place as the ice melts if the reinforcing material does not creep (continue to deform with time under the same load), the bond between the reinforcing material and the soil in the anchorage zone does not fail or creep, and the walls of the void do not collapse over time. Once the initial displacement has occurred and the ice has melted the embankment should remain stable.

The dynamic properties of the reinforcing materials and the lateral constraint placed on the fill become important in deflection of the road as a vehicle passes. Preliminary information, Kinney (1979), Koerner

et. al. (1980), and Gourc et. al. (1982) indicates that some geotextiles have a much higher dynamic modulus than static modulus. Based on the information available it is not unreasonable to assume that there are some reinforcing materials that have a high enough dynamic modulus to provide a reasonably stable road under traffic loading. However some movement is expected which may be too much to make paving a viable option. The amount of movement will be dependant upon the reinforcing material properties, the geometry, the traffic load, and the amount of bridging created in the embankment by the reinforcing material.

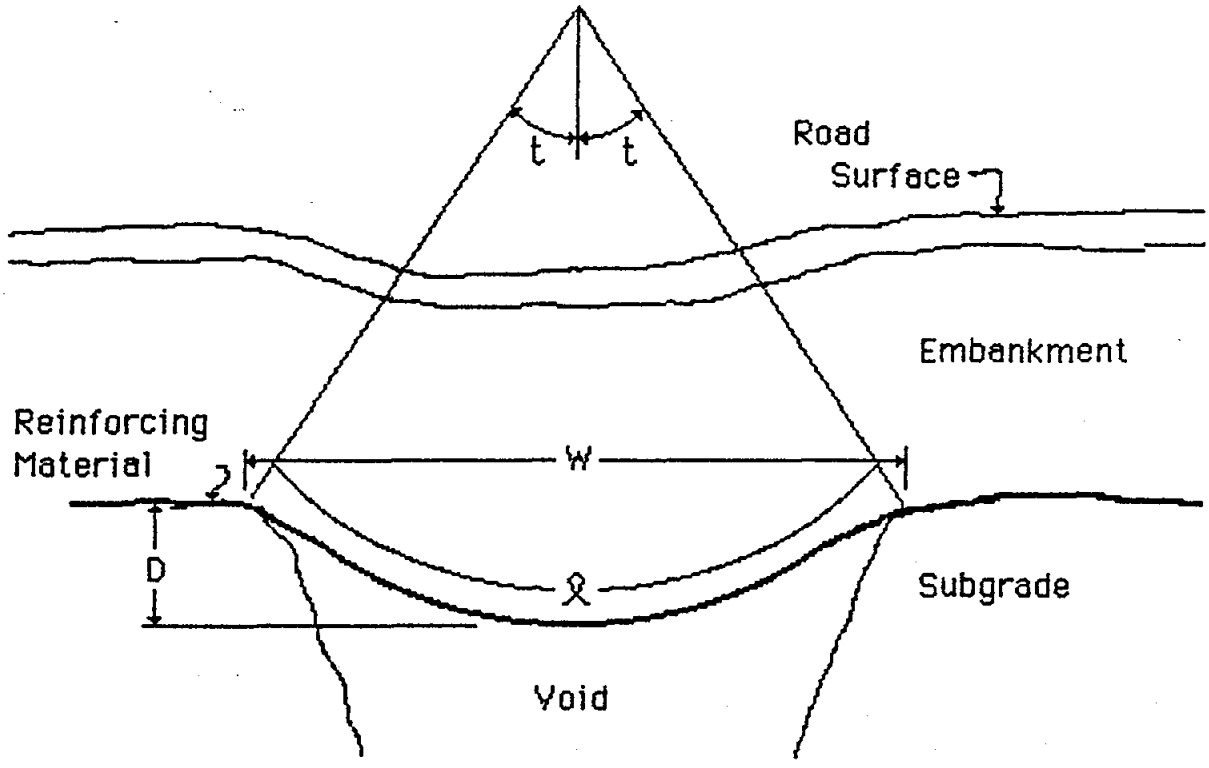
2.2 MATHEMATICAL MODEL

The mathematical model presented herein assumes that the supporting potential of the reinforcing material over the void follows the shape of a circular segment as shown on Figure 4. Equation 1 is based on the hoop tension theory, assuming that the stress on the reinforcing material is normal to the material, and that the curved surface is circular. This is not consistent with the experimental results obtained by Kinney (1979) where he found significant changes in tension in the reinforcing material over the depression in a soft subgrade, however the assumption appears to be conservative.

$$T = pr \qquad \text{(Equation 1)}$$

where: T = Tension in reinforcing material per unit length. (lbs/ft)
p = Normal pressure on reinforcing material. (psf)
r = Radius of curvature of reinforcing material. (ft)

The distance the reinforcing materials pulls out from the edge of the void is not easy to approximate. Although several studies have been performed, McGown and Andrawes (1977), Salomone et. al. (1980), and Collios et. al. (1980), and others, none have directly addressed the problem encountered here. In light of the number of variables and the lack of definition of each, it seems appropriate to take a simplified



Basic Equations

$W = 2r \sin(t)$

$\ell = \pi r t / 90$

$D = r - r \cos(t)$

$\ell = W + \frac{T}{E} W + 2d_2$

Note: d_2=Distance reinforcing material is pulled out from edge of void.

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GEOMETRY USED IN THEORETICAL MODEL

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Fig. 4

approach. The approach used herein assumes that the soil is rigid and that the reinforcing material is linearly elastic. Both of these assumptions could be easily eliminated with relatively minor changes in the equations if the input parameters were known. In accordance with the assumptions, there is a point of fixity beyond which the reinforcing material does not move relative to the soil and the full shear stress is developed on the reinforcing material between the void and the point of fixity, Figure 5. The tension in the reinforcing material varies linearly from zero at the point of fixity to a maximum at the void. Equation 4 relates the displacement of the reinforcing material at the edge of the void to the shear stress developed between the reinforcing material and the soil, and the stiffness properties of the reinforcing material.

$$d_2 = (T/2)*(1/E)*(L) \qquad \text{(Equation 2)}$$

Where: d_2 = Distance reinforcing material is pulled out from the edge of the void. (ft)

E = Effective secant modulus of the reinforcing material. (lbs/ft)

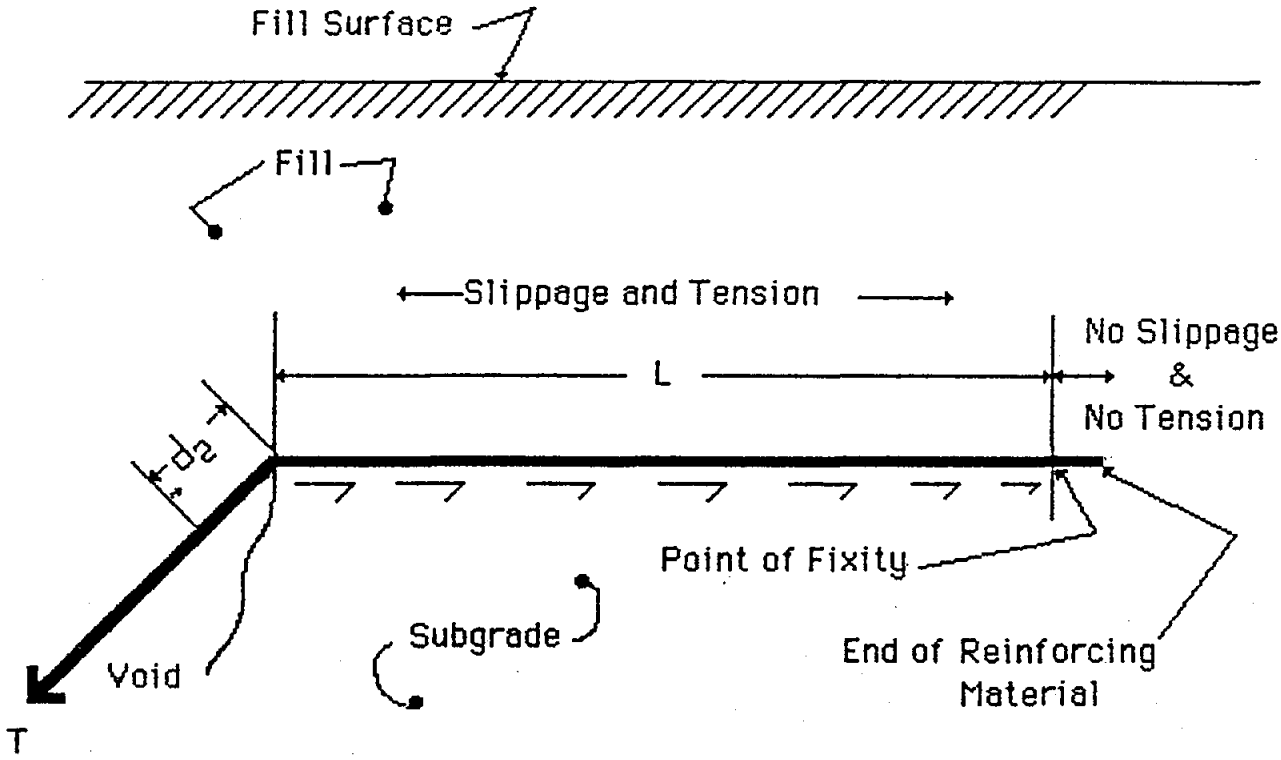
$$L = T/s \qquad \text{(Equation 3)}$$

L = Length of reinforcing material from the void to the point of fixity. (ft)

s = Maximum shear strength between the reinforcing material and the soil. (psf)

$$d_2 = T^2/(s * E * 2) \qquad \text{(Equation 4)}$$

The radius of curvature of the reinforcing material is related to the change in length of the portion of the material spanning the void. The original length is the chord length, W, shown on Figure 4. The increase



$$d_2 = \frac{T}{2E} L$$

d_2 = Distance reinforcing material is pulled out from the wall of the void

L = Length of reinforcing material that is under tension

T = Maximum tension reinforcing material

E = Elastic modulus of reinforcing material

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STRESS ON
REINFORCING MATERIAL
OUTSIDE VOID

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Fig. 5

in length is a combination of the strain over the width, W, caused by the tension in the material and the distance that the material is pulled out from the edges of the void . Equation 5 is the expression for the change in length due to strain in that portion of the material over the void. It is assumed that the tension is constant across the void.

$$d_1 = W*T/E \qquad \text{(Equation 5)}$$

Where: d_1 = Change in length of the portion of the reinforcing material spanning the void. (ft)

W = Effective width of void. (ft)

Assuming the chord length and arc length of a circular segment are known, the radius, the subtended angle, and the vertical displacement shown in Figure 4 can be determined by solving the geometrical relationships given in Equations 6, 7 and 9.

$$W = 2\ r \sin(t) \qquad \text{(Equation 6)}$$

$$L_1 = 3.1416 * r * t/90 \qquad \text{(Equation 7)}$$

$$L_2 = W+d_1 +d_2 * 2 \qquad \text{(Equation 8)}$$

$$D = r - r \cos(t) \qquad \text{(Equation 9)}$$

Where: r = Radius of the circular segment. (ft)

2 * t = Subtended angle. (Degrees)

L_1 = Arc length calculated from r & t (ft)

L_2 = Arc length calculated from calculated reinforcing material length. (ft)

D = Maximum vertical displacement of the reinforcing material. (ft)

The complete solution to the problem involves an iterative approach based on Equations 1 through 7 as shown in the following steps.

Step 1. Determine p and W from the geotechnical exploration, the design traffic loading, and an assumed embankment thickness.

Step 2. Choose a reinforcing material modulus, E.

Step 3. Estimate r. If no other information is available, start with r = (3/4) *W.

Step 4. Solve for T using Equation 1.

Step 5. Solve for d_2 and d_1 using Equations 4 and 5.

Step 6. Solve for t using Equation 6.

Step 7. Solve for L_1, using Equation 7.

Step 8. Solve for L_2 using Equation 8.

Step 9. Repeat steps 3 through 8 using various r's until $L_1 = L_2$.

Step 10. Compare the calculated tension in the reinforcing material(T) with the maximum tension that can be sustained by the material.

Step 11. Check the assumptions regarding the reinforcing material modulus to see that the values used are reasonable in light of the calculated tension.

Step 12. Solve for the vertical displacement of the reinforcing material using Equation 9 to see if it is within acceptable limits.

Step 13. Solve for the length of the embedment needed for anchorage using Equation 3.

An example of this procedure is shown in Appendix A.

The analysis shown above is valid if there is no slippage at the end of the reinforcing material and the calculated radius is greater than W/2. The exceptions to either of these conditions can be handled easily by adding the movement at the end of the material to the arc length. If the embedment length in the field is not sufficient for complete anchorage, the reinforcing material will slip throughout its entire length resulting in increased vertical displacement over the void. Once slippage at the end starts to occur, the tension will remain the same but the slippage will increase the arc length, thereby decreasing the radius which increases the load that can be carried by the reinforcing material. The system will continue to become more stable as slippage progresses because the radius continues to decrease until the radius is reduced to one half of the void width. At that point the system becomes unstable, and failure will occur by the reinforcing material pulling out, and the void closing.

If the modulus of the reinforcing material, E, is not high enough, there will be enough stretching to cause the material to sag into the void more than a distance of W/2. Mathematically this can be handled easily, however, a higher modulus material should probably be used.

The flow chart for the computer program written to solve these equations is shown on Plate 1 and the program is shown on Plate 2.

2.3 NECESSARY INPUT PARAMETERS

Use of the mathematical model requires several parameters that must be evaluated or estimated. They can be broadly grouped into four basic categories: intended use, geometry, reinforcing material properties, and reinforcing material - soil friction properties.

DESIGN ANALYSIS
PLATE 1 - FLOW CHART

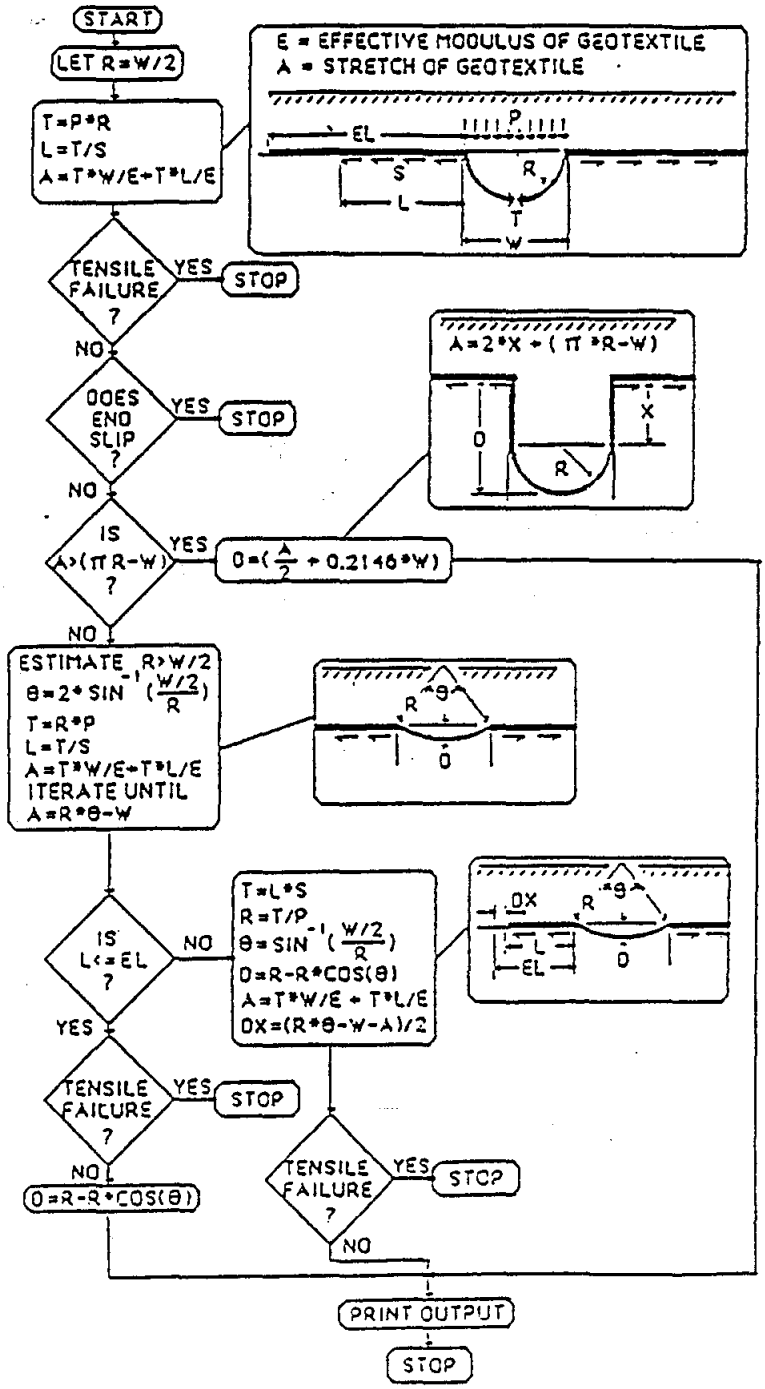


PLATE 2 COMPUTER PROGRAM

```
10 INPUT "WIDTH OF VOID (FT.)"; W
20 INPUT "EMBEDMENT LENGTH (FT.)"; EL
30 INPUT "APPLIED PRESSURE (PSF)"; P
40 INPUT "SHEAR STRESS ON GEOTEXTILE (PSF)"; S
50 INPUT "EFFECTIVE SECANT MODULUS OF GEOTEXTILE (LBS/FT)"; E
60 INPUT "ULTIMATE TENSILE STRENGTH OF GEOTEXTILE (LBS/FT)"; TMAX
70 PRINT
80 REM START WITH R = W/2
90 R = W/2
100 T = P*R
110 IF T < TMAX THEN GOTO 140
120 PRINT "MAXIMUM TENSION EXCEEDED WITH R = W/2   T = ";T;"LB/FT"
130 GOTO 510
140 L = T/S
150 IF L <= EL THEN GOTO 180
160 PRINT "INSUFFICIENT EMBEDMENT WITH R = W/2  -  REQUIRED L = ";L;"FT"
170 GOTO 510
180 A = T*W/E + T*L/E
190 IF A < 3.1416*R - W THEN GOTO 240
200 PRINT "VERTICAL DISPLACEMENT > W/2 AND ENDS DO NOT SLIP"
210 D = A/2 + 0.2146*W
220 DX = 0
230 GOTO 420
240 REM EQUILIBRIUM R > W/2
250 R = R + .01
260 T = P*R
270 IF T <= TMAX GOTO 300
280 PRINT "MAXIMUM TENSION EXCEEDED WITH R > W/2"
290 GOTO 510
300 L = T/S
310 A = T*W/E + T*L/E
320 TH = 2*ATN(1/((2*R/W)^2 - 1))
330 IF L >= EL THEN GOTO 370
340 IF A <TH*R - W THEN GOTO 250
350 DX = 0
360 GOTO 400
370 PRINT "R > W/2 AND ENDS SLIP"
371 L = EL
372 T = L*S
373 R = T/P
374 TH = 2*ATN(1/((2*R/W)^2 - 1))
375 A = T*W/E + T*L/E
380 DX = (R*TH - W - A)/2
390 GOTO 410
400 PRINT "R > W/2 AND ENDS DO NOT SLIP
410 D = R - R*COS(TH/2)
420 REM PRINT OUTPUT
430 PRINT "MAXIMUM TENSION (LBS/FT.) = "; T
440 PRINT "FACTOR OF SAFETY WITH RESPECT TO TENSION = "; TMAX/T
450 PRINT "MAXIMUM STRAIN (%) = "; 100*T/E
460 PRINT "CENTERLINE DISPLACEMENT (IN.) = "; D*12
470 PRINT "RADIUS (FT.) = "; R
480 PRINT "FACTOR OF SAFETY WITH RESPECT TO SLIPPAGE = "; EL/(T/S)
490 PRINT "END SLIPPAGE (IN.) = "; DX*12
500 PRINT "REQUIRED EMBEDMENT (FT.) = "; L
510 END
```

2.3.1 USE

A decision must be made as to the intended use of the road. High deformations are expected while the load is being transferred from the thawing ice to the reinforcing material. Some additional deformation may be expected due to creep of the reinforcing material and creep in the soil. Additional deformations may be caused by a combination of dynamic loading, overstressing materials, organic material decay, and freeze - thaw action. The large deformations totally eliminate any possibility of paving early in the life of the road. It is not certain at this point if paving will ever be a viable option due to dynamic movement caused by wheel loads over the void.

2.3.2 GEOMETRY AND LOADING

There are practical limits to the strength and stiffness that can be obtained from readily available geotextiles and related products. This limits the geometry and loading that should be considered. There will be a static load over the void equal to the original fill plus the additional fill required to level the surface as thawing progresses. In addition there will be a dynamic load due to the traffic loading. Adding these loads forms an upper bound because some of the load over the void will be transferred to the sides of the void due to the arching action of the fill material.

Most of the fill will probably be in a loose state for two reasons. First, good compaction can only be achieved on a firm subgrade and the subgrade over a thawing ice mass will only be firm when it is frozen and fill cannot be well compacted when it is frozen. Second, the expected large deformations over the void during thawing will cause large strains in the overlying fill causing it to loosen if it is at a void ratio which is less than the critical void ratio. For practical purposes it is probably accurate enough to assume a dry density of 105 to 115 pcf for a good quality granular fill. The fill could be nearly saturated at

some times of the year giving a total density of about 130 pcf to be used for design.

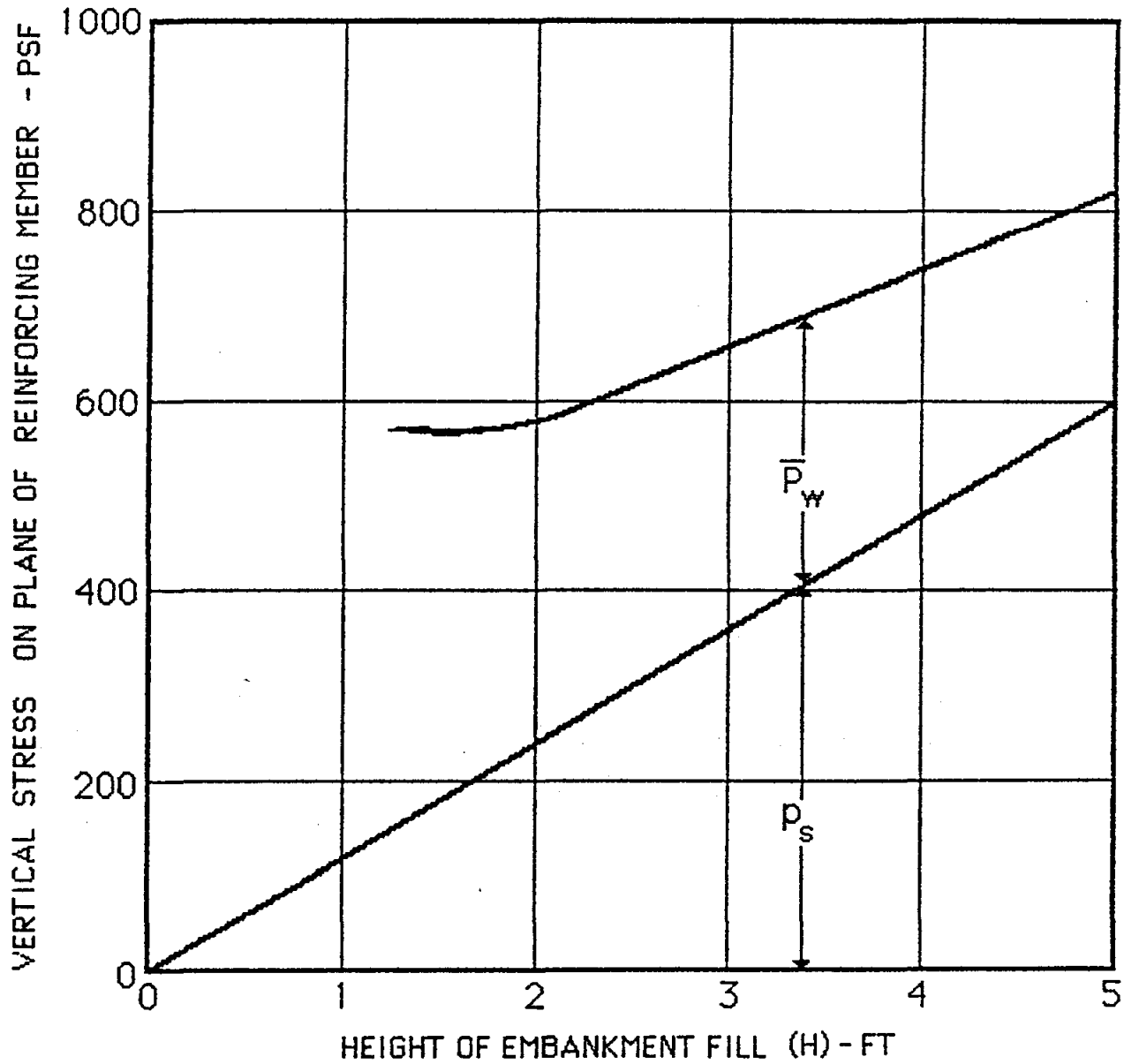
It would be prudent to estimate the fill thickness over the void to be the section design thickness plus about 0.3 times the design void width. If the reinforcing material deforms vertically more than 0.5 times the void width then the system is probably in a state of impending failure. If the system is designed for significantly less vertical deformation than 0.25 times the void width, then it is probably overdesigned.

Choosing the design void thickness is difficult. In polygonal ground it is possible to have ice wedges 10 or more feet in width and ice masses much larger in width. It will probably be necessary to design based on a maximum practical width and to accept the risk of failure if larger voids are encountered. If the road receives regular traffic, a failed reinforced section should perform much the same as a section without reinforcing regardless of the mode of failure.

The load applied to the reinforcing material over the void is a combination of soil weight and wheel load. The wheel load spreads out fairly rapidly through the soil and therefore the lightest average load across the void occurs at a fairly thin embankment section. A reasonable estimate of the average vertical load on the reinforcing material is shown on Figure 6. The loads shown on Figure 6 are based on a displacement of 0.4W, a soil unit weight of 120 pcf and a legal dual tandem axle load.

2.3.3 REINFORCING MATERIAL PROPERTIES

The wide width tensile test appears to be emerging as the most appropriate test for determining the stress-strain and strength properties of geotextiles and related products being used to span voids. Using this testing technique, the stress-strain properties appear to be a function of the loading history and the rate of loading for some materials, Kinney (1979), Koerner et. al. (1980), Gourc et. al. (1982) and others.



H = Use original height of fill + 0.3 * Width of void

$p = p_s + \bar{p}_w$

$\bar{p}_w$ = Average wheel pressure over width, W

p_s = Density of soil * H

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DESIGN VERTICAL PRESSURE
REINFORCING MATERIAL

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Fig. 6

More work is needed on both the dynamic properties and the creep properties of geotextiles and related products. To date there are no generally accepted test procedures or generally available list of dynamic and/or creep properties for geotextiles and related products.

Ice wedges do not always run perpendicular to roads, hence materials that are stiff in two directions and are soft on the bias must be used with caution. Stiff materials develop high tensions because of the low vertical displacement. If the tension exceeds the breaking strength of the material, it will fail.

2.3.4 FRICTION ALONG REINFORCING MATERIAL OUTSIDE VOID

Several studies have been published regarding the friction between geotextiles and the surrounding soil, Collios et. al. (1980), Salomone et. al. (1980), McGown and Andrawes (1977) and others. None of these studies address the situation encountered herein, however the general concepts presented provide some guidance. Kinney (1979) presented laboratory data which demonstrates that there may be little or no shear displacement between the reinforcing material and the overlying soil, hence there may be little or no shear stress between the two.

The reinforcing materials are usually placed on the original ground. Sometimes a leveling course of sand is used over the original ground to make a smooth surface so that the material can be placed without wrinkles. Shearing may occur between the reinforcing material and the soil, through the soil below the reinforcing material, or at the boundary between soil layers below the reinforcing material. Information regarding the shear strength between geotextiles and related products and organics is completely lacking. If the ground is stripped, the controlling factor will be the shear strength between the reinforcing material and the underlying soil which can be estimated from data in the literature.

3. TESTS

The testing phase of this project consisted of laboratory tests to establish the tension-strain characteristics of the geotextiles and geogrids used, laboratory test to determine the frictional characteristics between the geotextiles and geogrids and the soils used, and field tests to evaluate the mathematical model. The test facilities are described and the test results are presented in this section. The test results are discussed in Section 4.

3.1 GEOTEXTILE AND GEOGRID PROPERTIES

Two geotextiles and one geogrid were selected for use in the tests. The selection was based on the desired range of properties needed, the published material properties, and the availability of the materials. The significant properties recorded in the manufacturers' literature for the three materials used are shown in Table 1.

TABLE 1

GEOTEXTILE AND GEOGRID PROPERTIES REPORTED IN THE LITERATURE

	Material	Failure ** Tension	Failure ** Strain
A.	Woven Polypropylene Split Film Geotextile	540*	18%
B.	Woven Polypropylene Monofilament Geotextile	385*	30%
C.	Extended Polypropylene Two Dimensional Geogrid	1230+	10%

* Grab Tensile Test - lbs.

\+ Wide Width Tensile Test - lbs./ft.

** Values in the direction used in the field test series.

3.1.1 TEST FACILITIES

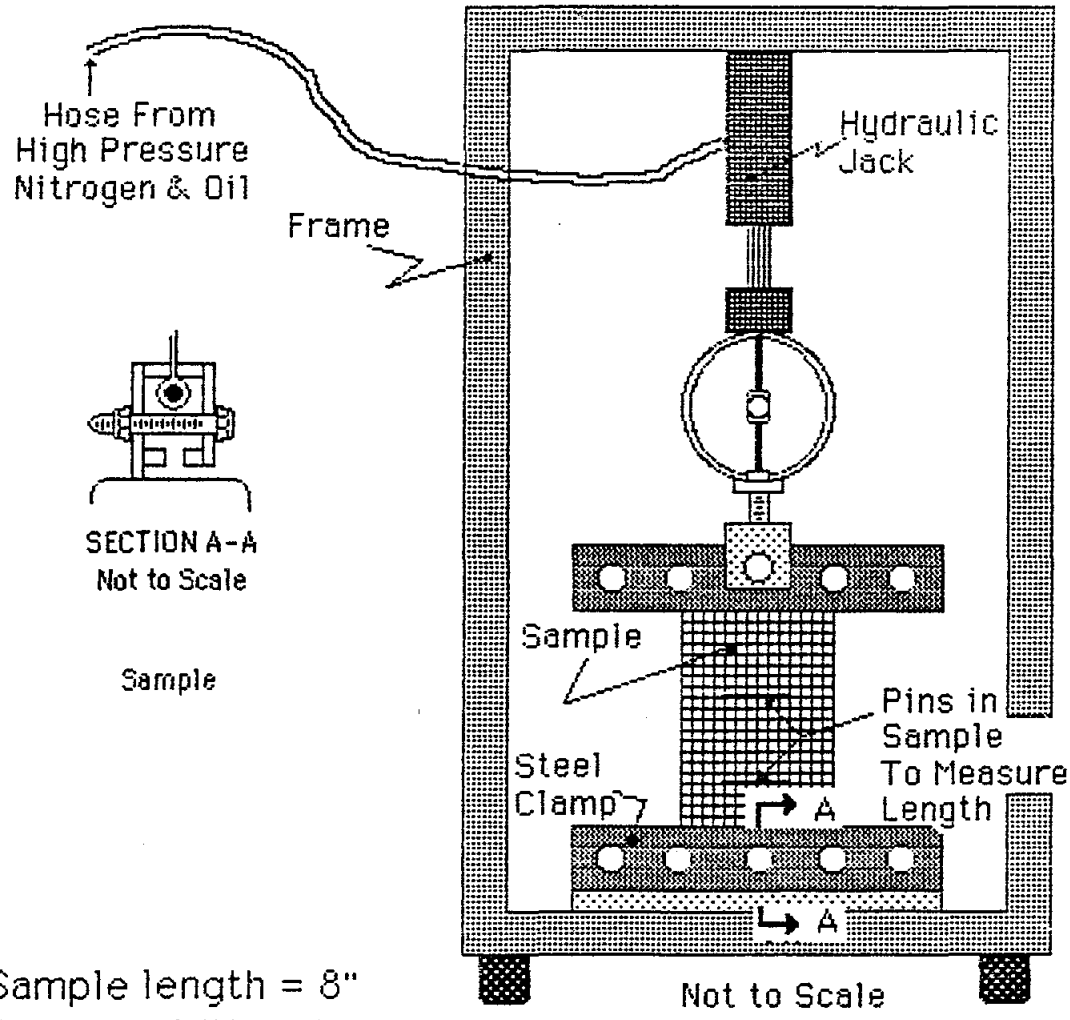
A special testing apparatus was designed and built to determine the necessary tension-strain relationships needed to analyze the field test results. The test apparatus consisted of two 8-inch wide clamps mounted in a frame so that they could be pulled apart by a hydraulic piston as shown on Figure 7 and in Photograph 1.

3.1.2 TEST RESULTS

The geotextiles used were made of polypropylene which is known to creep. Therefore creep was considered in developing the tension-strain relationship needed for analysis of the field test. Since it was impractical to run tests for several months several tests were run at varying loading rates and two tests were run where the load was held constant for several days.

The results of the constant rate of loading tests are shown on Figures 8 and 9. These test results demonstrate that the materials are nearly linearly elastic to failure if loaded fairly rapidly. They also demonstrate that the relationship between tension and strain in the materials is highly dependent on the loading rate. Whether or not the ultimate strength is a function of loading rate is not clear from these tests.

The ultimate strength and the failure strain reported in the literature are also shown on Figures 8 and 9. It is obvious that there is a disparity between the published stress-strain relationships and the stress-strain relationships measured in the tests done for this study. The published properties are based on the grab tensile test. The grab tensile test is performed by taking a strip of material 3 inches wide and stretching it to failure by increasing the tension at a constant rate. The clamps are 1 inch wide and are placed in the center of the 3 inch wide sample. The logical conclusion is that the grab tensile test should give results that are slightly higher than the wide width tensile test when used on woven geotextiles because both tests pull directly on



Notes:

Sample length = 8"
Sample width = 4"
Gage length = 4"
Length measured with micrometer calipers
Tension measured with proving ring
Pressure controlled with high pressure nitrogen

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REINFORCING MATERIAL
TENSION - STRAIN
TEST APPARATUS

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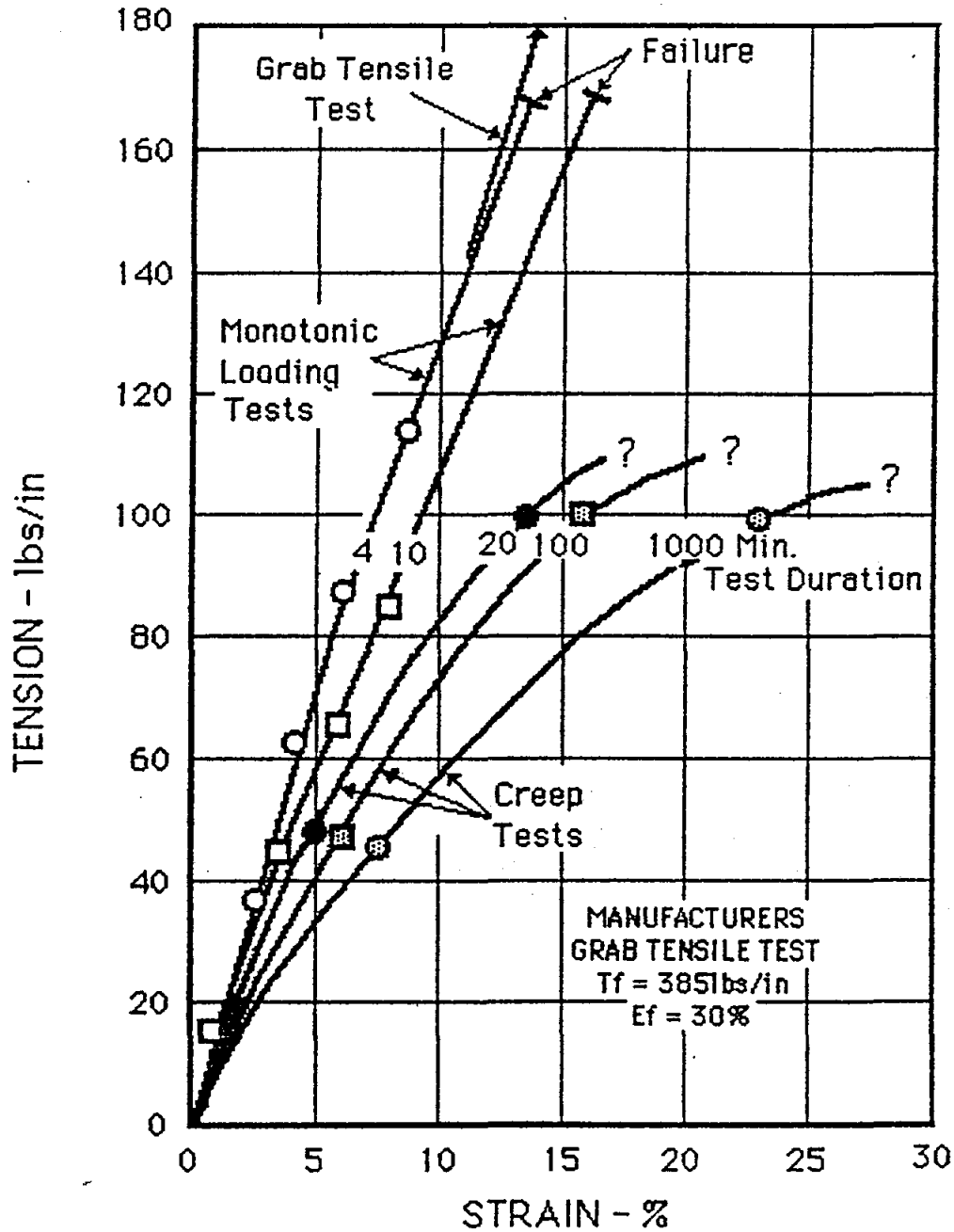
Fig. 7

GEOTEXTILE TESTING APPARATUS

PHOTO 1



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TENSION Vs. STRAIN
MATERIAL B

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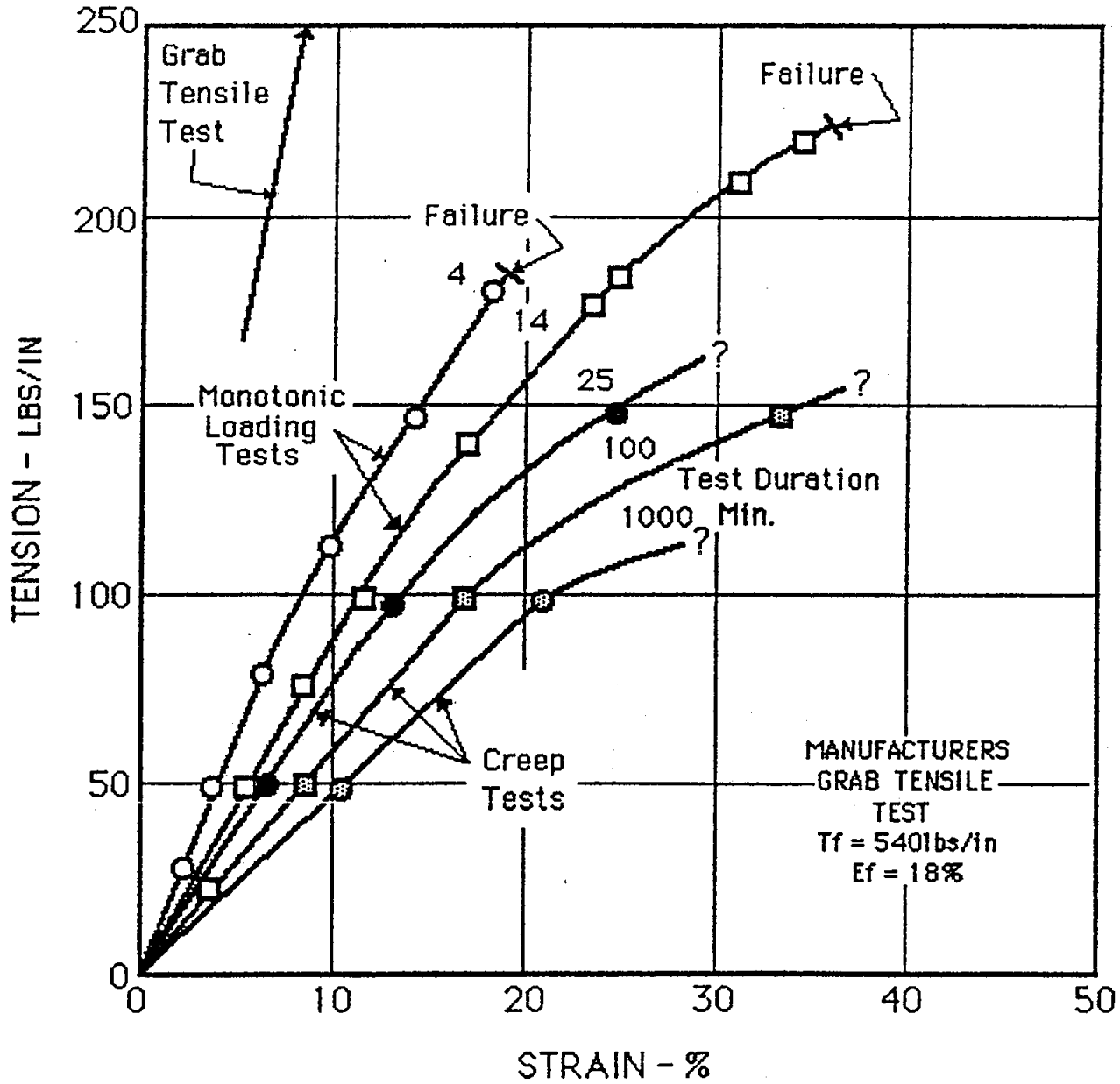
FIG. 8



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TENSION Vs. STRAIN
MATERIAL A

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Fig. 9

individual strands. A figure of 20 to 30 percent more is frequently used. This is evidently a poor assumption in the case of the geotextiles used herein since the rapid wide width tensile test yielded strengths that are about 1/2 the grab tensile test values.

The results of the constant load tests (creep tests) are also shown on Figures 8 and 9. Strain measurement are shown as a function of the applied load at a particular time. The curves shown are cross sections through several tests. The loading rates shown are the load on the sample divided by the time. It is evident from these tests that the rate of strain decreases with time under constant load. Tests were attempted at constant loads of 50,100 and 150 lbs./in. When the load was raised from one level to the next it was done farly rapidly and measurements were not made during the loading process. It is obvious from Figures 8 and 9 that the long term stiffness of the geotextiles used is much less than the stiffness reported in the literature.

3.2 FRICTION BETWEEN SOIL AND GEOTEXTILE OR GEOGRID

The trenches for the field test were cut through a fine sandy silt. Prior to placing the reinforcing material a thin layer of sand and gravel was placed over the area as a leveling course. It is difficult to determine the nature of the material under the reinforcing material because there was some mixing of the sand and gravel fill with the fine sandy silt during construction. It can be assumed that it is a mixture of the fine sandy silt and the sand and gravel. It is probably more like silt over Test Sections 2 through 4 and 6 through 8 and probably more like the sand and gravel over the other test sections. Tests were performed using each geotextile and the geogrid in combination with the fine sandy silt and the sand and gravel to develop a range of possible values.

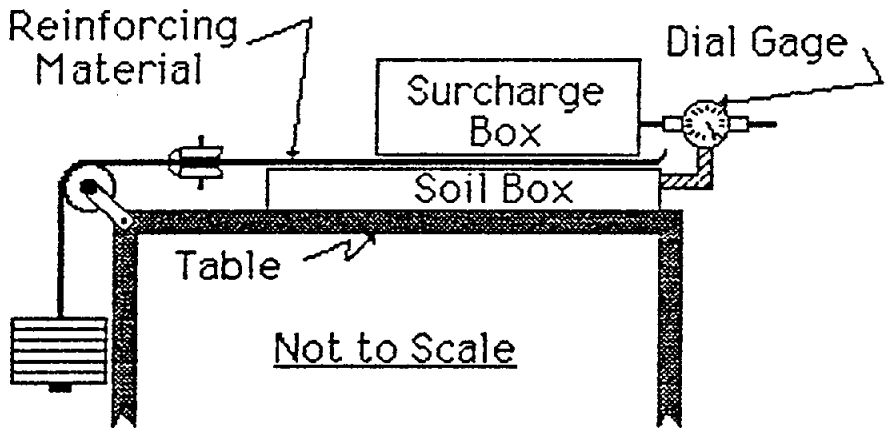
3.2.1 TEST FACILITIES

A special test facility was designed and constructed to measure the shearing resistance between the reinforcing material and the soil. It consisted of a soil box and a surcharge box as shown on Figure 10. The soil box was about 20 inches wide, 40 inches long and 4 inches deep. The surcharge box was about 15 inches wide, 36 inches long and 12 inches deep.

The soil which was to be used below the reinforcing material was placed in the soil box. The soil was placed at a known water content in a lightly compacted condition. The reinforcing material was placed on the soil. One end was attached to the clamp which was pulled laterally by a system of pulleys and dead weights. The surcharge box consisted of the four sides only. The surcharge box was placed on the reinforcing material and filled with sand and gravel. Extra weight was then added to the top of the sand and gravel with lead weights as needed.

Each test was performed by adding tension to the reinforcing material and measuring the movement of the surcharge box. Each load was held until there was less than 1/1000 inch movement in 1 minute. The test apparatus worked well at low normal loads, however, significant friction developed in the shearing mechanism at normal stresses in excess of about 1 psi.

The shear strength of the fine sandy silt was determined in an ELE direct shear machine. The shear box was 10 cm square and 4 cm high. The soil was placed in the shear box as a slurry and consolidated. The sample was then sheared at a rate of 1mm/min., to a displacement of 0.25 inches. Slower shearing rates were tried but with no apparent affect. After shearing, the halves of the shear box were slid back to their initial position and the procedure was repeated at a higher normal stress. Separate tests for each normal load were also tried but with no apparent effect.



Notes:

1) Surcharge box:

12" deep, 36" long, & 15" wide-No bottom
Box not attached to Reinforcing Material
Gravel placed on Reinforcing Material in box
Steel weights placed on gravel for extra surchage

2) Soil box:

4" deep, 48" long, & 20" wide
Soil to be tested is placed in the box at the desired water content and density.

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SOIL - REINFORCING MATERIAL
FRICTION TEST APPARATUS

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Fig.10

One standard consolidation test was performed on the fine sandy silt to determine the rate at which consolidation takes place and the equilibrium water content under various loads.

3.2.2 TEST RESULTS

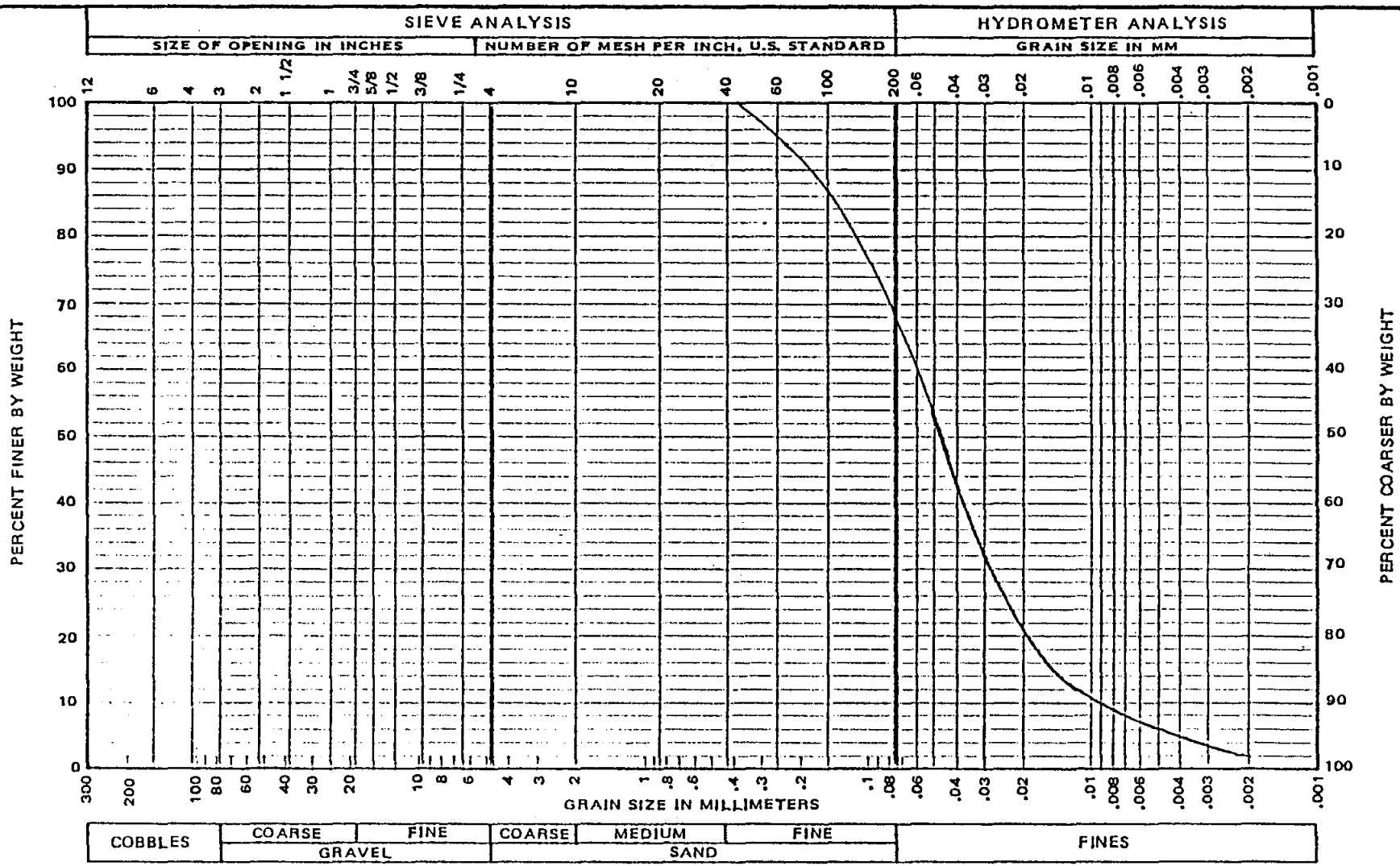
The test results are separated into two categories: one for the tests with the fine sandy silt on the bottom and sand and gravel on the top, and the other with the sand and gravel on both sides.

Fine Sandy Silt

The fine sandy silt has about 30 percent fine sand, all less than the #40 sieve, and no clay as shown on Figure 11. The relationship between water content and overburden pressure is shown on Figure 12. Ninety percent consolidation occurred within a few seconds. The water content under a consolidation pressure equal to that of 3 feet of sand and gravel is about 28 percent.

The shear tests with the reinforcing materials over the fine sandy silt were quite consistent. The shear stress increased rapidly over a displacement of a few hundredths of an inch and then increased slowly to a displacement of over 0.5 inches. One typical test is shown on Figure 13. In all tests the strength with the reinforcing material was greater than without and the difference appeared to be greater with higher initial water contents and rougher materials as shown on Figure 14.

A 39 degree friction angle was measured in the direct shear test on the fine sandy silt. The shear stress increased rapidly to a displacement of about 0.1 inches after which it continued to increase until a displacement of about 0.2 inches was reached. Past a displacement of 0.2 inches the shear stress remained relatively constant. Under a magnifying glass the material appears to be very angular which explains the apparently high friction angle.



SAMPLE NO.	DEPTH-FT.	U.S.C.	CLASSIFICATION	NAT. W.C. %	LL	PL	PI
1		ML	fine sandy SILT				

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GRAIN SIZE DISTRIBUTION

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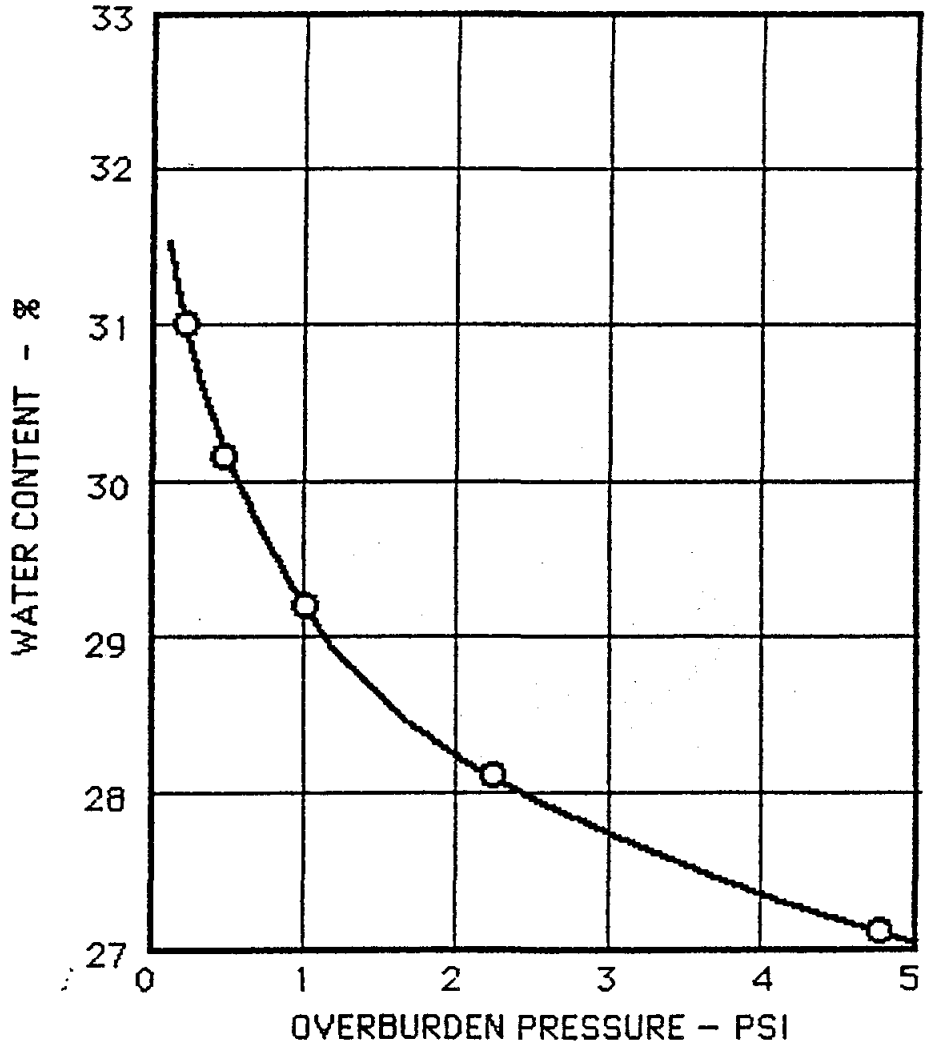
FIG. 11



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WATER CONTENT Vs.
OVERBURDEN PRESSURE
FOR FINE SANDY SILT

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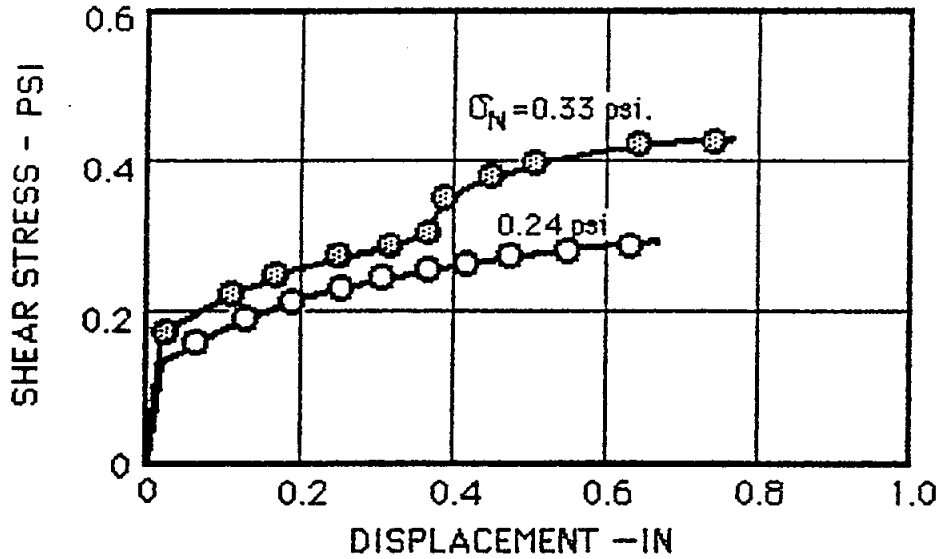
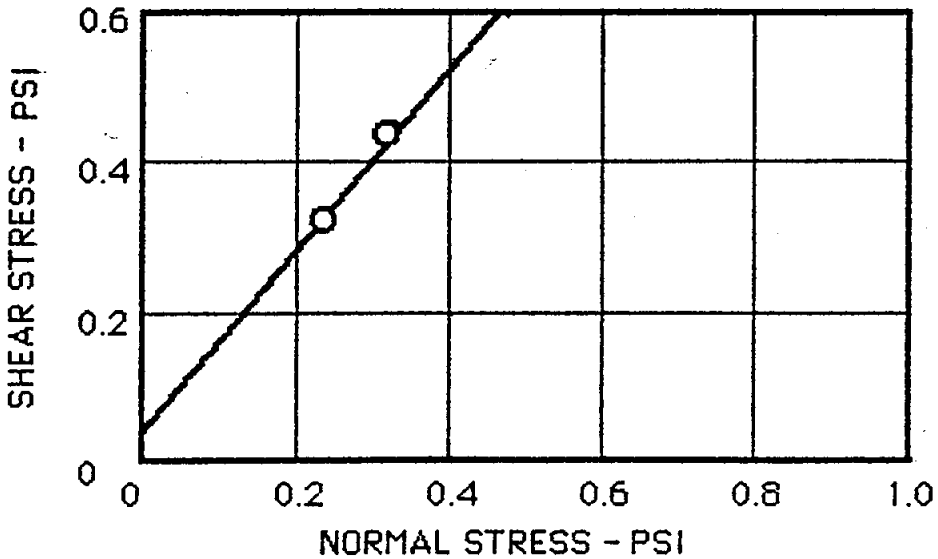
FIG. 12





Notes:
Water Content = 29%
Material A

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TYPICAL
LOAD - DISPLACEMENT
RELATIONSHIP BETWEEN
REINFORCING MATERIAL
AND FINE SANDY SILT

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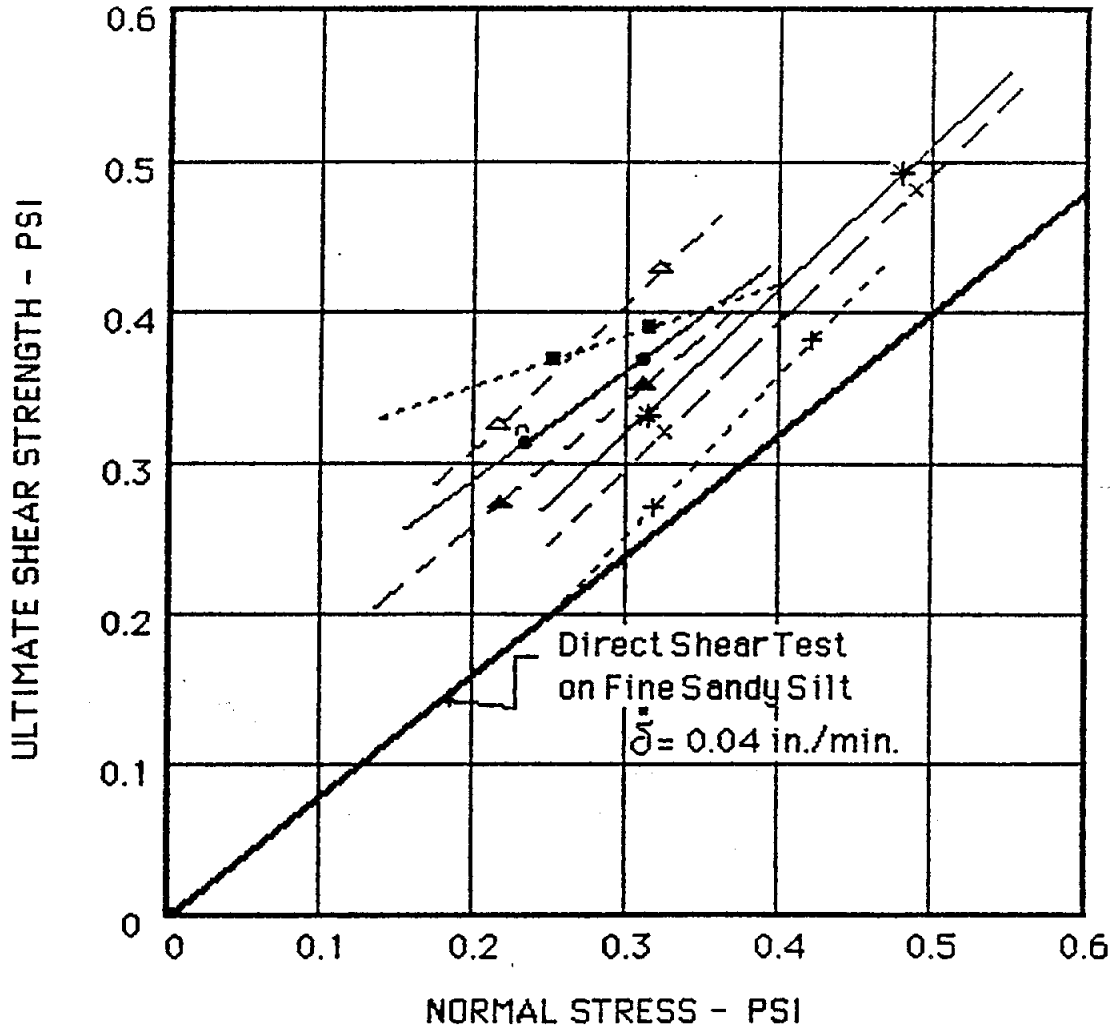
FIG. 13



SYMBOL	WATER CONTENT OF SOIL	REINFORCING MATERIAL	RELATIVE ROUGHNESS OF REINFORCING MATERIAL
△	NC	Material A	Smoothest
○	NC	Material B	Medium
▲	MOIST	Material A	Smoothest
●	MOIST	Material B	Medium
■	MOIST	Material C	Roughest
×	AIR DRY	Material A	Smoothest
+	AIR DRY	Material B	Medium
✳	AIR DRY	Material C	Roughest

NC = NORMALLY CONSOLIDATED

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SHEARING RESISTANCE BETWEEN REINFORCING MATERIAL AND FINE SANDY SILT

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FIG. 14

All of the tests with the reinforcing materials were run at normal stresses of less than 0.5 psi because of restrictions in the testing apparatus. The shear strength in this range appears to be significantly above the shear strength of the soil alone. If, however, the same test results are plotted on a large scale which includes the expected range of working stresses, as shown on Figure 15, it appears as though the strengths are only slightly higher and the differences could be attributed to differences in materials and in testing techniques as well as some small adhesion. A summary of the shearing resistance between the reinforcing materials and the fine sandy silt is shown on Figure 16.

Sand and Gravel

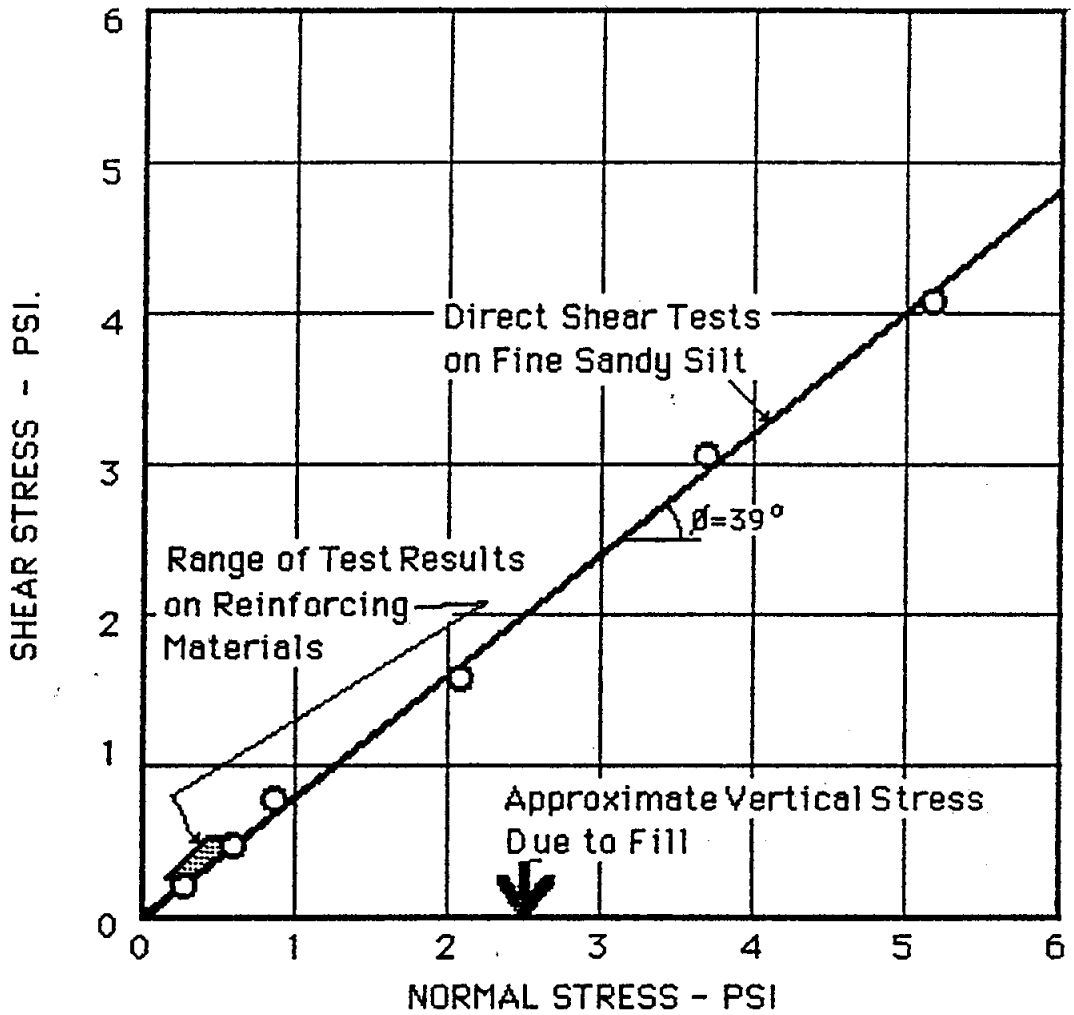
The test results for all of the tests with sand and gravel above and below the reinforcing material were all quite consistent. The shear stress increased rapidly for a few hundredths of an inch and then continued to increase slowly to a displacement of over 0.5 inches as shown on Figure 17. The shear strength was consistently lower than the anticipated shear strength of the soil alone and the shear strength increased with reinforcing material roughness as shown on Figure 18.

3.3 FULL SCALE FIELD TEST

The full scale field test was conducted along the north fence behind the Shannon and Wilson, Inc. complex at 2055 Hill Road in Fairbanks, Alaska. It consisted of 2.5 feet of alluvial sand and gravel over four wheel paths crossing three ice filled trenches. The width of the trenches and the type and length of the reinforcing materials covering the trenches were varied to form twelve test sections.

3.3.1 TEST FACILITIES

An area about 30 feet wide and 100 feet long was cleared along the fence in September 1983. Borings in the area indicate that there is about 6 to 8 feet of fine sandy silt overlying sand and gravel at the site. The



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SHEAR STRENGTH
OF FINE SANDY SILT

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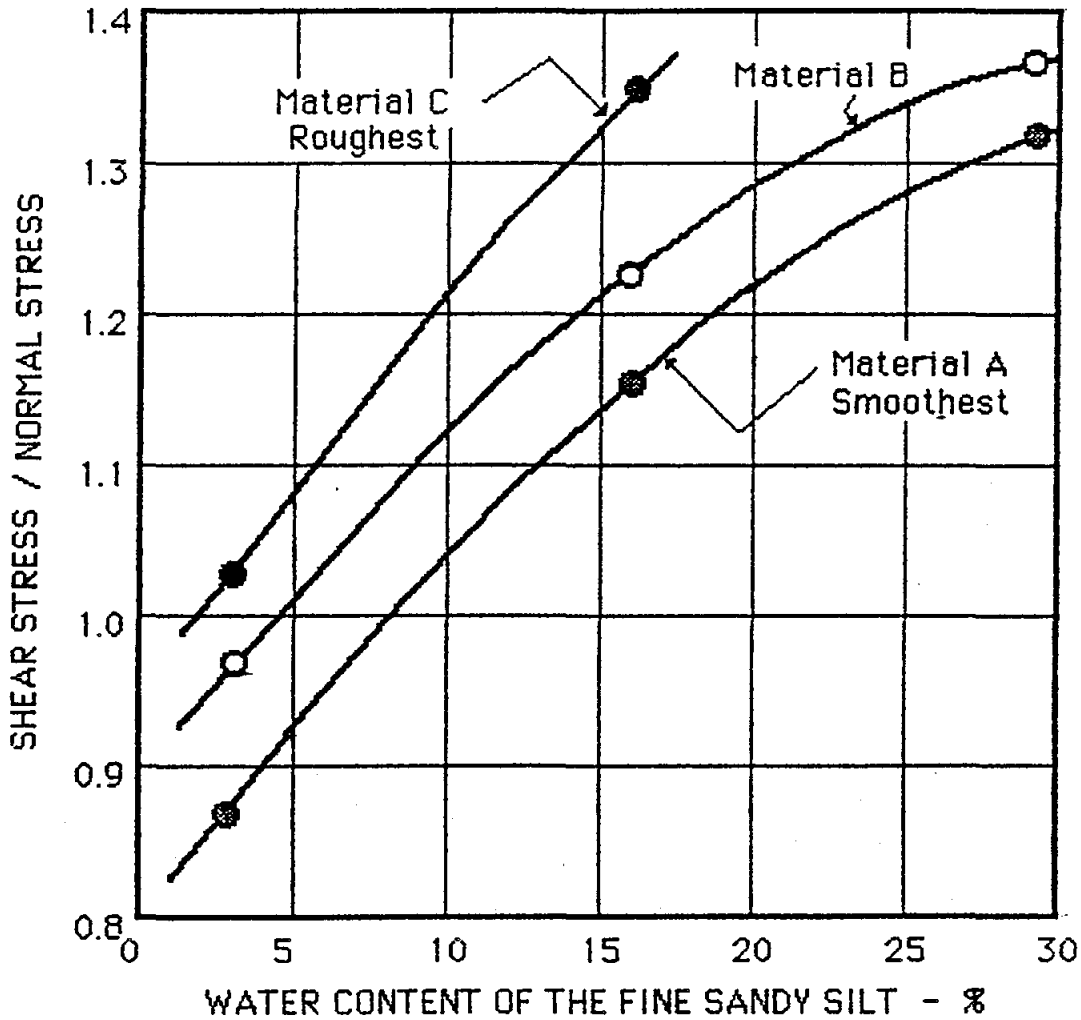
FIG. 15



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SUMMARY OF
SHEARING RESISTANCE
BETWEEN
REINFORCING MATERIALS
AND FINE SANDY SILT

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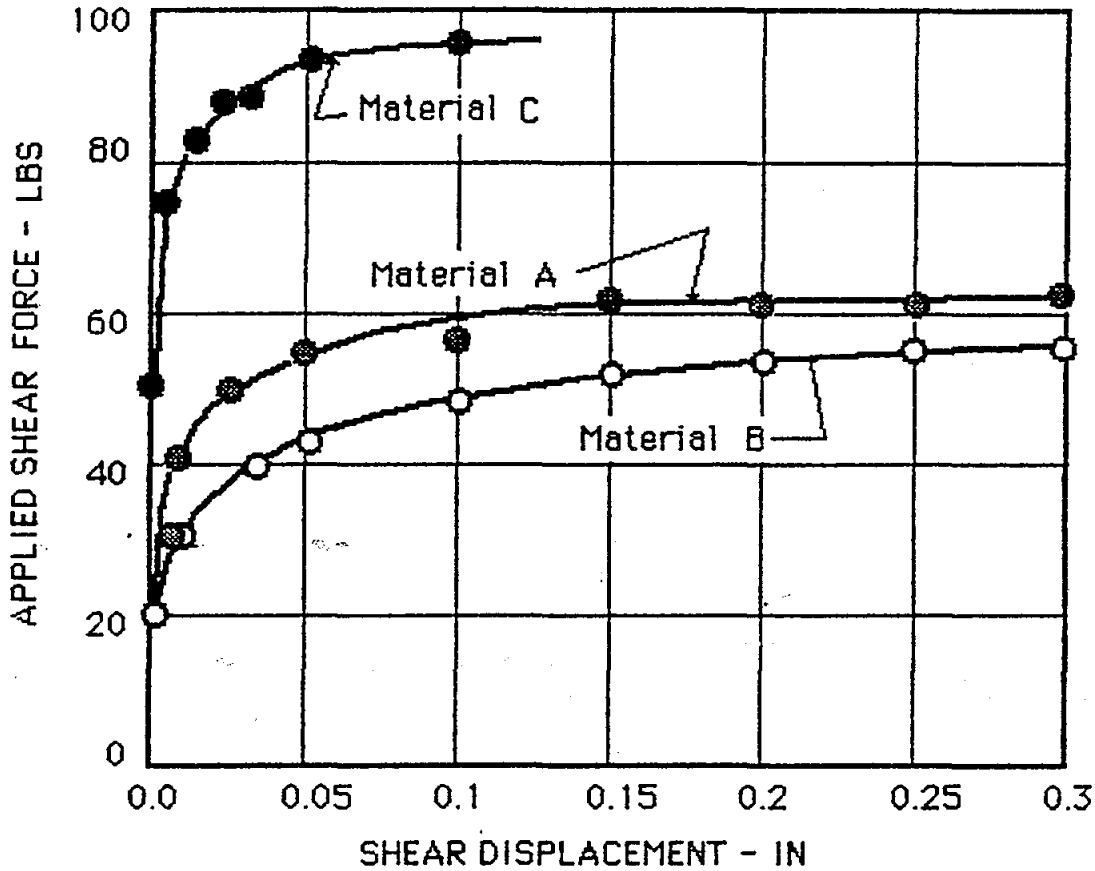
FIG. 16



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SHEARING RESISTANCE
BETWEEN
REINFORCING MATERIALS
AND SAND & GRAVEL

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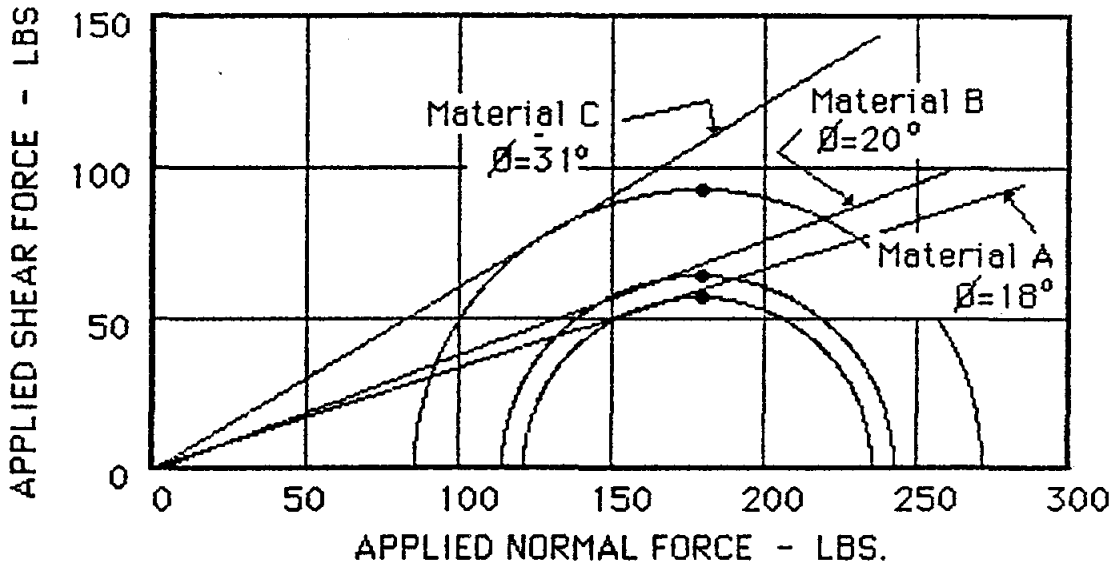
FIG. 17



RESEARCH SECTION
DEPARTMENT OF TRANSPORTATION
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SUMMARY OF SHEARING RESISTANCE BETWEEN REINFORCING MATERIALS AND SAND & GRAVEL

K-0617 MAY 1985

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FIG. 18

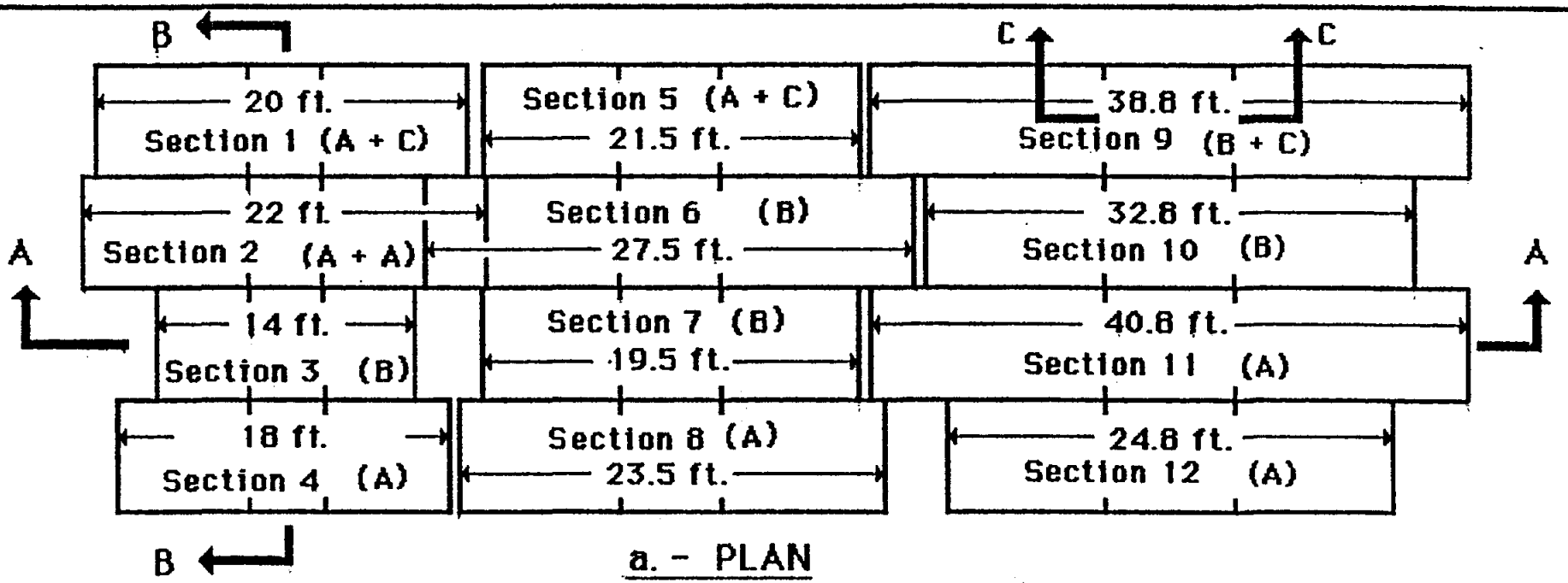
in situ water content of the silt is between 25 and 30 percent. The water table varies in depth from 12 to 14 feet.

Three trenches were excavated across the area in November 1983 as shown on Figure 19. A 6-inch wide trench was excavated along each side of each test trench using a trencher. The material in the center was removed using a backhoe. The trenches were initially 4.0, 5.5, and 6.8 feet wide and 2.5, 3.5, and 4.5 feet deep respectively.

Expandable settlement gages, shown on Figure 20, were placed at the bottom of each trench in the center of each test section. Water was then poured into the trenches and allowed to freeze at the rate of about 3 inches per week. It was noted that the water caused some thawing and minor sloughing even when it was placed in cold weather. The 4.0 foot wide trench is shown in Photograph 2.

Prior to breakup in the spring of 1984 about 200 yards of gravel were stockpiled at the site for use on the pad. At the first sign of thawing weather the test bed was leveled with a thin layer of sand and gravel. Most of the test bed received less than 2 inches while the northeast corner received up to 6 inches. The surface around Test Sections 1, 5, and 9 through 12 looked like sand and gravel whereas the surface over the other sections resembled a mixture of silt and sand and gravel.

The reinforcing materials were cut to length and marked with white lines across the width on a 12 inch spacing. The reinforcing materials were laid flat on the ground with the white lines directly under strings which were stretched between nails in batter boards on each side of the test area. The type of material and length of material on each test section are shown in Table 2. The reinforcing materials over the 4-foot trench are shown in Photograph 3 just prior to being covered with sand & gravel. A close up at the geogrid installation over Test Section 5 is shown in Photograph 4. The geotextile under the geogrid is to keep the geogrid from digging into the edges of the void.

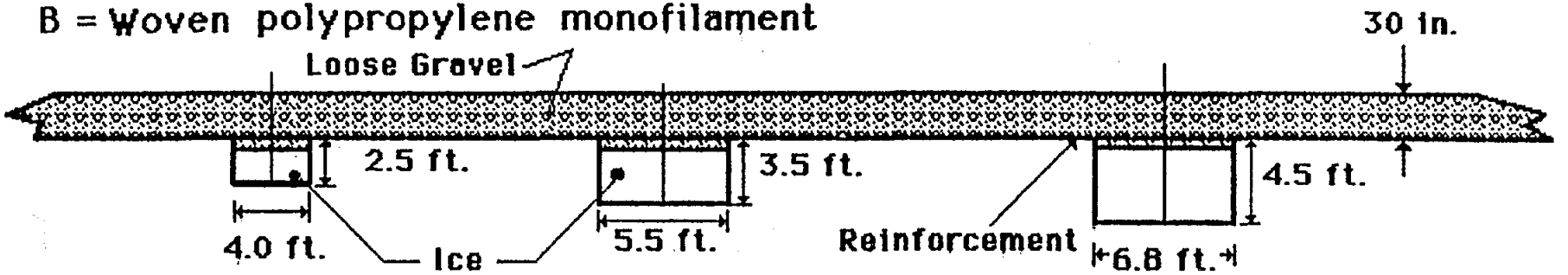


a. - PLAN

A = Woven polypropylene split film

B = Woven polypropylene monofilament

C = Two dimensional geogrid



Note:
See Figure 20 for
Sections A-A, B-B, & C-C

b. — SECTION A-A

FIGURE #2 — TEST FACILITY

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FIELD TEST SECTION
LAYOUT

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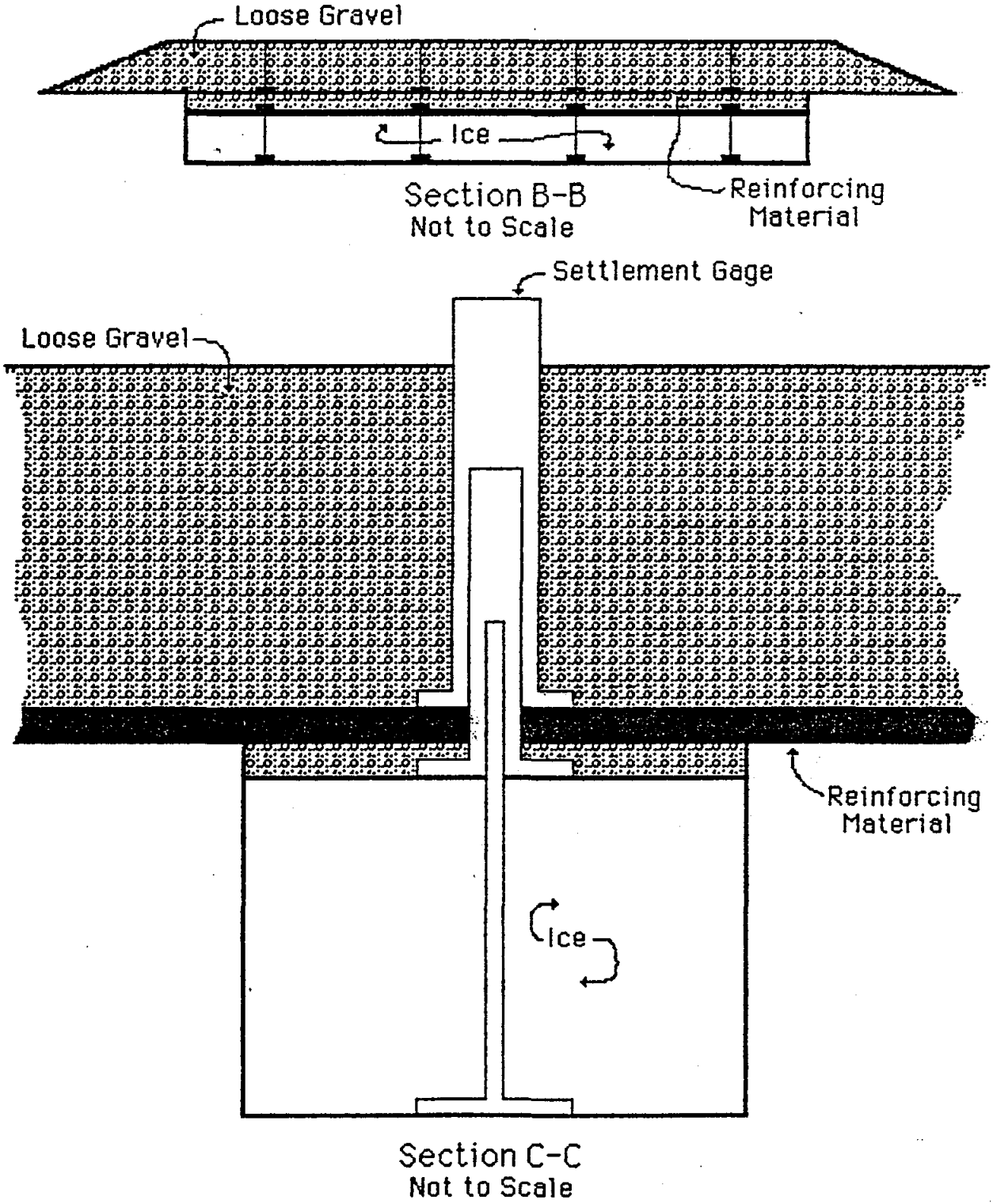
Fig. 19



RESEARCH SECTION
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EXPANDABLE SETTLEMENT PLATE DETAIL

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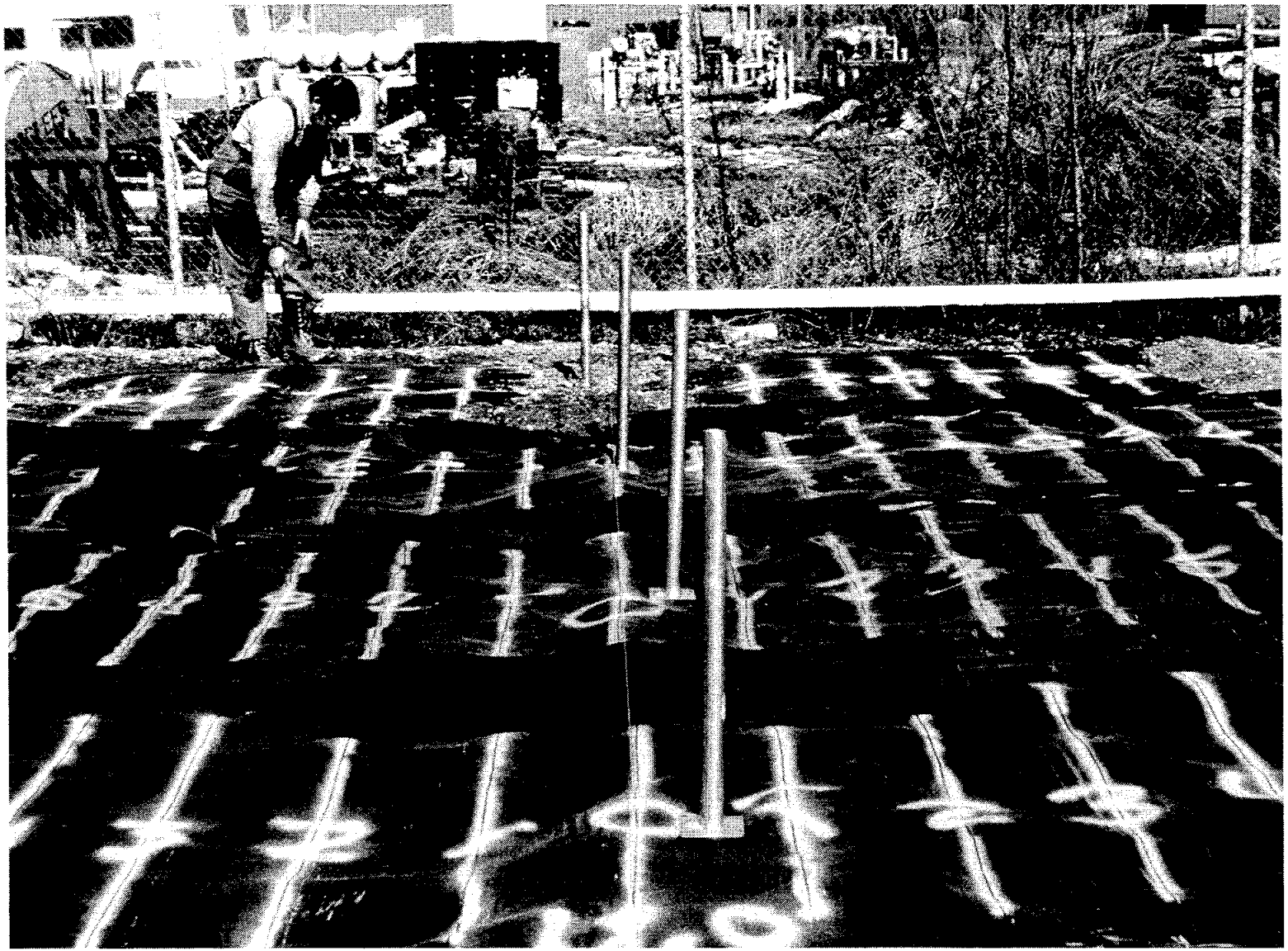
Fig. 20

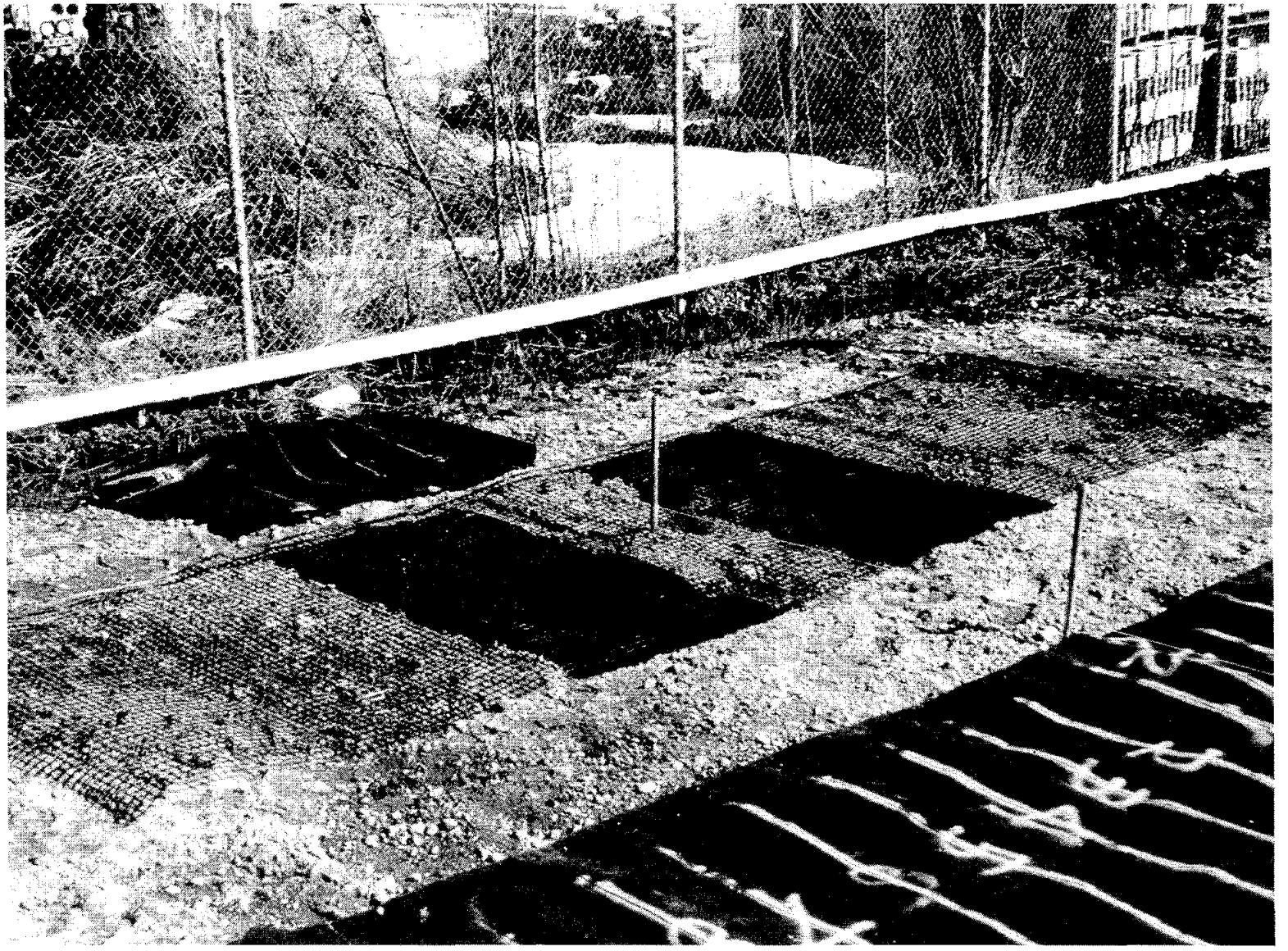
4 FOOT TRENCH FILLED WITH ICE

PHOTO 2



GEOTEXTILES OVER 4 FOOT TRENCH

PHOTO 3



GEOGRID OVER TEST SECTION 5

PHOTO 4

TABLE 2

DESIGN DETAILS OF TEST SECTIONS

Section No.	Trench Width ft.	Trench Depth ft.	Type	Reinforcing Material Width ft.	Reinforcing Material Length ft.
1	4.0	2.5	A + C	6	20
2	4.0	2.5	A (2 layers)	6	22
3	4.0	2.5	B	6	14
4	4.0	2.5	A	6	18
5	5.5	3.5	A + C	6	21.5
6	5.5	3.5	B	6	27.5
7	5.5	3.5	B	6	19.5
8	5.5	3.5	A	6	23.5
9	6.8	4.5	B + C	6	38.8
10	6.8	4.5	B	6	32.8
11	6.8	4.5	A	6	40.8
12	6.8	4.5	A	6	24.8

Small pieces of geotextile B, 2 feet wide and 3 feet long, were placed between the test sections to keep the overlying fill from falling between the reinforcing materials and into the trench. The pieces were placed in such a way that the gap was covered and yet they would have minimal effect on the stability of the test section. Sections of the expandable settlement gage were placed above and below the reinforcing materials as shown on Figure 20.

The reinforcing materials were covered with about 2 inches of sand and gravel and strings were placed on the sand and gravel directly under the strings stretched between the batter boards. The strings were covered with an additional 4 inches of sand and gravel by hand. The remaining fill was placed with earth moving equipment being careful not to disturb the position of the reinforcing materials or the strings.

Finally, 16 penny nails were pushed into the surface on a 1-foot grid over most of the test bed for horizontal and vertical measurements on the surface during thaw. The horizontal displacement of the nails was measured with a steel tape to an precision of 1/4 inch. The vertical displacement of the nails was measured with a surveyors level to an precision of 0.1 inch. The accuracy of displacements was clouded somewhat by the fact that the nails were pushed into loose sand and gravel which is not very stable. People and animals walking on the test sections disturbed some of the nails and, in the sections where large amounts of movement occurred, the nails were affected by surface ravelling of the sand and gravel.

There was a very warm break-up during the period when the test bed was being constructed. As a result there was up to about 4 inches of thaw of the ice during construction. Sand and gravel was used to create a level surface for test bed construction. This had the effect of giving somewhat less than perfect initial conditions but should not have affected the overall performance of the test sections.

Throughout the summer the settlement gages in the center of the test sections and the nails in the surface of the test bed were monitored and pictures were taken. The nails in the surface appeared to be giving a true picture of the surface movement that was occurring, and probing with a steel bar indicated that a void was forming as had been expected. The settlement gages in the center of each test section did not appear to be as reliable. The gage in Test Section 11 was destroyed during installation and the data from the gages in Test Sections 7, 10 and 11 yielded questionable results. The others appeared to give a reasonably accurate assessment of the reinforcing material displacement, however the data on the data on the surface of the ice was questionable and was not used in the report. Photographs 5, 6 and 7 were taken of the 4.0, 5.5 and 6.8 foot wide trenches respectively in late August.

On September 20, 1984 the test bed was leveled by adding about 40 yards of fill to the depressions and grading the surface with a small

4 FOOT TRENCH IN LATE AUGUST

PHOTO 5

5.5 FOOT TRENCH IN LATE AUGUST

PHOTO 6

6.8 FOOT TRENCH IN LATE AUGUST

PHOTO 7

bulldozer. Photograph 8 shows the 6.8 foot trench being filled. The surface was then compacted with 6 coverages of a vibratory steel wheel roller towed behind the bulldozer. This was the only formal compaction applied to the test bed and this was done only to make it easier to drive the dump truck on the surface and to observe the results during trafficking. No density tests were taken.

A dump truck was driven over the test sections 4 times when empty and 200 times when full. The truck was backed up across the test bed and then driven forward in the same wheel path. Each round trip caused two passes on each of six test sections. The truck had an 8 yard capacity and dual wheels on tandem axles in the rear. Photograph 9 shows the truck and the rutting that occurred. The weight of the truck is shown in Table 3.

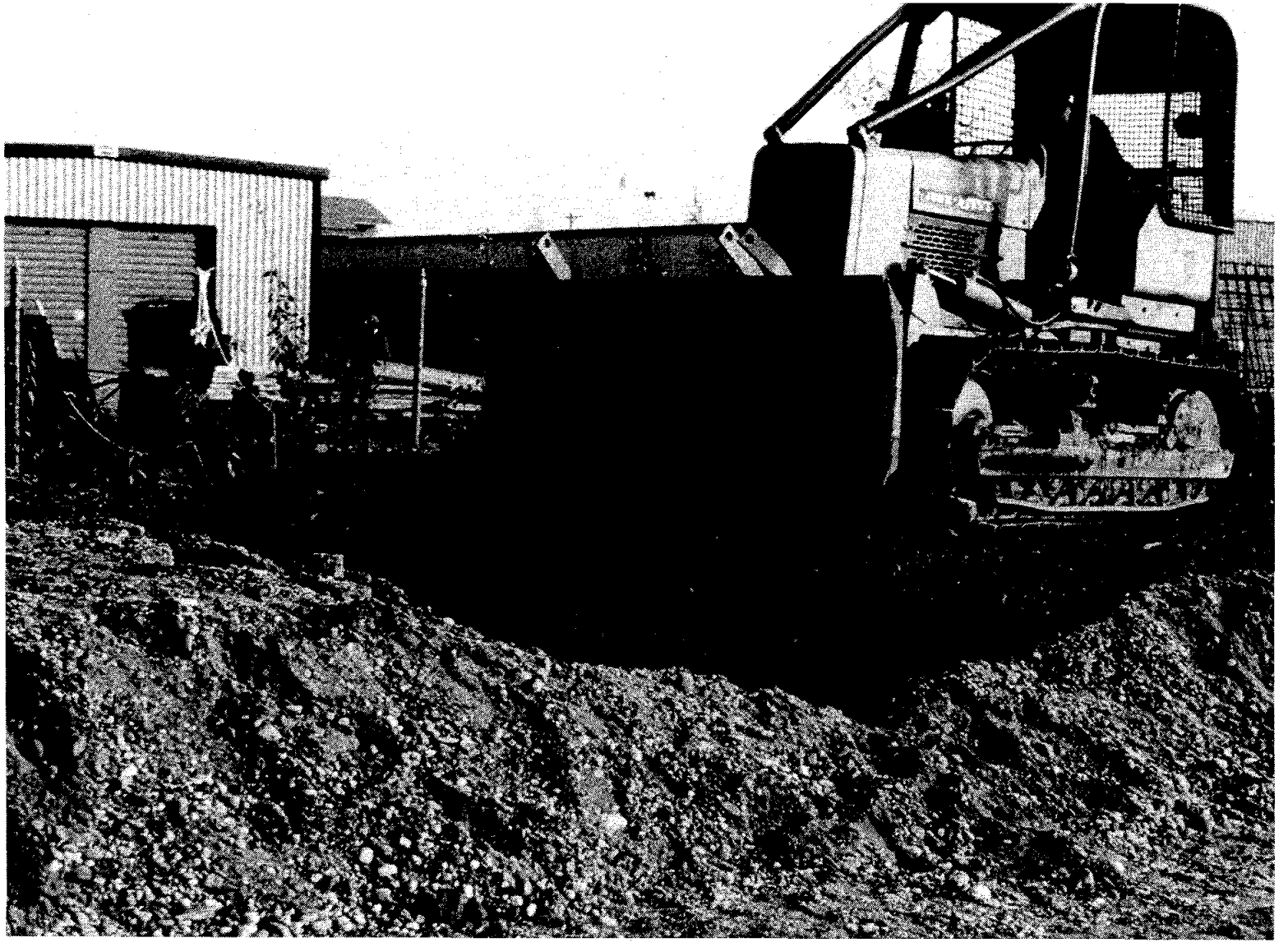
TABLE 3

WEIGHT OF TRUCK

Condition	Front Axle lbs.	Rear Axles lbs.
Empty	9,000	10,000
Full	13,000	31,000

Each pass was carefully observed by an engineer. Rut depth and shape measurements were made at selected times. The rut depth measurements were made by stretching a string along the wheel path and taking vertical measurements from the string. This eliminated much of the affect of compaction under the wheel load. Photograph 10 shows one of the ruts being measured.

After the trafficking was complete, the stiffness of the test bed was measured with a Benkelman Beam and a Falling Weight Deflectometer.

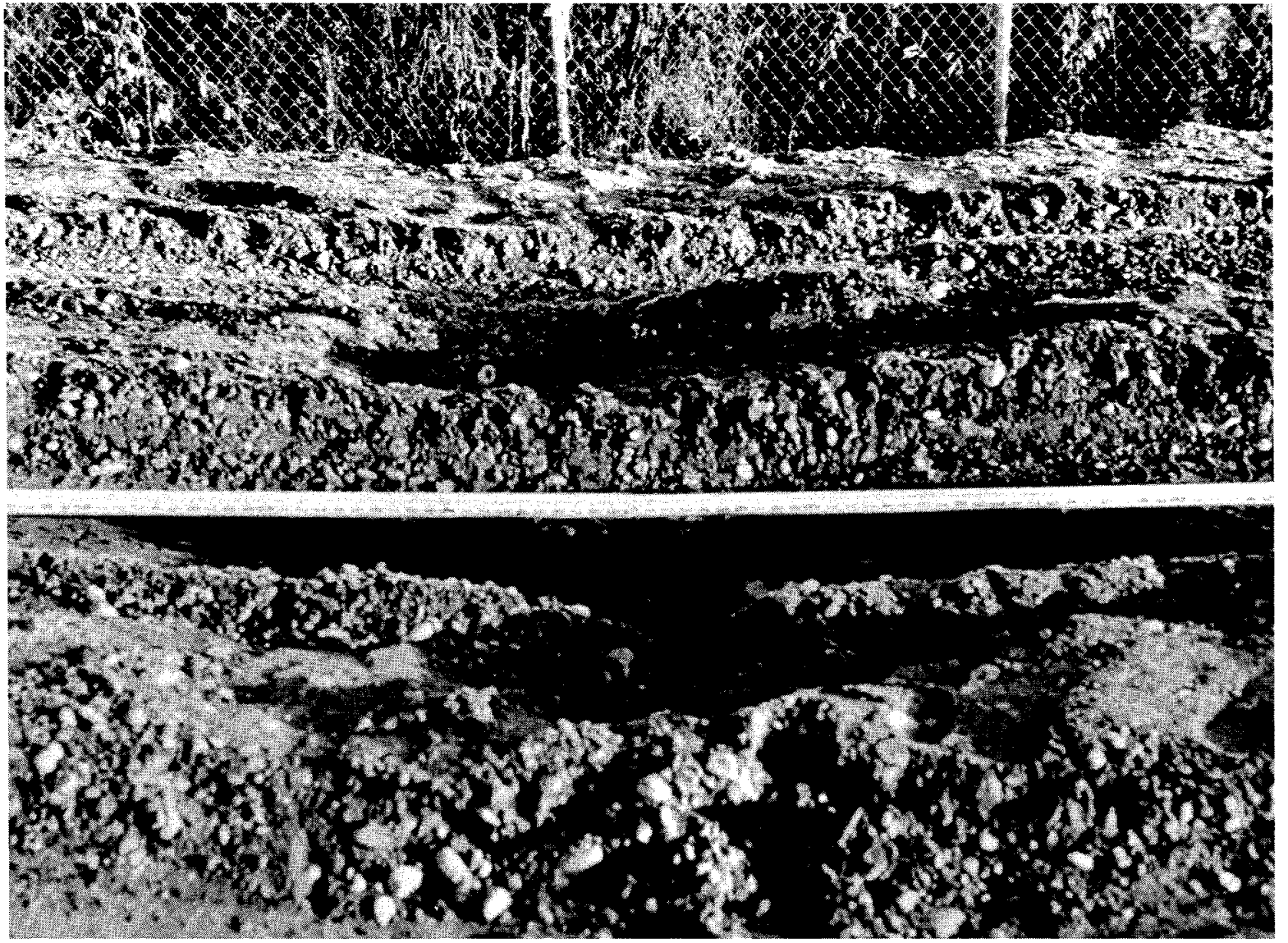


FILLING THE 6.8 FOOT TRENCH

PHOTO 8

RUTS DURING TRAFFICKING

PHOTO 9



SURFACE SETTLEMENT DUE TO TRAFFICKING OVER 5.5 FOOT TRENCH

PHOTO 10

Measurements were made in the wheel paths at the centerline of each test section and half way between the test sections.

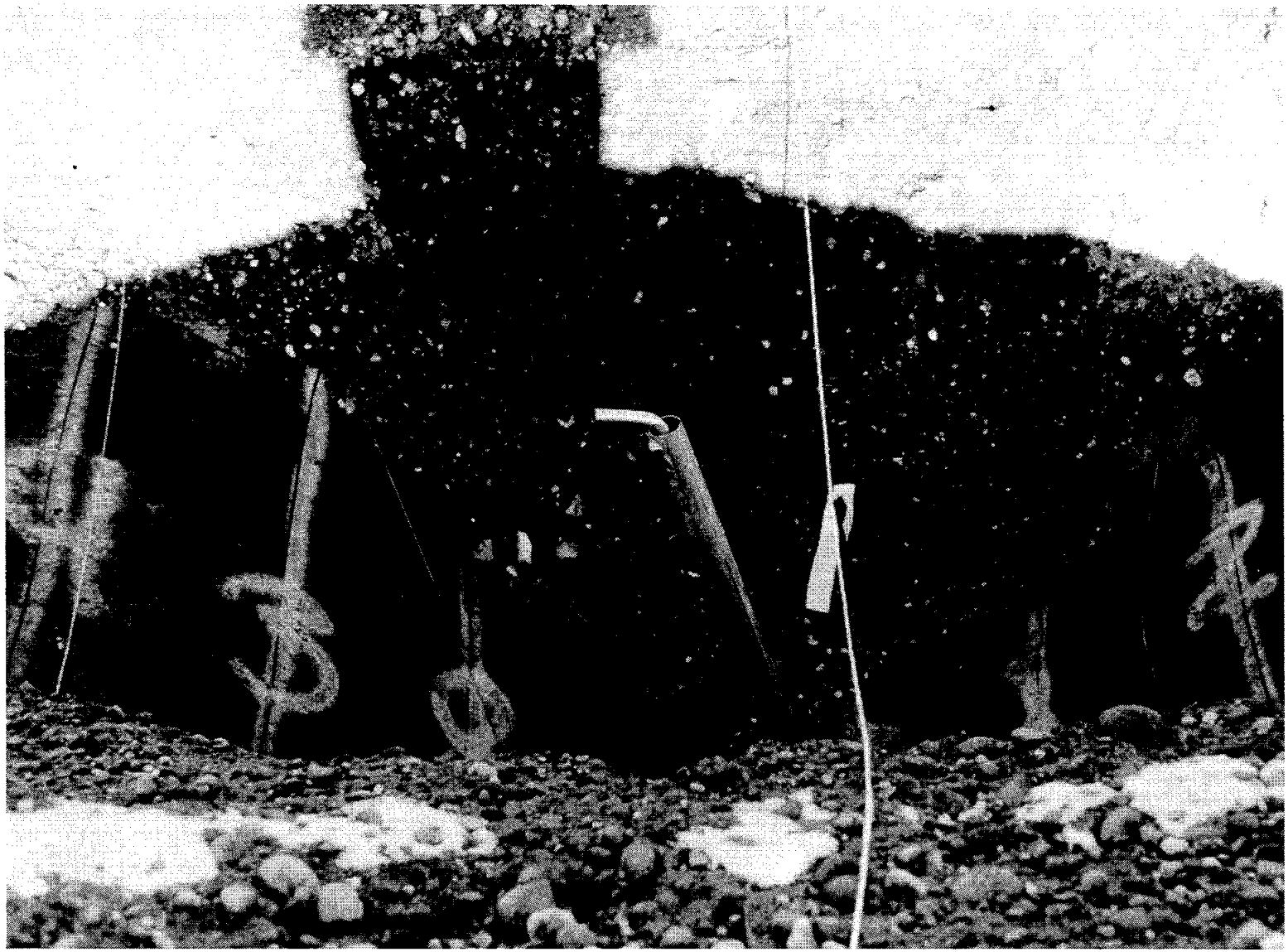
During the first week of October the test bed was dismantled. The upper 2 feet of fill was removed with a small bulldozer. A strip down the center of the length of each reinforcing material was cleaned by hand exposing the strings in the sand and gravel as well as the lines on the reinforcing materials. The strings between the batter boards were replaced and measurements were made from these strings to determine the horizontal and vertical displacement of the reinforcing materials and the horizontal displacement of the sand and gravel immediately above the reinforcing materials. Photograph 11 shows Section 6 after removal of the sand and gravel.

3.3.2 TEST RESULTS

The test results are divided into three distinct phases. The first relates to the behavior during the thawing period. During this period the loads are fairly well known and the deformations within the system including the maximum vertical displacement of the reinforcing materials were measured or can be inferred with reasonable accuracy.

The second phase relates to the behavior during trafficking. There was no way to determine the condition of the reinforcing materials after leveling and it could have been significantly different than it was before leveling because a considerable amount of fill was added. The trafficking results therefore could only be considered in a general way until the test bed was dismantled.

The third phase relates to the information gained during dismantling the test. This information is necessary to evaluate the trafficking portion of the test.



TEST SECTION 6 AFTER REMOVAL OF THE SAND AND GRAVEL

PHOTO 11

Thawing Period

During the thawing period the test progressed as expected. According to the settlement gages the ice thawed at a fairly constant rate of about 0.3 inches per day as shown on Figures B-1b through B-12b in Appendix B. The surface of the test sections settled with the surface of the ice until the strain in the reinforcing material was enough to allow it to carry the weight of the fill above. The movement of the surface of the fill over the trenches was toward the center of the trenches as shown on Figures B-1a through B-12a in Appendix B. The maximum vertical displacement measured at the center of each test section is summarized in Table 4.

TABLE 4

MEASURED SETTLEMENT AFTER THAWING

Test Section Number	Reinforcing Material Type	Trench Width (in)	Surface Settlement (in)	Reinforcing Material Settlement (in)
1	A + C	48	7	11*
2	A	48	7	13*
3	B	48	10	15*
4	A	48	15	15*
5	A + C	66	6	9+
6	B	66	11	22*
7	B	66	16	25*
8	C	66	16	22*
9	B + C	82	35	35+
10	B	82	38	38+
11	A	82	34	34+
12	A	82	34	34+

\+ Estimated

* Measured with settlement gages

Behavior During Trafficking

With two notable exceptions the trafficking was uneventful. In general the response to the load was not significantly different over the trenches than between the trenches. This was born out by the Falling Weight Deflectometer and the Benkelman Beam also.

The two exceptions are of significance. Test Section 1 failed abruptly on the sixth pass of the loaded truck. The rear duals dropped into a hole about 29 inches deep without warning. There wasn't any significant deformation under the first five passes.

Test Section 5 failed in an identical fashion but under the 38th pass of the loaded truck. The rear duals dropped about 21 inches without warning and, as in Test Section 1, there wasn't any significant deformation prior to failure. Photograph 12 shows the failure.

In both cases the hole was just the size of the tires. The holes were filled with sand and gravel and the test continued.

Measurements Made During Dismantling Test

Upon dismantling the test it was obvious that none of the sections had any significant void remaining. The geotextiles in Sections 2, 3, 4, 6, 7 & 8 were all intact but the geotextile had stretched or pulled out so that it was nearly on the bottom of the trench in each case. The reinforcing materials had failed in all the other test sections. Photograph 13 shows the edge of Test Section 8. Photographs 14 and 11 show Test Sections 9 and 6 respectively.

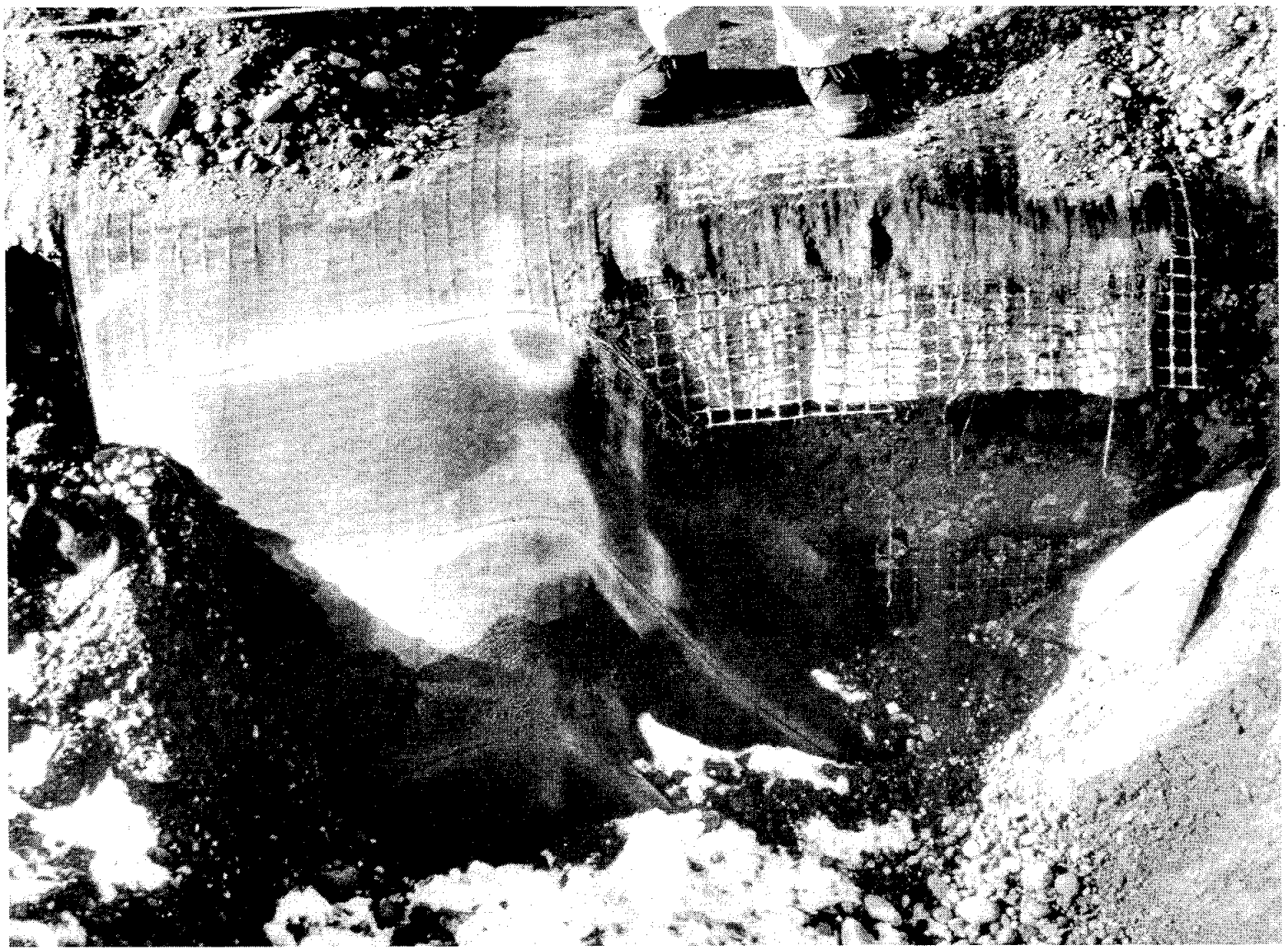
When the fill was removed from the trenches over the reinforcing materials that had not failed, there was a small void under each. This void was probably due to rebound of the reinforcing material giving the impression that it was not touching the bottom. The reinforcing

TRUCK AFTER FAILING TEST SECTION 5

PHOTO 12

TEST TRENCH AT END OF SECTION 8

PHOTO 13



TEST SECTION 9 AFTER REMOVAL OF THE SAND AND GRAVEL

PHOTO 14

material probably touched the bottom of the trench over at least part of its length in each section when under load.

After dismantling the test sections it was found that the trenches were not as deep as originally constructed. It appeared as though the fine sandy silt was about 3 to 6 inches higher than expected and that there was about 6 to 12 inches of sand and gravel over that. The fine sandy silt probably washed into the trench during filling with water in the fall and/or during thawing in the spring. Two to 6 inches of the sand and gravel could have resulted from the initial placement of the reinforcing materials. The remainder evidently fell through the split between the sections when the depressions were filled prior to trafficking or as a result of those sections that failed.

It was interesting to note that the walls of the trenches were intact with no more than a few inches of rounding at the top. There is a layer of sand and gravel between the water table and the bottom of the fine sandy silt which probably acted as a capillary break, thereby limiting frost heaving. In the absence of excess water the fine sandy silt is apparently quite strong.

The amount of movement that each line on the reinforcing material underwent and the amount of movement that each string in the sand and gravel over the reinforcing material underwent is shown in Appendix C on Figures C-1 through C-12. The measured residual strain at the centerline and the movement of the reinforcing materials are summarized in Table 5.

TABLE 5

RESIDUAL STRAIN AND MOVEMENT OF REINFORCING MATERIAL AFTER TEST

Test Section Number	Reinforcing Material Type	Strain - % Average Across Trench	Strain - % At Edge of Trench West	Strain - % At Edge of Trench East	Movement - in. At Edge Of Trench West	Movement - in. At Edge Of Trench East	Movement - in. At End Of Material West	Movement - in. At End Of Material East
1	A + C	*	2.0	2.9	2.5	2.5	2.0	1.0
2	A	1.7	3.5	0.9	4.8	4.6	3.3	4.0
3	B	5.0	2.2	2.8	5.1	5.3	4.3	4.2
4	A	5.0	2.3	3.2	4.4	4.8	3.6	3.3
5	A + C	*	5.4	3.2	7.4	7.2	3.1	4.9
6	B	8.3	2.8	3.3	4.9	7.0	3.5	4.0
7	B	18.3	2.3	2.4	6.1	6.3	4.9	4.7
8	C	5.0	5.4	3.2	7.4	7.1	3.1	5.1
9	B + C	*	1.2	2.6	1.1	1.3	0.0	0.0
10	B	*	7.0	9.8	3.5	3.5	2.0	*
11	A	*	7.3	15.8	10.5	10.5	6.7	*
12	A	*	12.5	16.2	8.7	9.2	4.5	*

* Data Lost

There is a considerable amount of scatter in the data making it difficult to establish meaningful trends. It was intended that the movement of a point on the reinforcing material would be measured to an accuracy of 1/4 inch and a point in the sand and gravel to an accuracy of 1/2 inch. Due to inherent difficulties in construction and dismantling the test facility the accuracy was probably reduced to about 1 inch on the reinforcing material and 2 inches in the sand and gravel.

4. DISCUSSION OF TEST RESULTS

The tests performed in this study provide considerable insight into the design of geotextiles and related products to span voids under roads. The field test did not perform as intended because the geotextiles were weaker and stretched more than anticipated from the manufacturers' published literature. Using the laboratory test data on the geotextiles, the results obtained in the field test can be explained. The following discussions describe these comments in more detail.

4.1 GEOTEXTILE PROPERTY TESTS

The tests on the geotextiles demonstrated that the strength and tension-strain characteristics of the geotextiles used are highly dependent on the type of test and the rate of loading. The grab tensile test has been widely used in the textile industry to determine the strength and the tension-strain characteristics of textiles. The test consists of grabbing a 3 inch wide sample with 1 inch wide jaws and pulling them apart at a fairly rapid rate. The results appear to be meaningful for comparing various fabrics to determine which one will hold up the best in the elbow of a shirt. Unfortunately the test does not relate well to the mode of use in most geotextile applications. Equally as unfortunate is the fact that the grab tensile test has, until quite recently, been the industry standard for lack of something better and is frequently the only test result available on which to choose a geotextile.

This problem was recognized by ASTM in 1979 in the first meeting of the newly formed committee on geotextiles (D-13/D-19). Many tests were studied by the committee and the wide width tensile test became the favorite by the early 1980's. The wide width tensile test consists of stretching a sample which is 8 inches wide and 4 inches long at a desired rate of strain. This test was voted on by ASTM in March of 1985 and will soon be a standard. The geotextile committee has its own number now, D-35, and is publishing the approved geotextile test methods along with the soils test methods in Volume 04.08 of the ASTM Standards.

It is currently used extensively for developing geotextile design parameters. The length to width ratio is particularly important in nonwoven geotextiles and is less important in woven geotextiles. The geotextiles tested as part of this study were 4 inches wide and 9 inches long. The apparatus was built before the wide width tensile test was well developed. In our opinion the test used is valid for use in this study.

The problem of creep in geotextiles has been discussed in general terms as long as people have been doing geotextile designs. In spite of its importance to the design and performance of tensile reinforced structures, there has been surprisingly little quantitative work done on the subject. It has been established that creep is highly dependant on the polymers and manufacturing technique used as well as on such things as temperature, humidity and stress history, not to mention the possible effects of confining pressure and soil impregnation. No consensus has been reached regarding testing standards other than it is generally accepted to use the wide width tensile test and to try to duplicate the field conditions in the test. No manufacturer is publishing creep data on geotextiles at the present time.

The manufacturers' published data upon which the materials were chosen for tests in this study consisted of polymer type and manufacturing technique for all materials, grab tensile strength and elongation at failure for the geotextiles, and wide width tensile strength and elongation at failure for the geogrid. The reported data are shown in Table 1 in this report.

The manufacturer indicated that the geogrid should not creep significantly due to the manufacturing technique used and therefore the published wide width tensile test data was assumed to give reliable values for design. The reported characteristics for the geotextiles were not considered to be as accurate. The grab tensile strength was reduced by 33% in an attempt to account for the difference in testing techniques and the creep that polypropylene is known to exhibit. All materials

were assumed to be linearly elastic upon loading. All data interpretation was done by estimation. The values used in design of the test facility are shown in Table 6:

TABLE 6

REINFORCING MATERIAL TENSION-STRAIN RELATIONSHIPS ESTIMATED FROM MANUFACTURER'S LITERATURE

Reinforcing Material Type	Failure Tension lbs/ft	Failure Strain %	Elastic Modulus lbs/ft
A	4300[1]	18	23,900[2]
B	3100[1]	30	10,300[2]
C	1230	10	12,300[2]

1 Manufacturer's Grab Tensile Test Data Divided by 1.5

2 Failure Tension/Failure Strain

After reviewing the test data described in Section 3.1.2 it appears that the design values chosen based on the published data for the geotextiles were considerably in error. It is assumed that the properties for the geogrid were reasonably accurate. The values shown in Table 7 appear to be reasonable based on the laboratory tests conducted and were used in the analysis of the field test.

TABLE 7

REINFORCING MATERIAL TENSION-STRAIN RELATIONSHIPS USED IN FIELD TEST ANALYSIS

Reinforcing Material Type	Failure Tension lbs/ft	Failure Strain %	Elastic Modulus lbs/ft
A	1800	50	5,000
B	1500	30	6,500
C	1230	10	12,300

The values used for analysis of the field tests should not be used for any other application. The testing was carried out in enough detail to satisfy the requirements of this analysis only and was not done in enough detail to develop properties for general use. The general conclusion is valid though manufacturers' published grab tensile test data may not be sufficient for reinforcement design purposes.

4.2 SOIL-GEOTEXTILE FRICTION

Several studies have been performed in an attempt to characterize the general relationships involved in the shearing resistance between various soils and various reinforcing materials. The following general guidelines have evolved:

1. If the size of the void spaces in the reinforcing material is of the same order of magnitude or larger than the size of the soil grains, the failure will be through the soil and not at the contact. It is not uncommon for the shearing resistance with a reinforcing material to be greater than the shearing resistance through the soil alone in a direct shear test.

2. If the size of the void spaces in the reinforcing material is smaller than the size of the soil particles, the failure may

be along the contact. If the failure is along the contact, the shearing resistance will be a function of the relative roughness of the reinforcing material and the soil as well as the shear strength of the soil.

3. If conditions are such that the plane of the reinforcing material is rough, it may force the failure through the soil rather than at the contact. This is likely if the underlying soil is soft, the overlying soil is coarse grained, the reinforcing material is drapable, and there is enough vertical stress to force the roughness of the overlying material through the reinforcing material and into the underlying soil.

In the field test the reinforcing material had a coarse grained sand and gravel on the top and either the same material or a combination of it and a fine sandy silt on the bottom. The fill on the top was generally expected to move with the reinforcing material and not slide relative to it. The laboratory tests were designed to test this situation.

The validity of the general guidelines found in the literature was born out in the tests performed during this study. In those tests where the reinforcing material rested on the fine sandy silt, the shearing resistance between the reinforcing material and the soil was consistently slightly higher than the shear strength of the soil alone as shown on Figure 16.

In those tests where the reinforcing material rested on the sand and gravel the shearing resistance was less than the assumed shear strength of the soil with the slicker, less drapable geotextile having the lowest value as shown on Figure 18. When the geogrid was placed between the geotextile and the sand and gravel, the shearing resistance was about that expected for the soil alone. In all tests the shearing resistance reached nearly its peak value at a relative displacement of about 0.1 inches. The maximum shearing resistance used in analysis of the tests was expressed as a friction angle and is shown in Table 8.

TABLE 8

MAXIMUM FRICTION BETWEEN REINFORCING MATERIAL AND SOIL USED IN ANALYSIS OF FIELD TEST RESULTS

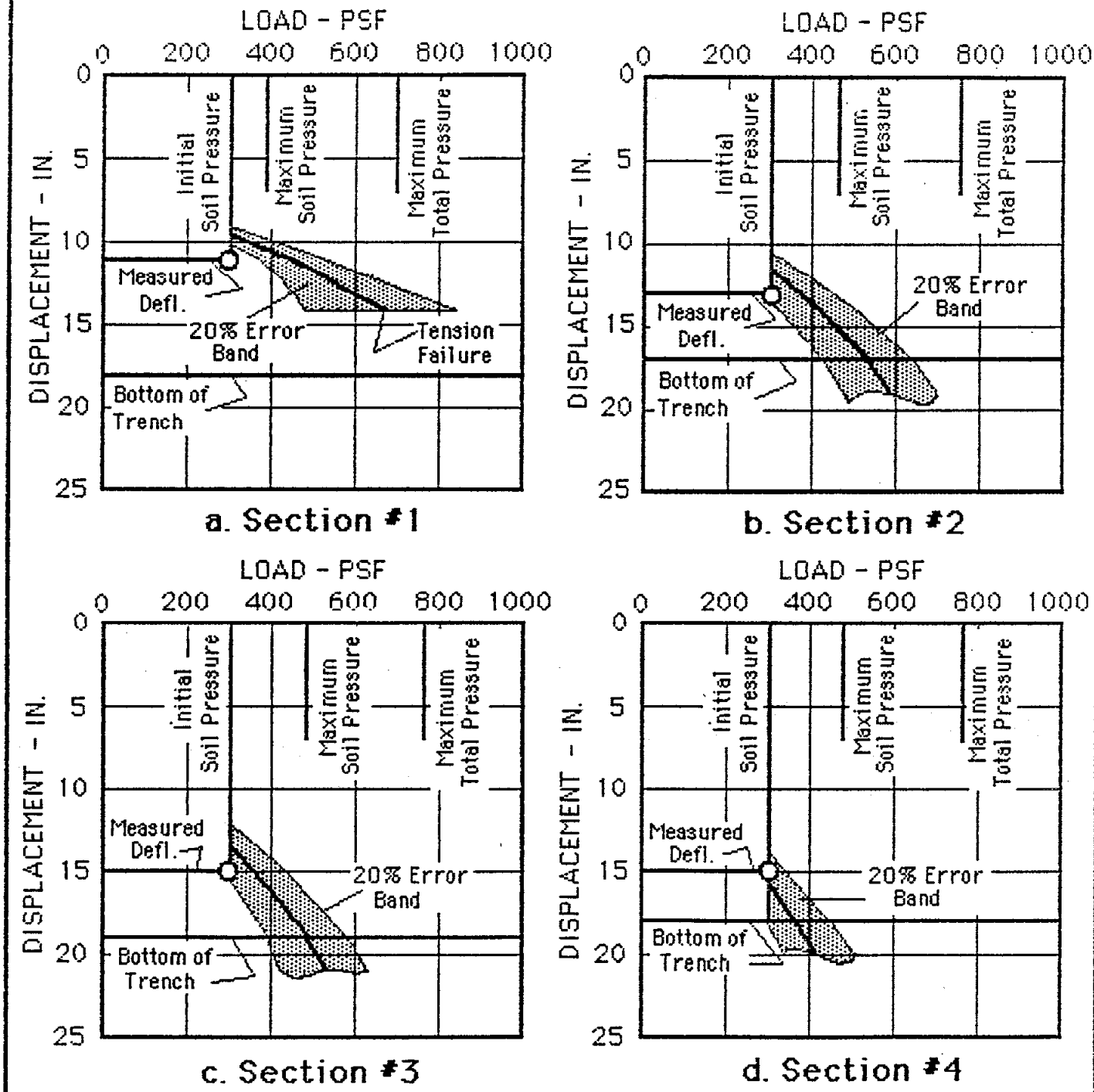
Reinforcing Material Type	Angle of Shearing Friction
A	19°
B	21°
C	31°

4.3 FIELD TESTS

The initial phase of the field test progressed as anticipated using the tested material properties. The ice melted and the reinforcing materials spanned the void space after reaching an equilibrium condition of tension, slippage, strain and displacement. The anticipated conditions of settlement and tension in the reinforcing materials were calculated using the theoretical model presented in Section 2 and estimated material properties based on the test results presented in Tables 7 and 8. The results are shown on Figures 21 through 23 as pressure over the void versus centerline displacement relationships for each section.

In performing the calculations it was assumed that the fill was 30 inches thick and weighed 120 pcf. The total shear stress was assumed to be the maximum possible on the bottom and 1/2 of the maximum on the top.

In Test Sections 1, 2, 5 and 9 where two materials were used, the elastic modulus of the materials were added and the failure tension was weighted to reflect the tension at the failure strain for the material that fails at the lowest strain. There is very little evidence that this technique is valid and there may be slippage between the two materials.



Notes:

1. The measured deflection is the last reading from the settlement gage prior to filling the depression.
2. The bottom of trench was measured after dismantling the test sections.
3. The maximum soil pressure was estimated to be the pressure after filling the depressions.
4. The maximum total pressure includes the pressure due to the fully loaded truck.
5. The 20% error bound was calculated using the best estimate of the shear stress, modulus, and the tensile strength all multiplied by 0.8 and 1.2.

RESEARCH SECTION
DEPARTMENT OF TRANSPORTATION
AND PUBLIC FACILITIES

CALCULATED
LOAD - DEFLECTION - FAILURE
RELATIONSHIPS, SECTIONS 1-4

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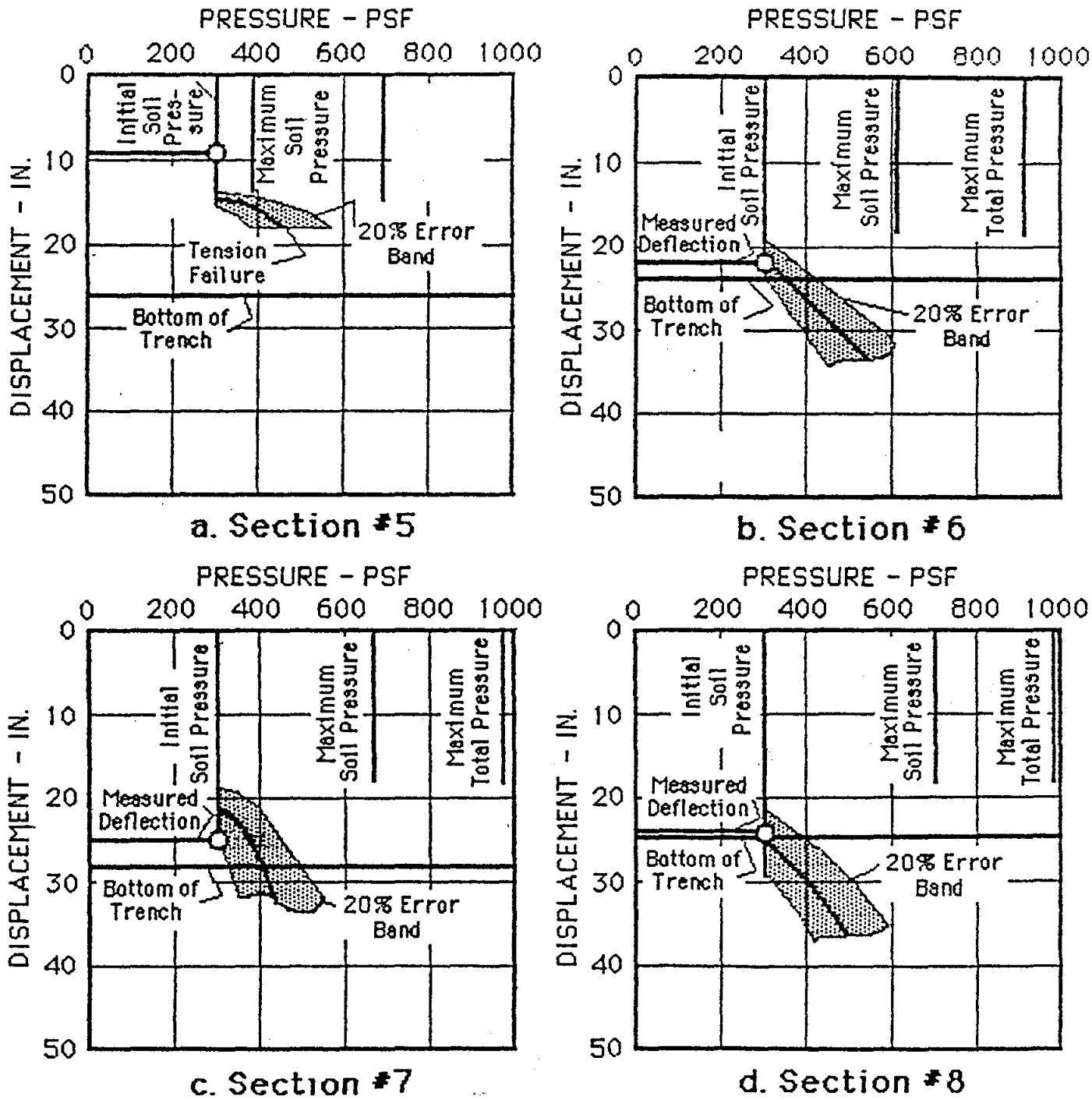
Fig. 21



Notes:

1. The measured deflection is the last reading from the settlement gage prior to filling the depression.
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4. The maximum total pressure includes the pressure due to the fully loaded truck.
5. The 20% error bound was calculated using the best estimate of the shear stress, modulus, and the tensile strength all multiplied by 0.8 and 1.2.

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CALCULATED
LOAD - DEFLECTION - FAILURE
RELATIONSHIPS, SECTIONS 5-8

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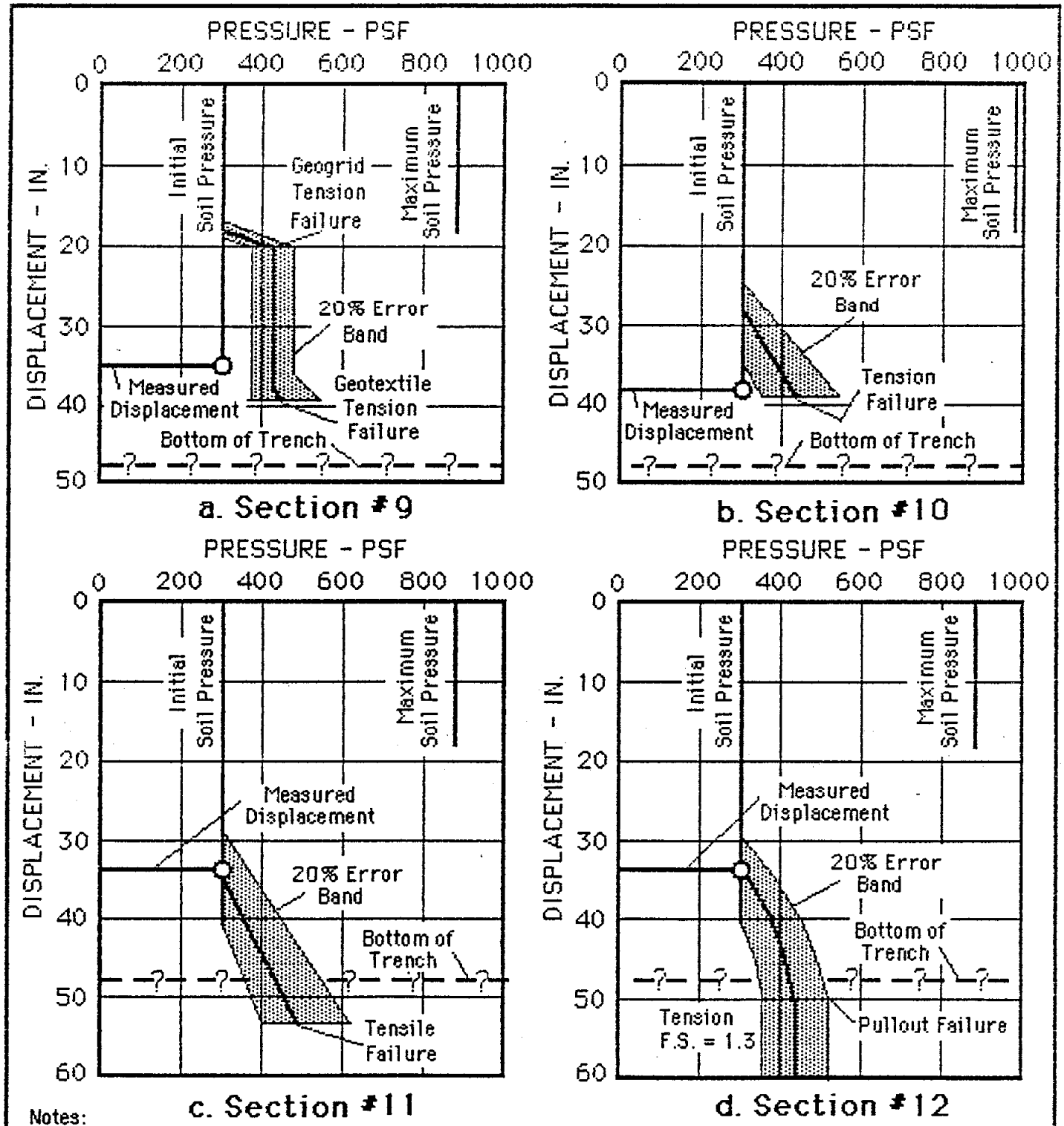
Fig. 22



Notes:

1. The measured deflection is the last reading from the settlement gage prior to filling the depression.
2. The bottom of trench was measured after dismantling the test sections.
3. The maximum soil pressure was estimated to be the pressure after filling the depressions.
4. The maximum total pressure includes the pressure due to the fully loaded truck.
5. The 20% error bound was calculated using the best estimate of the shear stress, modulus, and the tensile strength all multiplied by 0.8 and 1.2.

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CALCULATED
LOAD - DEFLECTION - FAILURE
RELATIONSHIPS, SECTIONS 9-12

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Fig. 23

Calculations were also made using the original shear stress, modulus, and failure tension values multiplied by 1.2 and 0.8 to give an error bound. The range of values give the load - displacement relationships shown in shading on Figures 21 through 23.

The observations made during thawing agree with the calculations in that the materials could and did carry the weight of the fill over the void. The calculated centerline displacement agrees well with the measured displacement for all test sections except Section 9. One explanation for this discrepancy may be that the geogrid failed during thawing forcing the geotextile to carry the load. The calculated factor of safety without slippage between the layers is 1.33. If slippage occurred between the layers this could have been reduced causing failure.

The calculations for required anchorage indicate that there was enough anchorage on every test section except perhaps Sections 3, 7 and 12. Although there is no way of telling, it is possible that the reinforcing material in these test sections had started to slip at the ends during the thawing phase of the test.

No attempt was made to measure the fill surface or the reinforcing material shape during the time when the depressions were being filled before trafficking. Based on the performance of the test sections during trafficking and the observations made after the test bed was dismantled it appears as though the reinforcing materials in all of the 6.8 foot wide sections failed completely as the depressions were filled. This is in agreement with the calculations as shown on Figures 21 through 23 on all sections except Section 12. The calculations on Section 12 indicate that the ends should have pulled out before the reinforcing material failed although the calculated factor of safety with respect to tension failure was only 1.3, well within the possible error in the measured material properties.

The fate of the reinforcing members in the remaining test sections during the filling process is not as clear cut. The geotextiles

remained intact through the test. A void remained under Sections 1 and 5 but the reinforcing members in all of the other sections either stretched or pulled out of the embankment enough so that some of the load may have been supported on the bottom of the trench. There was little or no void under the reinforcing members, however the material under the reinforcing members was loose and had obviously not been compacted by a heavy load.

The anticipated load on the reinforcing materials was calculated for the conditions following filling the depressions. Since no measurements were made, it was necessary to make some assumptions. It was assumed that there would be additional displacement of the reinforcing material as the depression was filled and that the additional displacement would be in proportion to the ratio of the measured displacement prior to filling divided by the original fill height. In addition, it was assumed that the average additional stress on the reinforcing material would be eight tenths of the total displacement of the reinforcing material times the unit weight of the fill. The performance of the test sections is in good agreement with the calculated results shown on Figures 21 and 22. In each case the weight of the fill was enough to cause the geotextile to displace to the bottom of the trench.

The two remaining Sections (1 and 5) withstood the filling and compaction operations but failed during trafficking. No data was gathered regarding the maximum displacement of the geogrid during filling and preparation for trafficking but, from the depth of the hole after the truck tire fell through, it appears that there was little additional deformation. This is in agreement with the calculations as shown on Figures 21 and 22. The reinforcing materials should not have displaced significantly more upon filling the depressions and they should have failed at a load slightly less than the load anticipated with the loaded truck.

5. DESIGN METHODOLOGY

The theoretical model indicates that it is possible and practical to use geotextiles and/or related materials under roads constructed across polygonal ground in discontinuous permafrost zones to limit the catastrophic settlement that occurs when the ice melts. The use of these materials should decrease the abruptness of settlement of roads constructed over ice-rich permafrost and massive ice formations. However, the results are not expected to be as dramatic, and these applications are not considered directly herein. Although the field test did not prove that the concept would work, it did give some validity to the theoretical approach. It is not recommended that the design guidelines given below be used in design until there is experimental verification that it will work.

5.1 DESIGN GEOMETRY

Ice wedges vary greatly in size, shape, orientation and depth below the ground surface. It must be assumed that they will have a random orientation in any situation, so a geotechnical investigation may not yield a detailed description of the subsurface profile. It may be necessary to pick some design void width and spacing, use an appropriate factor of safety, and accept some finite probability that there will be some failures. A design width of eight feet and a factor of safety of three appears reasonable unless geotechnical investigation reveals some reason to use something else.

5.2 MATERIAL PROPERTIES FOR DESIGN

5.2.1 SOILS

Ice wedges generally grow in soils that are primarily silts. In general, the surrounding soil has a high moisture content and may contain ice lenses and massive ice features. Thawing of the surrounding ground may

cause uneven settlement with large deflection basins that the reinforcing material will do little to even out.

The thawed soil will probably be soft, and the edges of the voids may tend to collapse. Edge collapse would create a wider void than would otherwise be expected, however, the wide void would require more side support causing more collapse. It seems likely that the thawed soil will redistribute itself until equilibrium is reached between the collapsing soil and the reinforcing material. The road surface would probably settle more if the edges collapse than if the sides do not collapse, but less than if the reinforcing material had not been there at all, assuming the reinforcing material does not fail in tension. If the reinforcing material fails in tension, the road performance should be similar to that experienced if the reinforcing had not been there.

There will probably be a thick organic mat over the area. It appears prudent to leave the organic mat in place in most areas where voids are expected to occur. The beneficial effects (including construction expedience, insulation, separation, reinforcement and load distribution) should outweigh the detrimental effects of the low modulus material and the possibility of long-term settlement.

5.2.2 GEOTEXTILES AND/OR RELATED MATERIALS

Some of the ice wedges will run at odd angles to the axis of the road, and others will be near the toe of the embankment and perhaps parallel to the edge of the road. It will, therefore, be desirable to use a reinforcing material that is not highly directional in its tension-strain characteristics. In addition, the material should be relatively insensitive to creep over a period of several years and insensitive to cold down to about -10°F.

The wide width tensile test being developed by the American Society of Testing Materials (ASTM) appears to be the best method for determining the short-term tension-strain characteristic of a geotextile in the high

modulus directions. Similar tests can be used on geogrids and other such materials, but it is not as important to keep the wide width tensile test dimensions. It is necessary to test the joints in geogrids since they are usually the weak spots in the material and change the tension-strain characteristics of the material. The joints may also behave differently with time and the environment than the webbing material. Currently there is no generally accepted test method for considering long-term effects, the effects of soil impregnation and confinement, or the effects of temperature on the mechanical properties of either geotextiles or related materials.

Creep and temperature are known to effect some materials more than others. It seems prudent to avoid those materials that are sensitive to creep and temperature unless detailed testing is done and it is established that the materials will satisfy the design criteria. The wide width type of tensile test can be used to study creep and temperature effects, but the details of the loading and environment must approach those in the field, an not those in the test procedure as proposed at this time.

There is some unpublished preliminary research indicating that confinement and soil impregnation greatly increase the tension-strain properties of some geotextiles and related products, particularly on the bias. ASTM is currently considering tests that consider these effects. But until such time as additional information becomes available, it seems reasonable and conservative to use the wide width tensile test on the warp, woof and bias.

5.2.3 INTERACTION BETWEEN SOILS AND GEOTEXTILES OR RELATED MATERIALS

The interaction between soils and foreign materials has been the subject of many studies. There is no widely accepted test method for determining the stress displacement characteristics of the interface between geotextiles and related material and soils. The studies that have been

made on the contact between various mineral soils and various reinforcing materials (using a variety of equipment and methods) indicate the following trends.

- If the holes in the reinforcing material are about the same size or larger than the soil grains, then the failure will be through the soil, and the shear strength will be that of the soil or slightly higher.

- If the holes in the reinforcing material are significantly smaller than the soil grains, then the failure may be at the interface, and the shear strength may be significantly lower than the strength of the soil alone. The controlling factors are the shape of the reinforcing material, the roughness of the surface, the type of material and the soil properties. Strengths as low as two thirds of the soil strength are common.

- If the soil under the reinforcement is soft, the reinforcing material is drapable, and the soil over the reinforcement is large grained, the plane of the reinforcing material may be deformed to the point where the failure will be forced through the soil. The shear strength will be that of the soil.

- A movement of the reinforcing material of 0.1 to 0.2 inches appears to be enough to develop nearly the full shear strength between most materials.

To our knowledge, there is no information in the literature on the shearing behavior between geotextiles or related materials and organic mats. Until such information is obtained, it appears reasonable to consider the relationship frictional with a coefficient of friction of about 0.035.

It is not clear whether the shear stress acts on the top and bottom of the reinforcing materials or just on the bottom. If the material on top of the reinforcing material moves with the reinforcing material as it moves, there will not be relative motion between the two. Therefore, no shear stress will develop. This will happen unless there is resistance built up in the void area to inhibit the motion of the fill over the reinforcing material. Resistance will be built up if there is a relatively small deformation into the void relative to the amount of movement of the reinforcing material outside the void. This effect has not been quantified. Until it has, it appears reasonable to take the conservative approach in design and assume that there is shear stress only on the bottom of the reinforcing material.

5.3 DESIGN TECHNIQUE

The field data indicate that the theory can be used for design. The major uncertainties lie in developing the design criteria from field exploration data. At this stage in the development of the design techniques, it seems reasonable to gather as much field information as practical, to develop the design criteria from this information, to use the available theory to choose appropriate reinforcing materials, and then to monitor the performance following construction. These steps are addressed separately.

5.3.1 FIELD EXPLORATION

The field exploration should establish the extent and character of the ice formations, the magnitude of anticipated thaw settlement not associated with ice formations of limited extent, the competency of the thawed permafrost and the character of the active zone material.

Aerial photographs should be analyzed to determine the extent of ice wedge formation and the extent of other thermokarst features. A surface reconnaissance should be performed looking in detail for surface expressions of the subsurface conditions. It may be appropriate to do

some geophysical testing in search of the size and shape of massive ice formations.

Borings should be made in areas where massive ice is expected and in areas between the massive ice formations where massive ice is not expected. Undisturbed samples should be taken of the material without massive ice and of any mineral soil in the active layer for laboratory testing. Laboratory testing on the frozen soil should consist of at least water content tests, in situ density test, thaw strain tests and thawed strength tests. Laboratory tests on the active layer should consist of at least water content tests and a measure of the thawed strength which is appropriate for the material encountered.

5.3.2 DESIGN CRITERIA

Based on the field and laboratory tests and on experience, the character of the subsurface materials should be established. The required parameters are:

- Extent of thaw settlement excluding that caused by ice masses of limited lateral extent.
- Width of voids to be spanned.
- Minimum distance between anticipated voids.
- Stability of the edges of the void.

The design methodology presented herein is most applicable when there are widely spaced ice masses of limited lateral extent, a minimum amount of other thaw settlement, and a relatively firm material remaining after the permafrost thaws. If these conditions do not exist, it does not mean that the use of geotextiles will not improve the performance of the road. But it does mean that the results will not be as dramatic, and the design procedure presented herein should not be used without modification.

The estimation of the void width must take into account the distribution of the void as it passes through the active zone. There is virtually no factual information to guide the engineer in making this projection. Thermokarsts are typically smaller at the surface when they first form. This may reverse itself as weathering and other forces cave in the sides. The pressure from the reinforcing material on the edges of the void will probably preclude any overhang but may or may not cause additional caving of the sides.

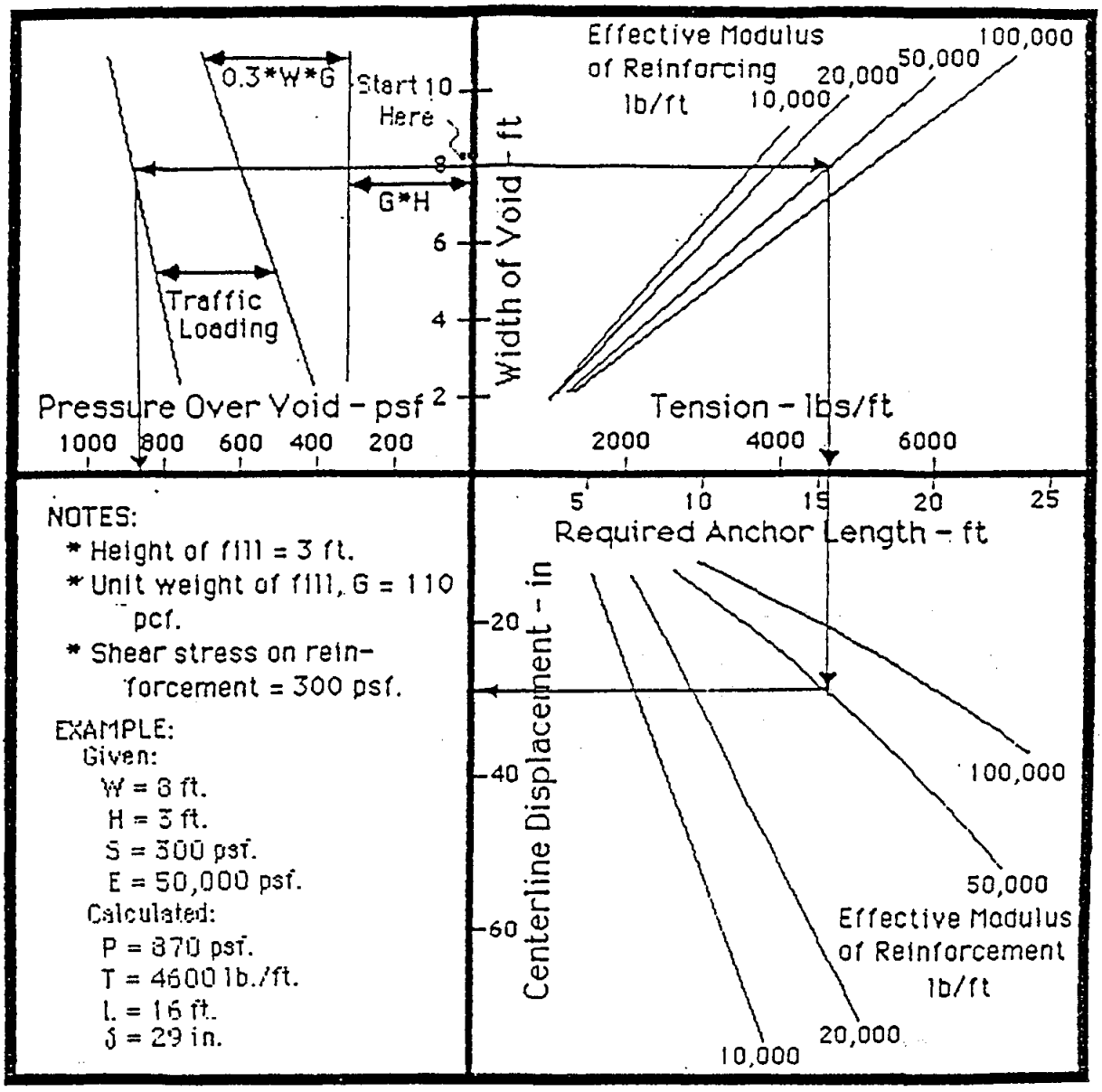
Any real situation will be less than perfect, and it will be necessary to make some reasonable assumptions. These assumptions will undoubtedly be unconservative at some locations, so some reasonable probability of failure must be accepted. Assuming that conditions permit using this technique, it will probably be adequate to use a design width of 8 feet and a design spacing of 20 feet in most areas.

5.3.3 REINFORCING MATERIAL AND EMBANKMENT CROSS SECTION

The embankment must be thick enough to distribute the traffic loading sufficiently on the subgrade material, and to provide any insulation or frost heave protection that is included in the design. The reinforcing material must support the weight of the embankment plus the traffic loading. The traffic loading decreases but the embankment load increases as the embankment height increases. Therefore, there is some optimum embankment thickness that is ultimately determined by economics.

The design procedure contained herein can be used directly to determine the optimum conditions. Parametric runs of the computer program have been included in Figure 24 to give the designer a place to start in the analysis. The following assumptions were made in the parametric runs:

- Void width = 8 feet.
- Embankment fill height = 3 feet at 110 pcf.
- Shear stress on reinforcing material = 300 psf.
- Effective traffic load on the reinforcing material = 300 psf.



RESEARCH SECTION
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SAMPLE CALCULATIONS

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Fig. 24

- Ends of reinforcement are buried enough so that they do not slip.
- The reinforcing material is linearly elastic.

If the reinforcing material used has holes that are large enough to let the subgrade material mix with the fill material or to let the fill material fall through, it will be necessary to add a separator over the reinforcing material. Any lightweight geotextile with the appropriate opening size would work.

6. CONSTRUCTION TECHNIQUES

6.1 SITE PREPARATION

The use of reinforcing material considered herein requires that the material be smoothed when the fill is placed and that it be anchored at the ends. Anchoring may be done by artificial means such as burying the end in a trench or by burying enough length of material under the embankment. These conditions require that the grade be relatively smooth when the reinforcing material is placed. This does not mean that there cannot be holes in the subgrade or that the subgrade must be graded. It merely means that it must be possible to lay the reinforcing materials relatively flat in relatively good contact with the subgrade. If the tundra is left in place, it must be walked down with construction equipment prior to construction. All stumps and other protrusions must be cut off as close as practical to the ground surface. If practical, it would be preferable to remove them.

In most applications, a fill depth on the order of 2.5 to 4.0 feet will probably be the optimum, considering the design procedure presented herein and economics. There may be other considerations that require a higher fill, for instance, to avoid flooding or to cross a low spot in the grade. The reinforcing material will be most beneficial if it is placed as low as practical in the profile, but the strength and stiffness criteria increase as the fill height increases above about three feet. The problem is complicated by the shape of the void width as it passes up through the fill material if the reinforcing material is placed higher in the road profile. There is virtually no solid information to guide the engineer. It appears reasonable to assume that the void width remains constant through the fill. Using this assumption, the design procedure can be used as though the reinforcing material were directly over the void.

It is difficult to get sufficient anchorage along the outside edge of the road by merely burying the end of the reinforcing material under the

road fill. Either the embankment must be extended far enough to give anchorage to the main part of the roadway, or some form of anchorage must be used. There is some evidence that an anchor trench might work. The present information indicates that, if an anchor trench is to be used, it should extend at least 18 inches into mineral soil and be at least 18 inches wide. It should be at a point in the embankment where there are at least 18 inches of fill on top of the outside of the trench.

6.2 REINFORCING MATERIAL INSTALLATION

The reinforcing material should be laid flat on the surface. All seams must be tied together in a manner that will take the full design tension. Overlapping seams will not carry the required tension. When practical, the stiff and strong direction should be orientated perpendicular to the direction of the expected void. Reinforcing materials with large opening sizes should be covered with a separation material if necessary. Overlapping the separation material a minimum of six inches at the seams will be acceptable if it is not considered to be part of the reinforcing material.

6.3 EMBANKMENT CONSTRUCTION

The embankment material must be placed in such a way as to avoid damaging the reinforcing or separating materials. Under most conditions, it would be appropriate to add the first two feet of material in a single lift by end dumping it and spreading it with a small bulldozer from the end that has been previously covered. Following the first lift, the fill should be compacted. All additional material should be placed in no more than nine-inch loose lifts and compacted.

Good compaction would be desirable in most installations, but it may be difficult to attain. It appears most appropriate to use a performance specification such as so many passes with a certain size piece of equipment instead of an acceptance criterion such as a certain minimum

density. This would allow the owner to get the best project possible under the conditions with a minimum of contract disputes which might arise from the possibility that a given density might not be possible to attain because of the soft subgrade.

It is often desirable to do the construction in the winter when the ground is frozen and access is easier. Fill material cannot be compacted with any degree of success when it is frozen, regardless of the amount of compactive effort that is applied or the water content of the fill material (Kinney and Goetz, 1984). Winter construction is a viable option for at least the lower portions of the embankment as long as the reinforcing material can be worked in the cold weather. The fill should be spread and a minimal amount of compactive effort should be applied in the cold weather. After the fill is thawed in the spring, it should be compacted again. It would be desirable to leave at least the final one foot of fill for spring placement to allow better compaction.

6.4 SURFACING

There will be a significant amount of settlement on the surface of the road during thawing regardless of the strength or stiffness of the reinforcing material. Several inches to two or more feet of settlement may occur in a system that is functioning properly. Depending upon the thermal regime, it will probably take one season for most of the thaw settlement to take place. During the period when rapid thawing is taking place, it will be necessary to regrade the road frequently to keep it passable if that is to be attempted at all.

Once the thaw has progressed to the point where voids have formed below the reinforcing material, the rate of settlement should taper off quickly and a high quality gravel surface could be applied. Once the maintenance records indicate that the settlement rate is negligible, consideration could be given to paving the surface.

The test results were unrealistically severe and still indicated that the void areas were marginally stiff enough for paving. It seems appropriate to test the surface with a falling weight deflectometer and run an analyses to determine the expected life of a pavement. The results may indicate that paving is economically viable.

7. CONCLUSIONS

Although not conclusive, the study indicates that the design methodology presented herein is valid and that the application tested is feasible if the true material properties are used. There is some maximum width of void that it is practical to span depending upon the material properties and the anchorage available. Under some conditions geotextiles or related products should be able to span voids of 6 or 10 feet or more. Geotextiles and related products should not be expected to significantly effect road surface settlement due to thawing of large ice masses or ice rich permafrost.

If the void runs parallel to the road near the shoulder, the anchorage on the outside is limited. If sufficient anchorage is not available, the reinforcing material will slide into the void as the ice melts and little, if any, benefit will be gained.

Large displacements should be expected until the reinforcing material has stretched enough to carry the load. No information was obtained regarding additional displacement caused by traffic loading.

The published tension-strain data from a grab tensile test is not necessarily sufficient for reinforcement design. The wide width tensile test should be used and creep must be considered.

The reinforcing material properties of concern are those perpendicular to the void. In an unconfined state woven geotextiles generally have a low tension-strain modulus in any direction except parallel to the weave. This severely limits their usefulness in any direction except parallel to the weave. Confining stress changes the stiffness on the bias but the amount is not known. Non-woven geotextiles tend to be isotropic, but readily available ones do not generally have the strength or stiffness required.

8. RECOMMENDATIONS

1) Another test facility similar to the one described herein should be constructed. The facility could be constructed in the summer by excavating a trench, stretching the reinforcing material across the trench, loading the ends of the reinforcing material before the center, and doing the trafficking immediately after construction. The entire test could be conducted in two weeks.

2) A test section should be constructed on a section of road that is typical of the construction and terrain where this approach should be beneficial.

3) All reinforcing materials being considered for use should be tested for their dynamic and long term tension-strain properties. The wide width tensile test should be used and creep testing should be carried out for at least 2 weeks, preferably for months, and ideally for years.

4) Testing should be carried out to determine the effects of soil impregnation and soil pressure on the dynamic and long term tension-strain properties of all reinforcing materials being considered for use. These tests should include the properties in all directions, not just parallel to the weave.

5) Additional theoretical work and testing should be done to establish the viability of anchoring the reinforcing material to avoid the necessity for long anchorage lengths under the embankment.

6) Additional theoretical work and testing should be done on the stability of the void walls so that a reasonable estimate can be made regarding their stability.

7) Additional theoretical work, laboratory studies, and field studies should be carried out to determine quantitatively the effect of the

lateral restraint imposed on the fill material by the reinforcing material. Considering the large deformations that occur these effects could be very significant and they have been completely ignored up to this point.

8) Additional field or laboratory studies should be performed to further define the shear stress characteristics between reinforcing materials and the soils in the road profile. Two aspects of particular concern are the maximum shear stress that can be developed when the reinforcing material is placed on organics and relationship between the soil on top of the reinforcing material and the reinforcing material itself.

9. IMPLEMENTATION

The design methodology which was developed and the testing performed in this study went beyond the state-of-the-art at the time. The testing substantiated the published literature in the appropriate areas and tended to support the design methodology developed.

The design methodology presented appears to be reasonable and the conclusions and recommendations presented appear to be valid. However, the test results were not conclusive and caution must be used in progressing with designs based on the information presented herein until further experimental work has been done which demonstrates that the concepts can be used reliably in design.

In light of the inconclusive nature of the test results it does not appear prudent to detail design guidelines at this time. All of the information necessary to prepare design guidelines is presented in the body of the report and could easily be extracted once it is established that the concept is viable and that it should be used in production.

10. ACKNOWLEDGEMENTS

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APPENDIX A

EXAMPLE CALCULATION

Appendix A
Example Calculation

Design Geometry:

Ice wedge width = 5 ft.
Dual tandem loading 4500 lb. / wheel
Embankment height = 30 in.

Reinforcing Material:

Max tension = 400 #/in.
Failure strain = 10%
Shear stress = 200 psf.

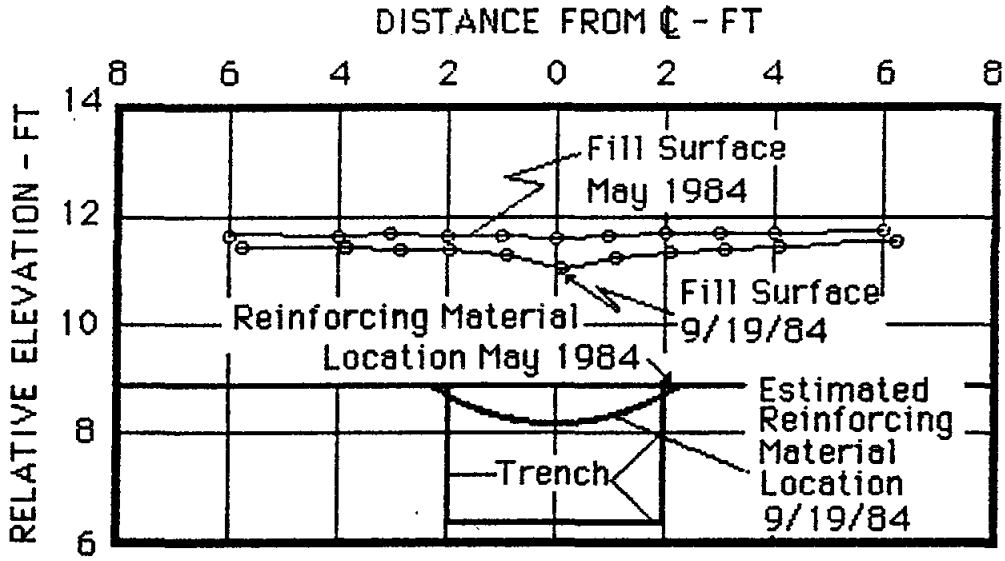
		Initial Try	Final Try
Step 1			
W (Given) ft.		5	
P (Fig. 6) psf		780	
Step 2			
E lb./ft.	$\frac{400*12}{0.10}$ =	48,000	
Step 3			
R (Estimate) ft.	3/4*5 =	3.75	3.07
Step 4			
T(Eq 1) 16ft.	780*3.75 =	2925	2395
Step 5			
d_2(Eq 4)ft.	$\frac{2925^2}{200*48,000*2}$ =	0.45	0.299
d_1 (Eq 5)ft.	$\frac{5*2925}{48000}$ =	0.30	0.249
Step 6			
θ (Eq 6) Deg.	$\sin^{-1}(\frac{5}{2*3.75})$ =	41.8	54.5
Step 7			
L_1(Eq 7) ft.	$\frac{\pi*3.75*41.8}{90}$ =	5.47	5.840
Step 8			
L_2(Eq 8) ft.	5 + 2*0.45 + 0.30 =	6.20	5.847
Step 9			
Iterate if $L_1 \neq L_2$		5.47 ≠ 6.20	5.840 = 5.847
Step 10			
Compare T and T_{max}			2395<<4800
Step 11			
Check E			OK
Step 12			
D (Eq 9) ft.		3.06(1-cos(54.5)) = 1.28	
Step 13			
L (Eq 3) ft.			2395/200 = 11.8

APPENDIX B

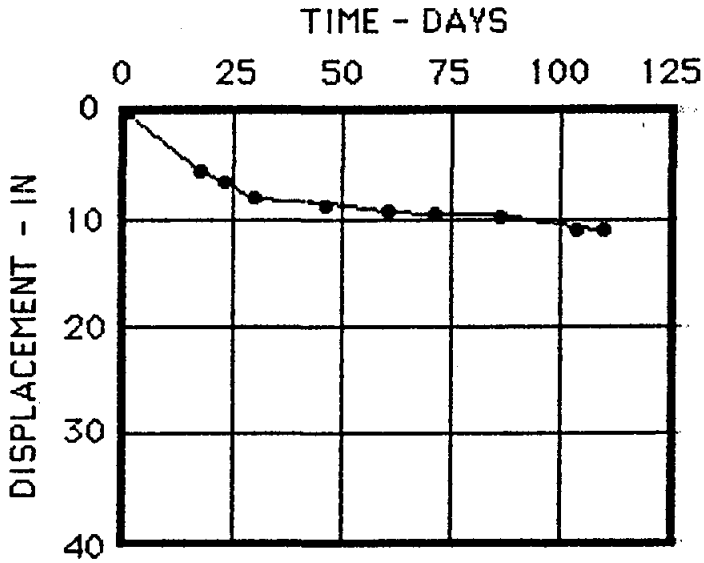
SUMMARY OF

SURFACE SETTLEMENTS

FIGURES B-1 THROUGH B-12



a) SURFACE DISPLACEMENT ACROSS TRENCH



b) CENTERLINE DISPLACEMENT OF REINFORCING MATERIAL

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AND PUBLIC FACILITIES

DISPLACEMENTS
DURING THAWING
TEST SECTION #1

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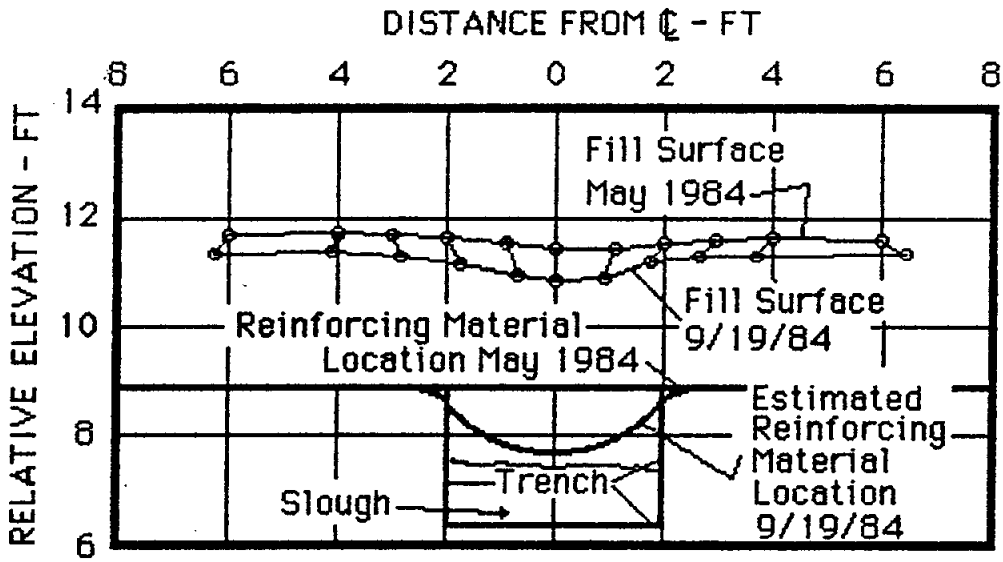
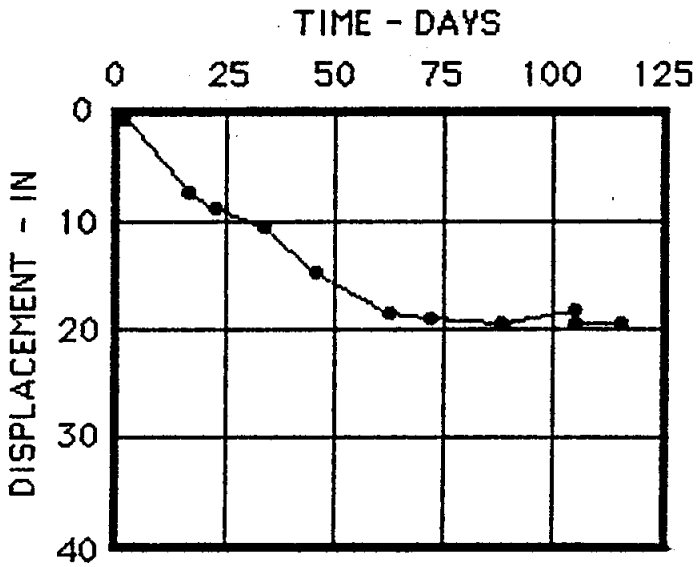
FIG. B-1



a)SURFACE DISPLACEMENT ACROSS TRENCH



b) CENTERLINE DISPLACEMENT OF REINFORCING MATERIAL

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DURING THAWING
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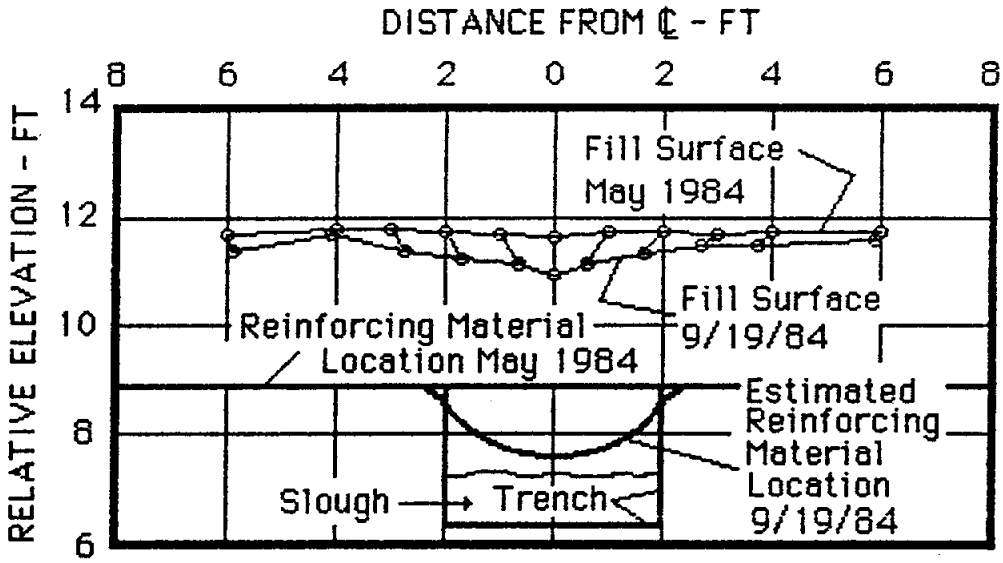
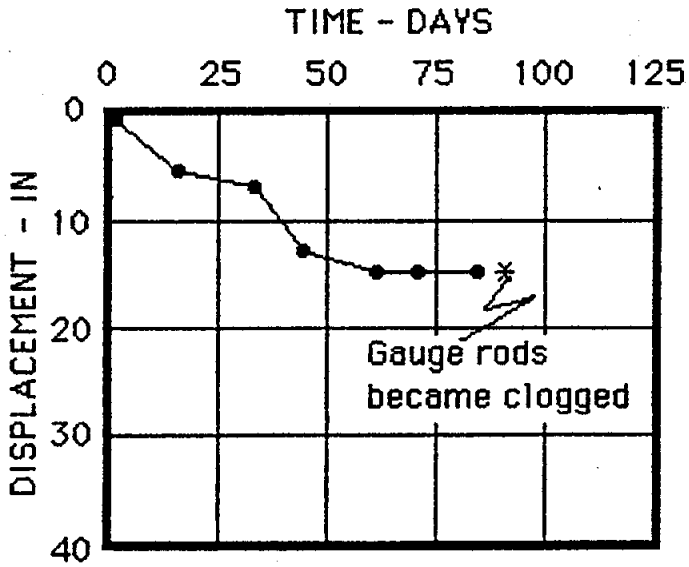
FIG. B-2



a)SURFACE DISPLACEMENT ACROSS TRENCH



b) CENTERLINE DISPLACEMENT OF REINFORCING MATERIAL

RESEARCH SECTION
DEPARTMENT OF TRANSPORTATION AND PUBLIC FACILITIES

DISPLACEMENTS DURING THAWING TEST SECTION #3

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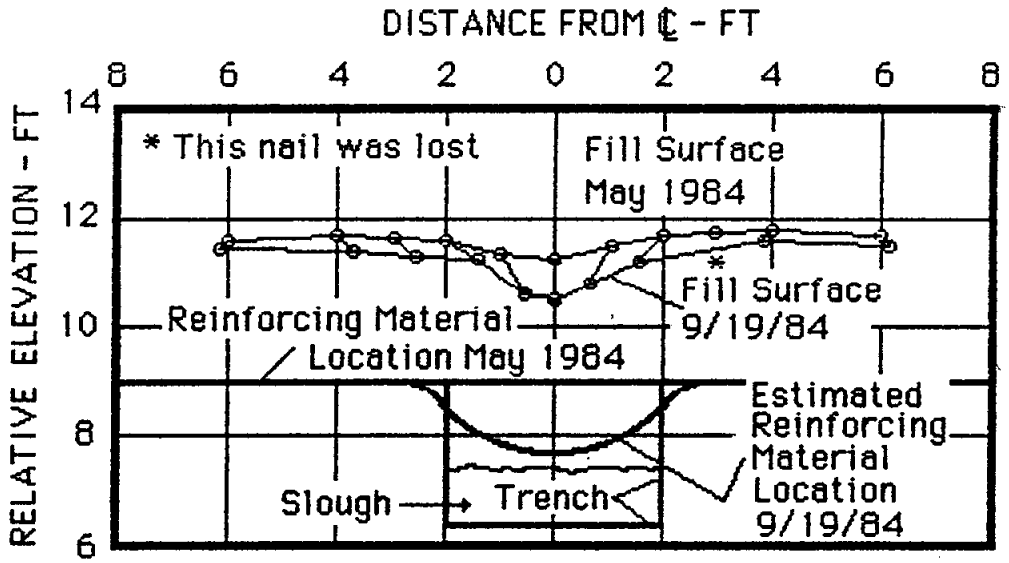
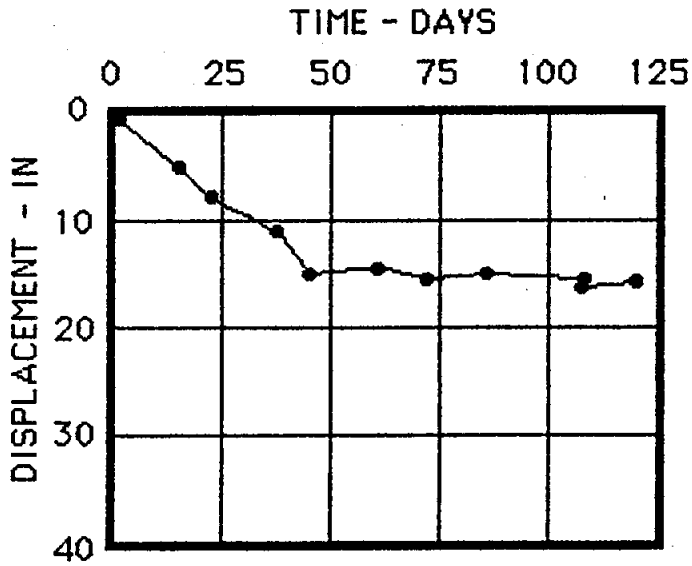
FIG. B-3



a)SURFACE DISPLACEMENT ACROSS TRENCH



b) CENTERLINE DISPLACEMENT OF REINFORCING MATERIAL

RESEARCH SECTION
DEPARTMENT OF TRANSPORTATION AND PUBLIC FACILITIES

DISPLACEMENTS DURING THAWING TEST SECTION #4

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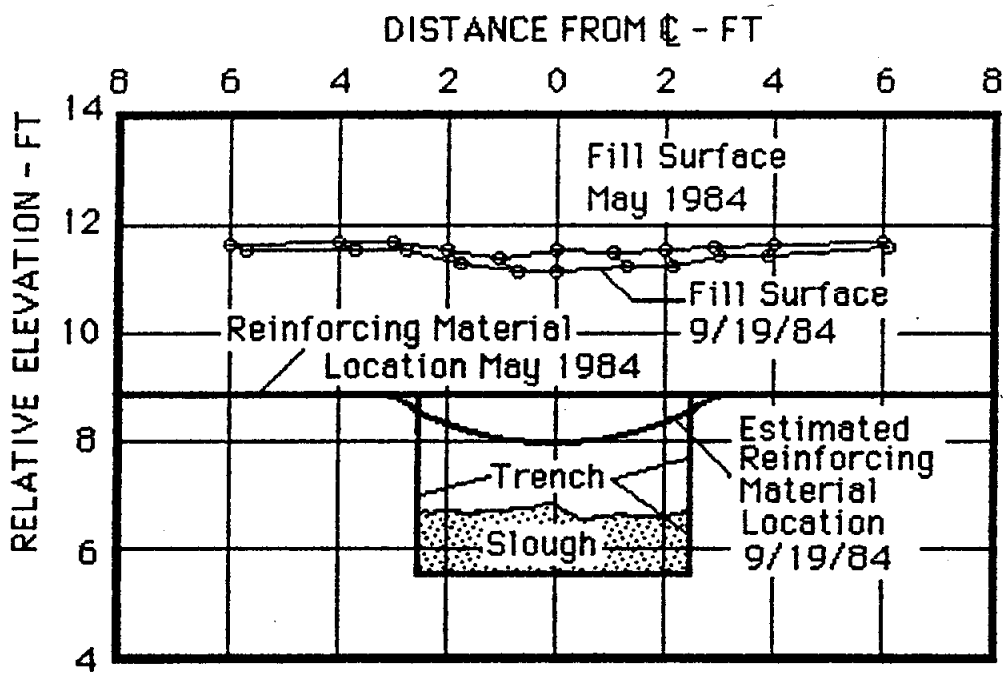
FIG. B-4



a)SURFACE DISPLACEMENT ACROSS TRENCH

NOTE: Gaging rods became clogged during original backfilling

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DISPLACEMENTS
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TEST SECTION #5

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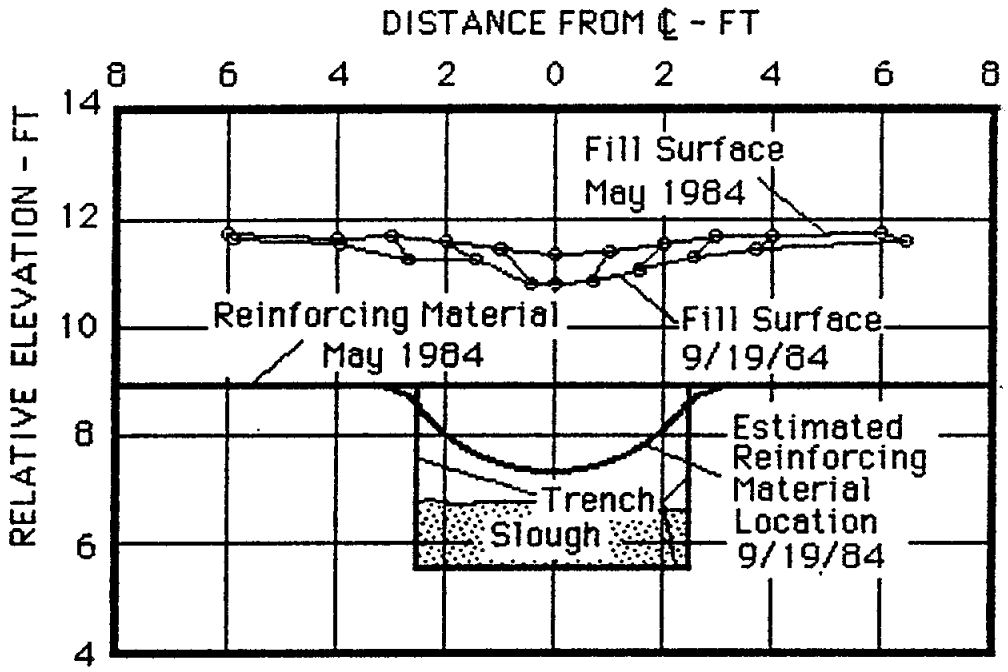
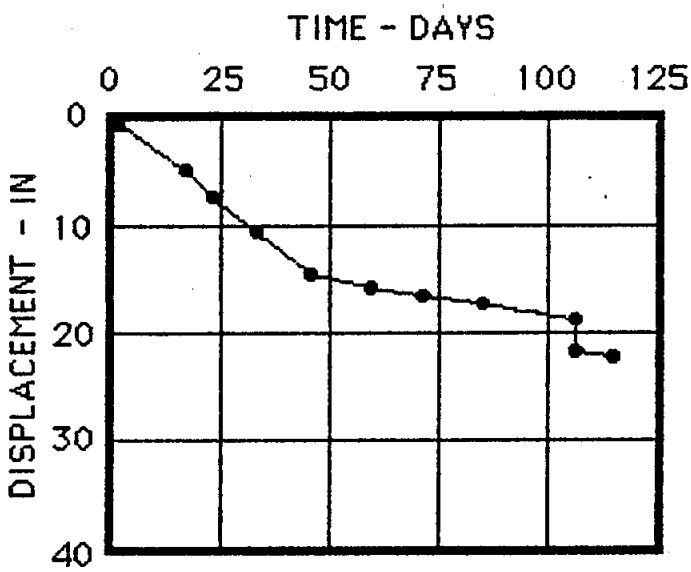
FIG. B-5



a)SURFACE DISPLACEMENT ACROSS TRENCH



b) CENTERLINE DISPLACEMENT OF REINFORCING MATERIAL

RESEARCH SECTION
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DISPLACEMENTS
DURING THAWING
TEST SECTION #6

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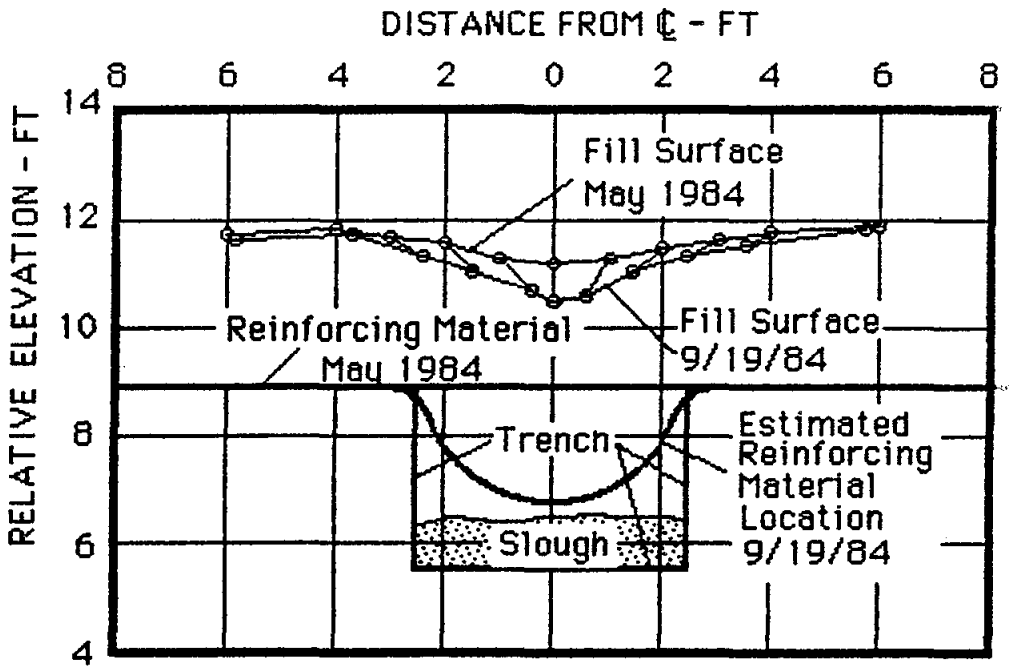
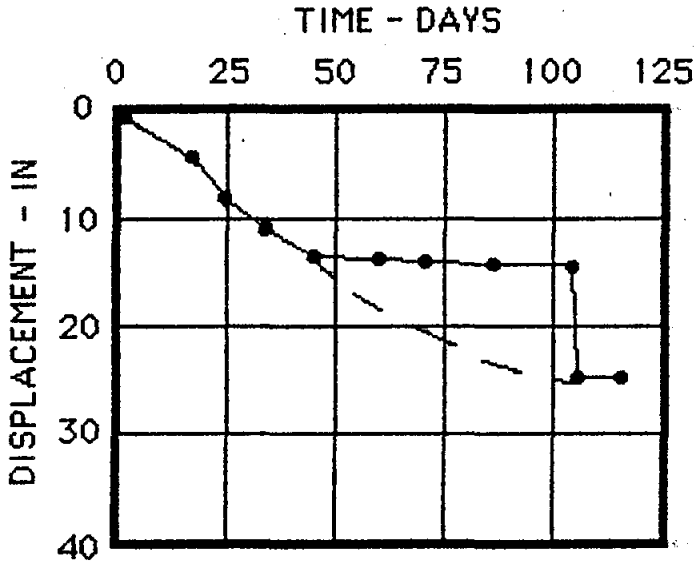
FIG. B-6



a)SURFACE DISPLACEMENT ACROSS TRENCH



b) CENTERLINE DISPLACEMENT OF REINFORCING MATERIAL

RESEARCH SECTION
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AND PUBLIC FACILITIES

DISPLACEMENTS
DURING THAWING
TEST SECTION #7

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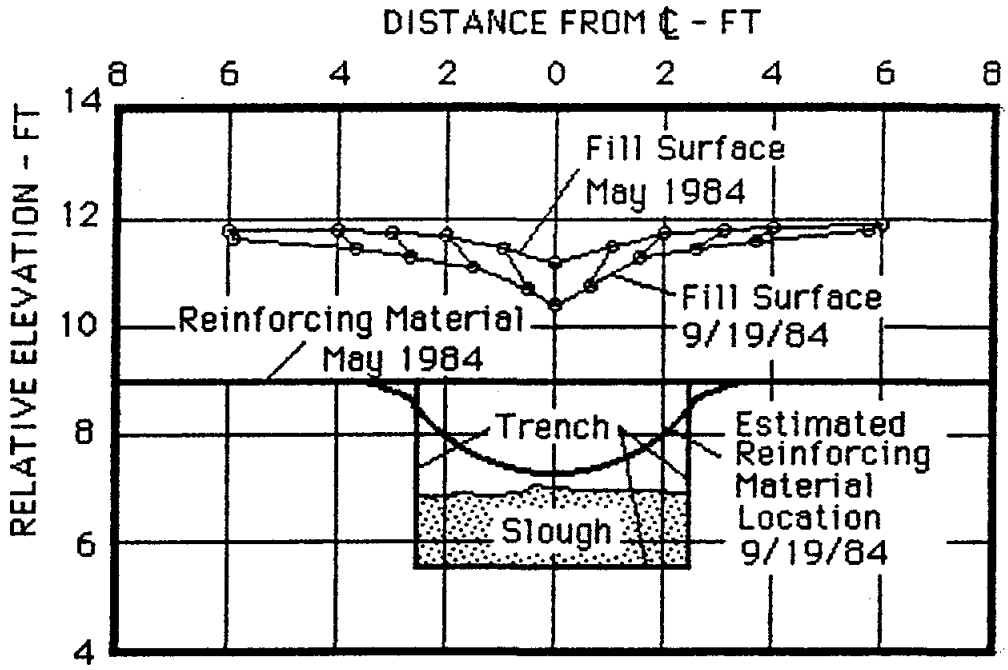
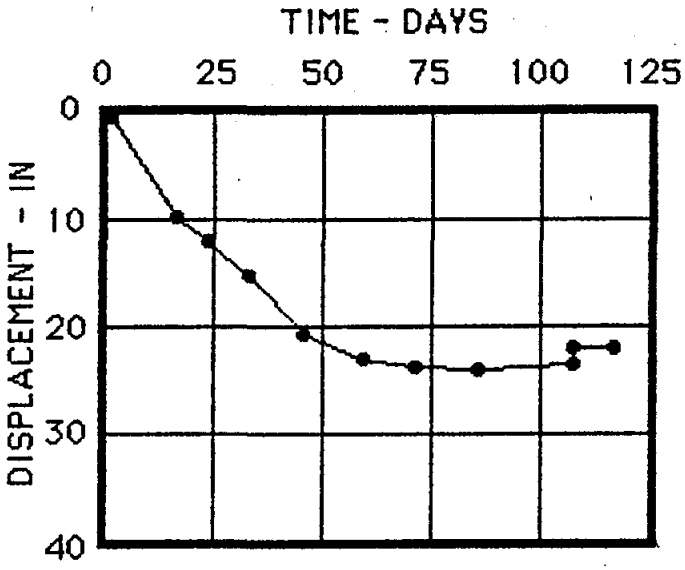
FIG. B-7



a)SURFACE DISPLACEMENT ACROSS TRENCH



b) CENTERLINE DISPLACEMENT OF REINFORCING MATERIAL

RESEARCH SECTION
DEPARTMENT OF TRANSPORTATION
AND PUBLIC FACILITIES

DISPLACEMENTS
DURING THAWING
TEST SECTION #8

K-0617 MAY 1985

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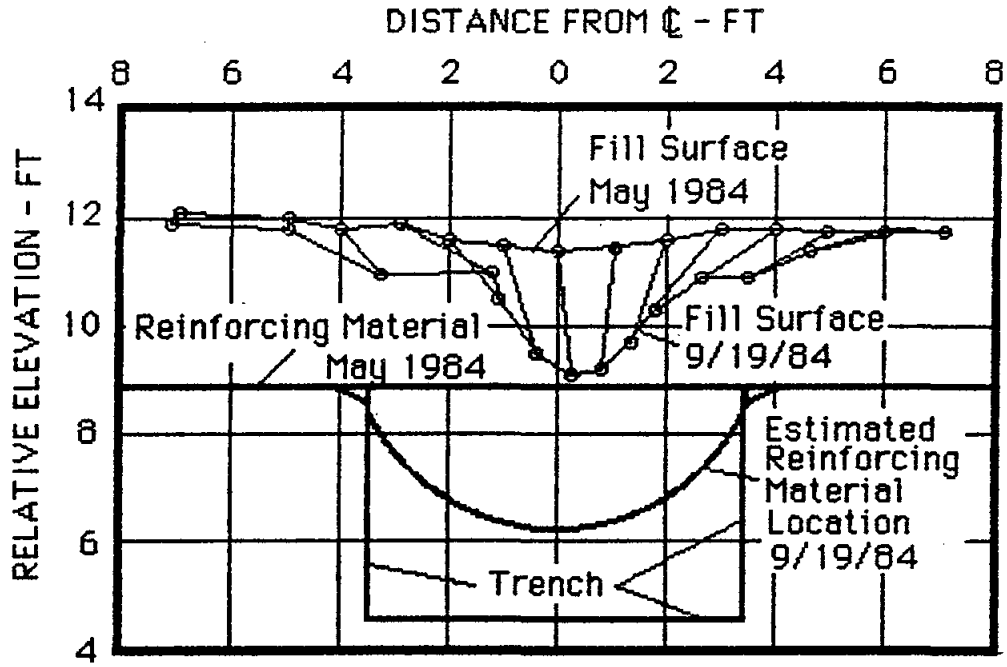
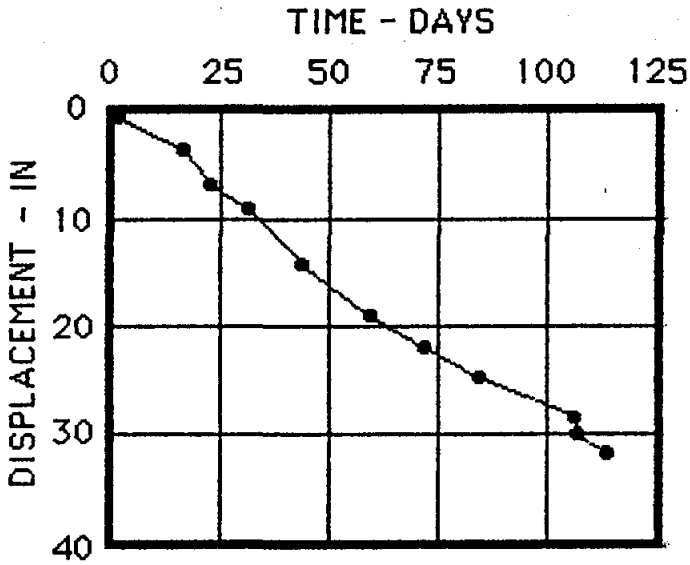
Fig. B-8



a)SURFACE DISPLACEMENT ACROSS TRENCH



b) CENTERLINE DISPLACEMENT OF REINFORCING MATERIAL

RESEARCH SECTION
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AND PUBLIC FACILITIES

DISPLACEMENTS
DURING THAWING
TEST SECTION #9

K-0617 MAY 1985

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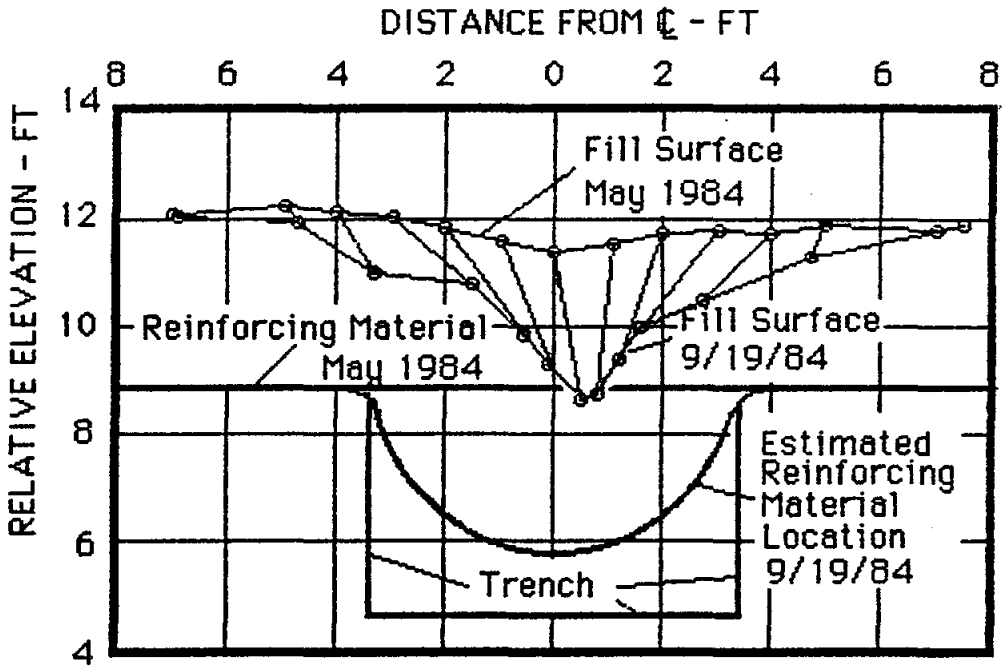
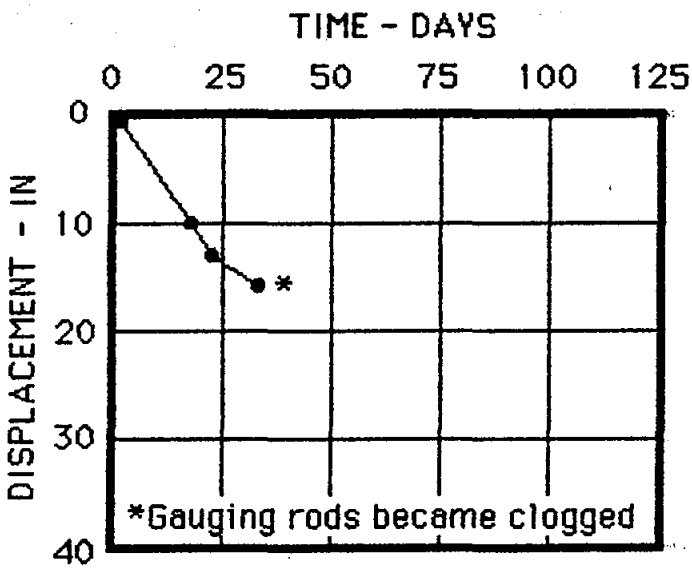
Fig. B-9



a)SURFACE DISPLACEMENT ACROSS TRENCH



b) CENTERLINE DISPLACEMENT OF REINFORCING MATERIAL

RESEARCH SECTION
DEPARTMENT OF TRANSPORTATION
AND PUBLIC FACILITIES

DISPLACEMENTS
DURING THAWING
TEST SECTION #10

K-0617 MAY 1985

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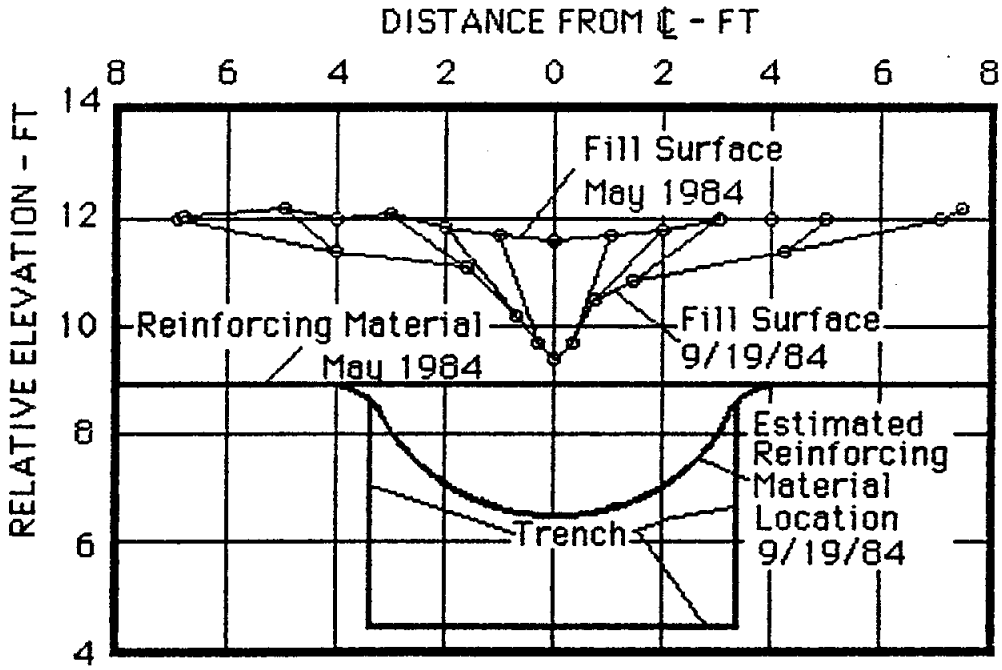
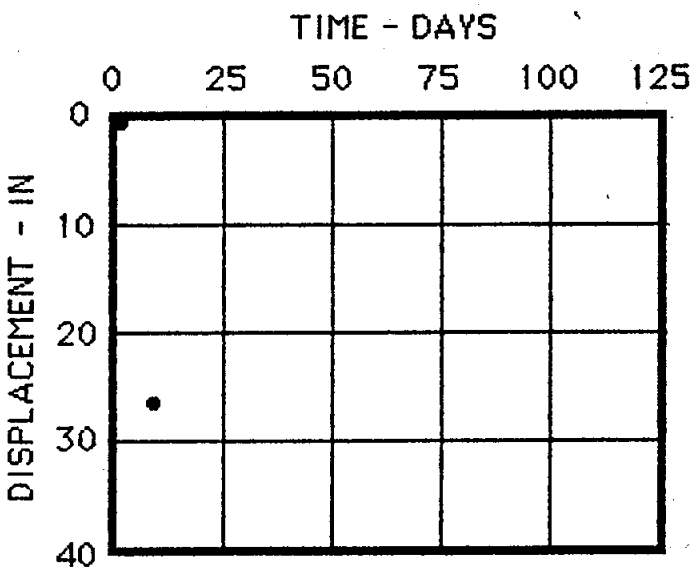
Fig. B-10



a)SURFACE DISPLACEMENT ACROSS TRENCH



b) CENTERLINE DISPLACEMENT OF REINFORCING MATERIAL

RESEARCH SECTION
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AND PUBLIC FACILITIES

DISPLACEMENTS
DURING THAWING
TEST SECTION #11

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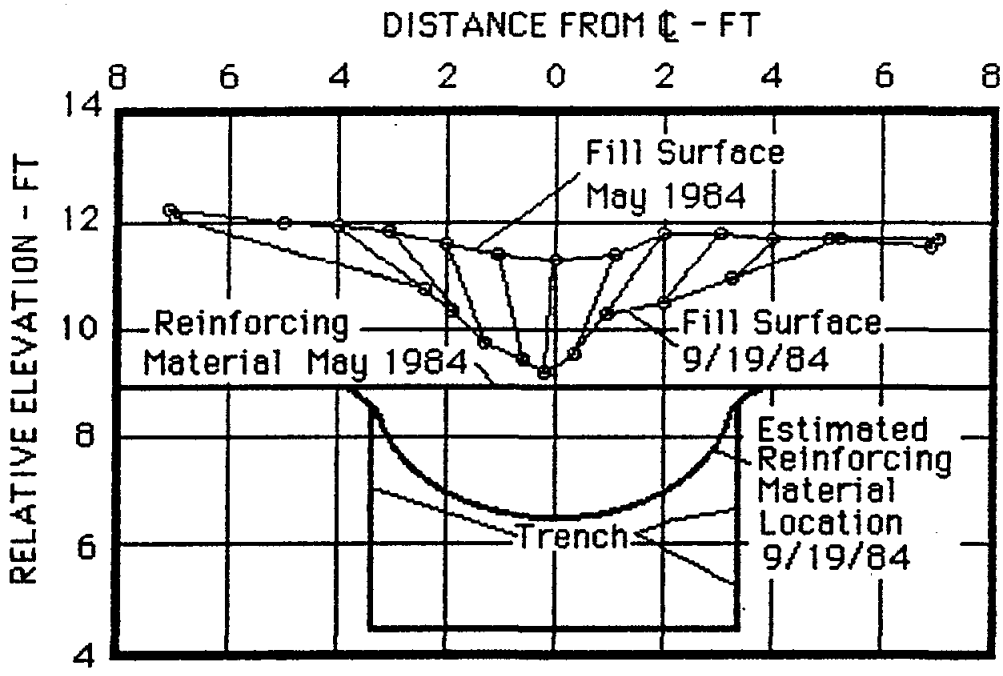
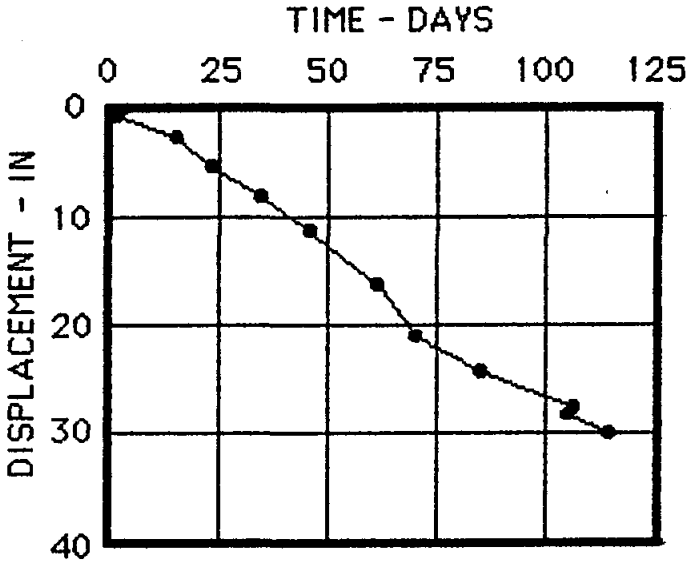
Fig. B-11



a)SURFACE DISPLACEMENT ACROSS TRENCH



b) CENTERLINE DISPLACEMENT OF REINFORCING MATERIAL

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DISPLACEMENTS
DURING THAWING
TEST SECTION #12

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Fig. B-12

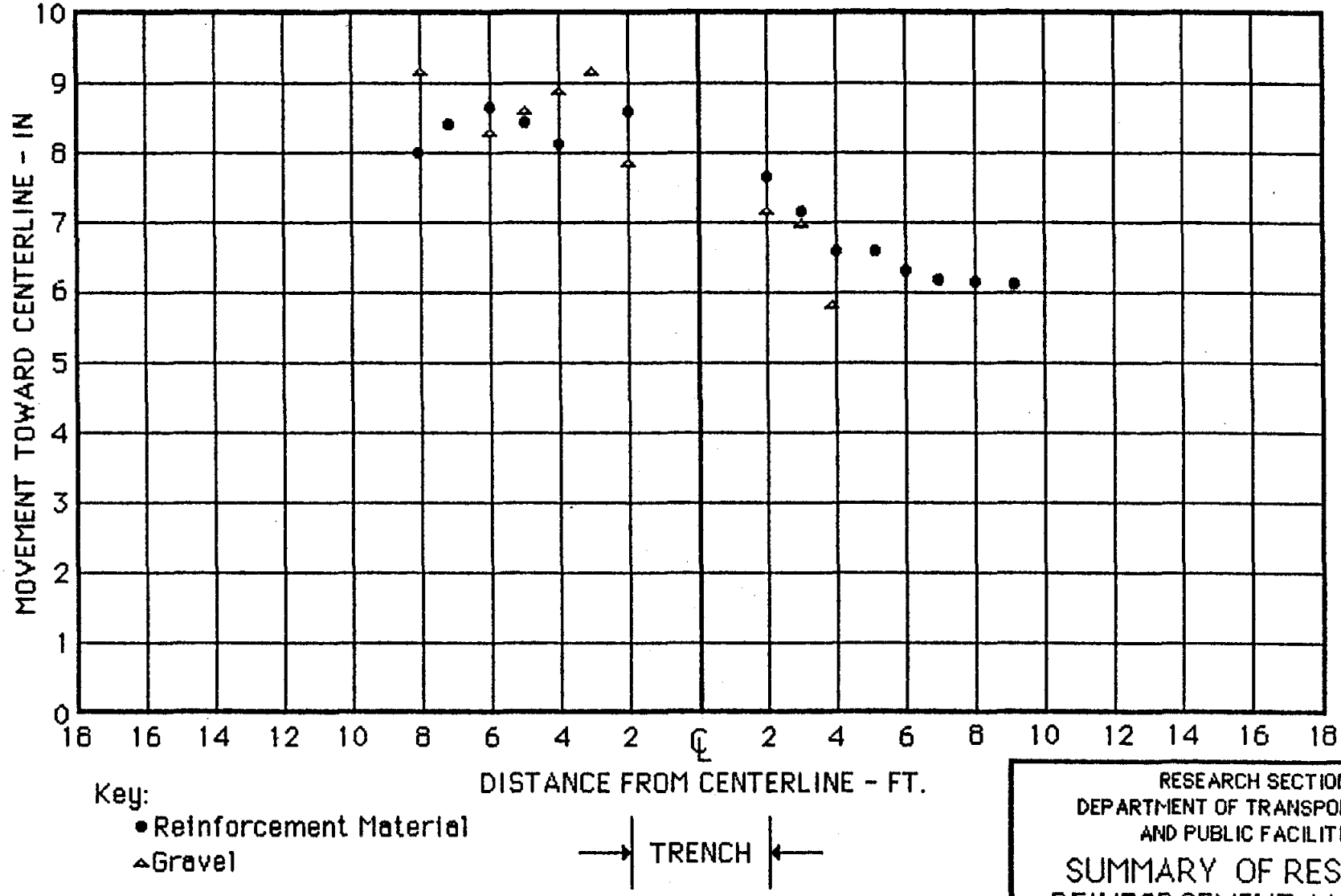
APPENDIX C

SUMMARY OF

RESIDUAL

REINFORCING MATERIAL LOCATION

FIGURES C-1 THROUGH C-12



RESEARCH SECTION
DEPARTMENT OF TRANSPORTATION
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SUMMARY OF RESIDUAL REINFORCEMENT MATERIAL AND GRAVEL LOCATIONS SECTION 1

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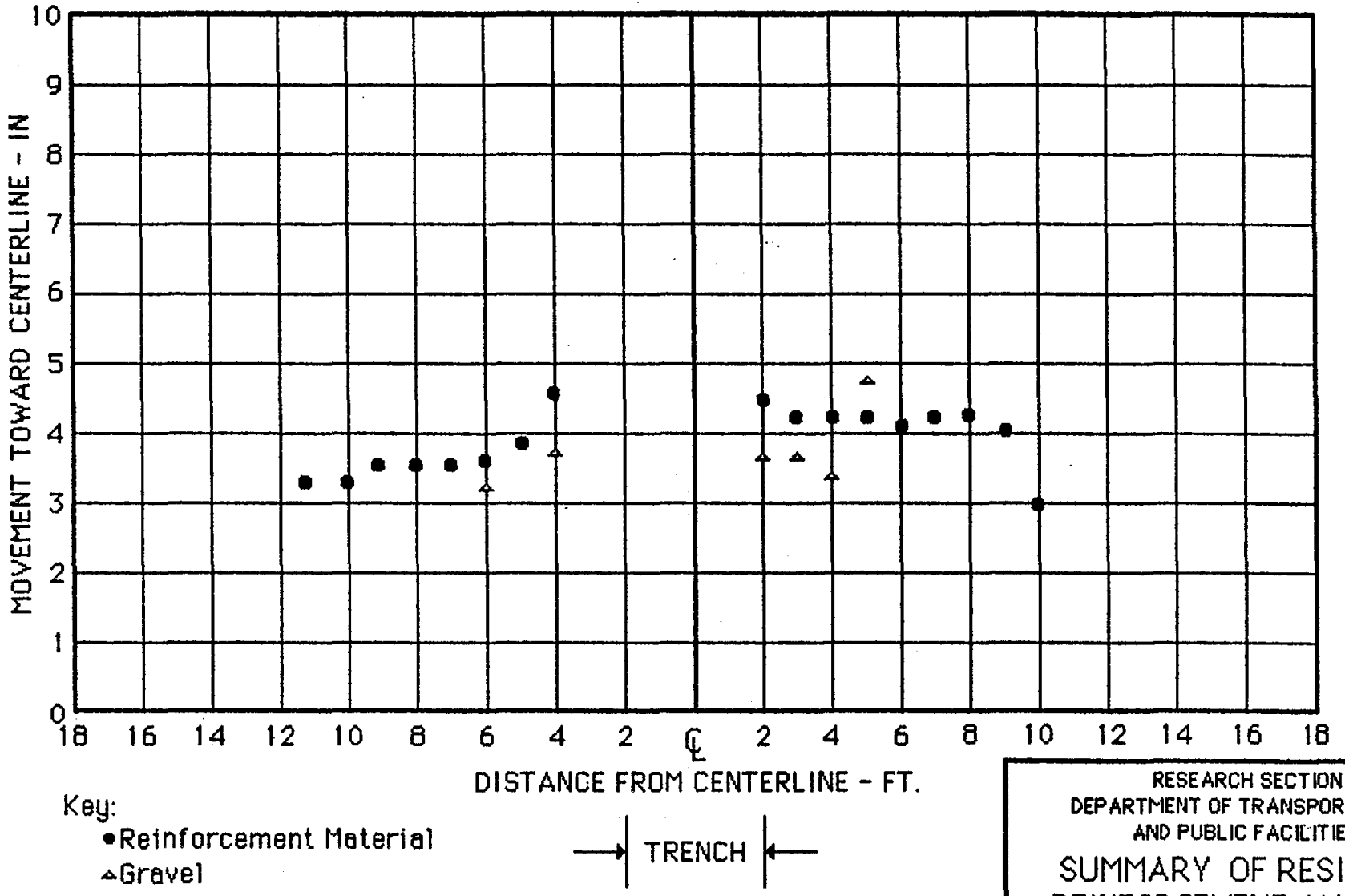
Fig.C-1



RESEARCH SECTION
DEPARTMENT OF TRANSPORTATION
AND PUBLIC FACILITIES

SUMMARY OF RESIDUAL
REINFORCEMENT MATERIAL
AND GRAVEL LOCATIONS
SECTION 2

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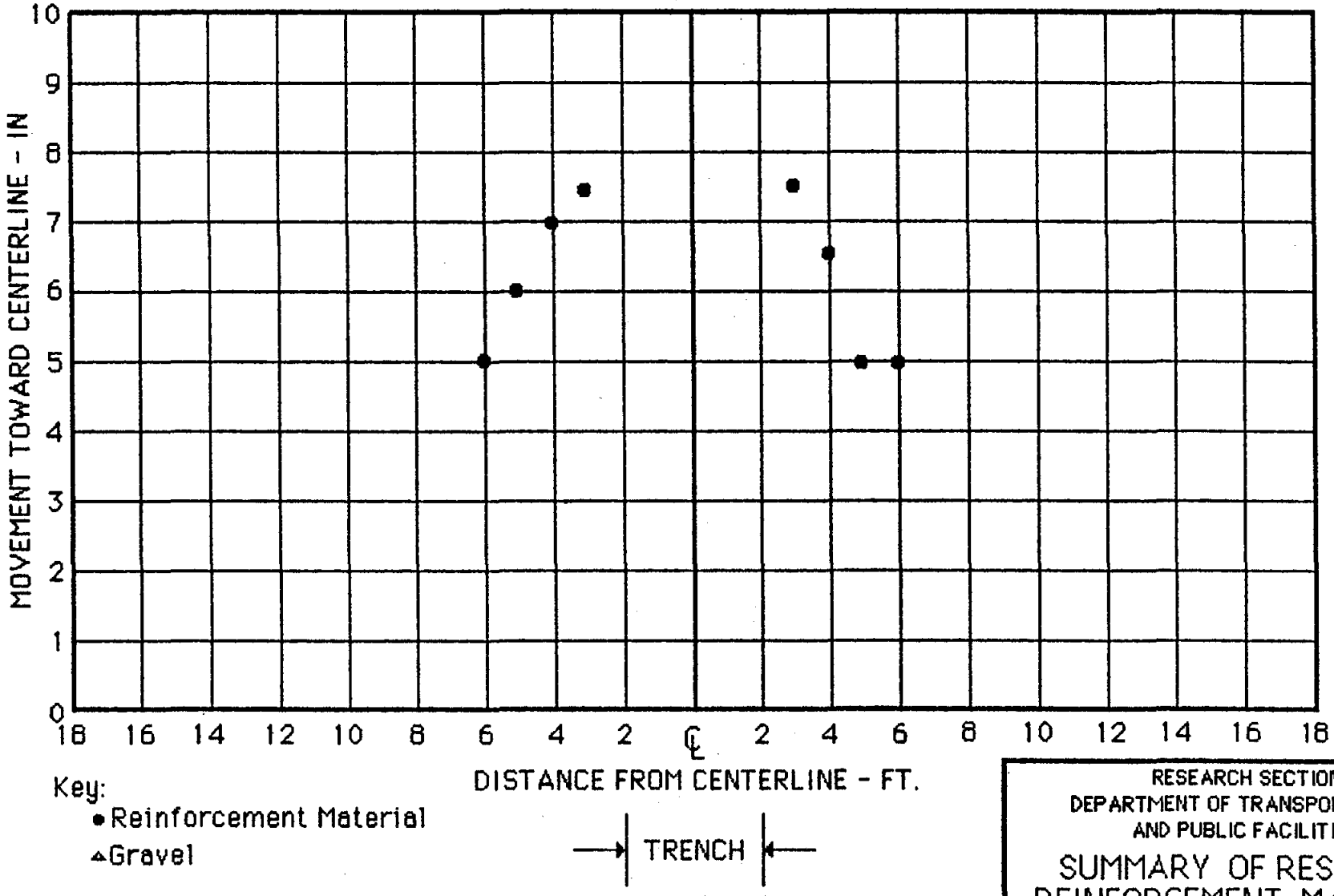
Fig. C-2



Key:
- ● Reinforcement Material
- ▲ Gravel

RESEARCH SECTION
DEPARTMENT OF TRANSPORTATION
AND PUBLIC FACILITIES

SUMMARY OF RESIDUAL REINFORCEMENT MATERIAL AND GRAVEL LOCATIONS SECTION 3

K-0617 MAY 1985

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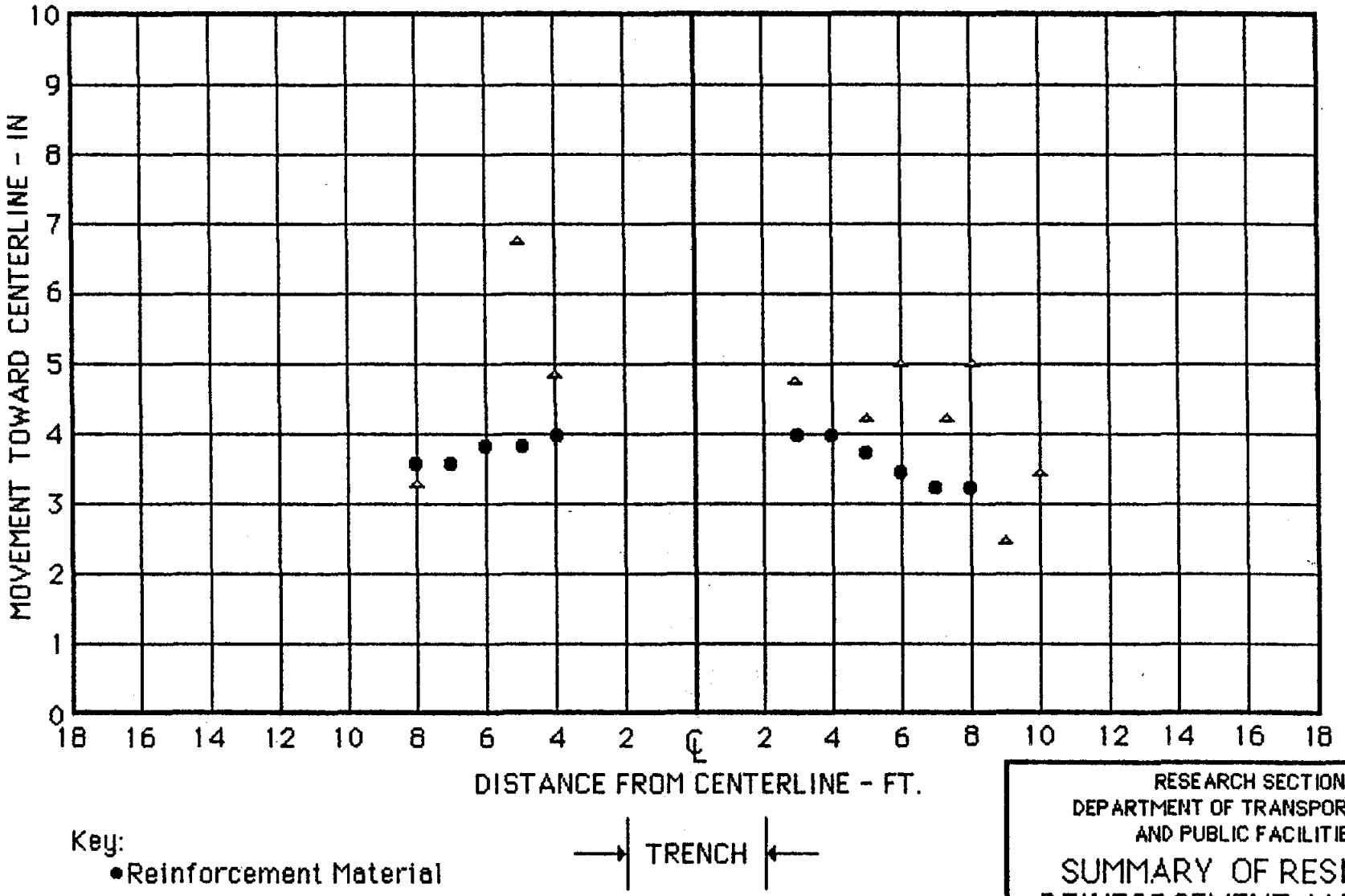
Fig.C-3



Key:
- ● Reinforcement Material
- △ Gravel

RESEARCH SECTION
DEPARTMENT OF TRANSPORTATION
AND PUBLIC FACILITIES

SUMMARY OF RESIDUAL REINFORCEMENT MATERIAL AND GRAVEL LOCATIONS SECTION 4

K-0617 MAY 1985

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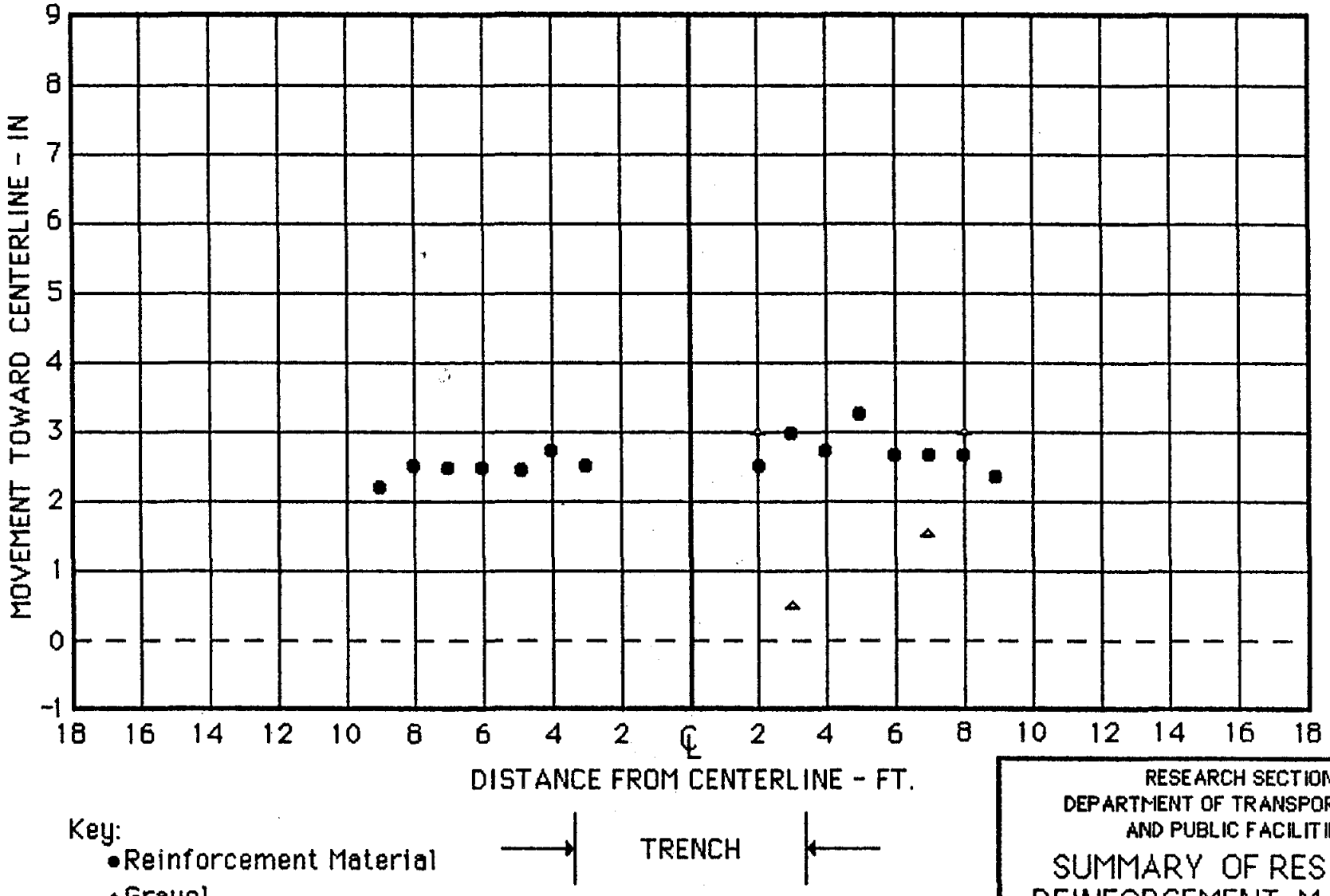
Fig. C-4



RESEARCH SECTION
DEPARTMENT OF TRANSPORTATION
AND PUBLIC FACILITIES

SUMMARY OF RESIDUAL REINFORCEMENT MATERIAL AND GRAVEL LOCATIONS SECTION 5

K-0617 MAY 1985

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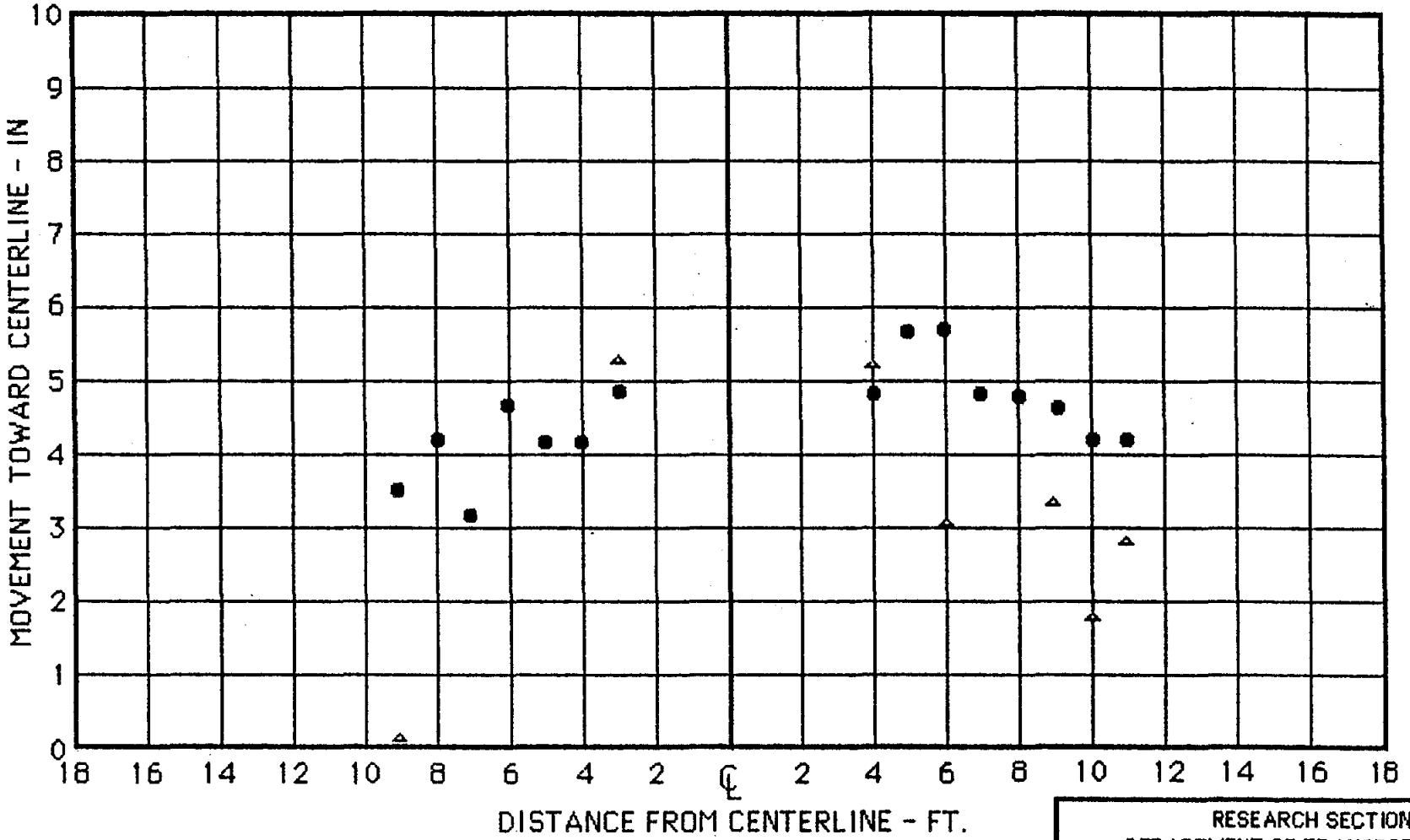
Fig. C-5



Key:
- ●Reinforcing Material
- △Gravel

→ TRENCH ←

RESEARCH SECTION
DEPARTMENT OF TRANSPORTATION
AND PUBLIC FACILITIES

SUMMARY OF RESIDUAL REINFORCEMENT MATERIAL AND GRAVEL LOCATIONS SECTION 6

K-0617 MAY 1985

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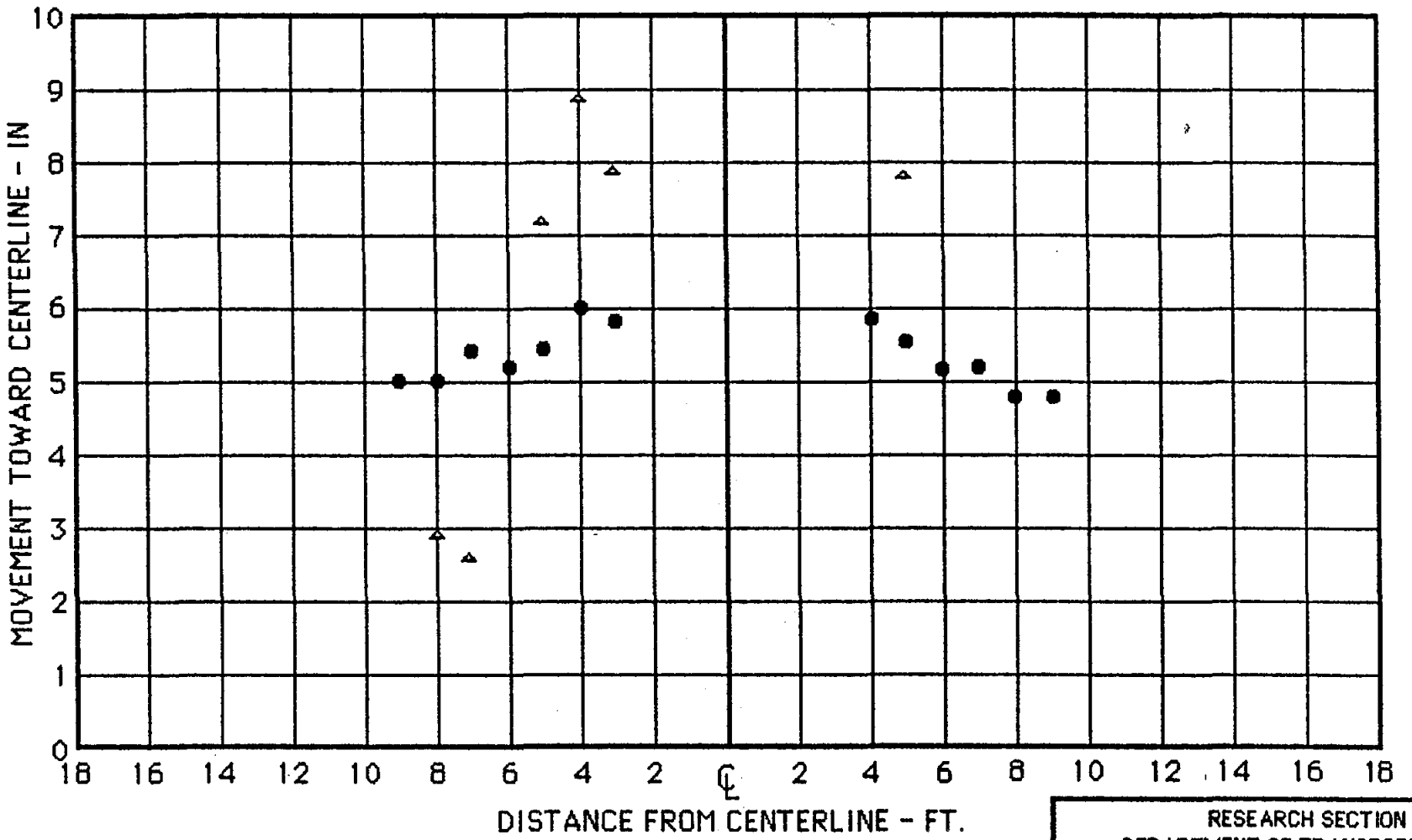
Fig. C-6



Key:
- ● Reinforcement Material
- △ Gravel

→ | TRENCH | ←

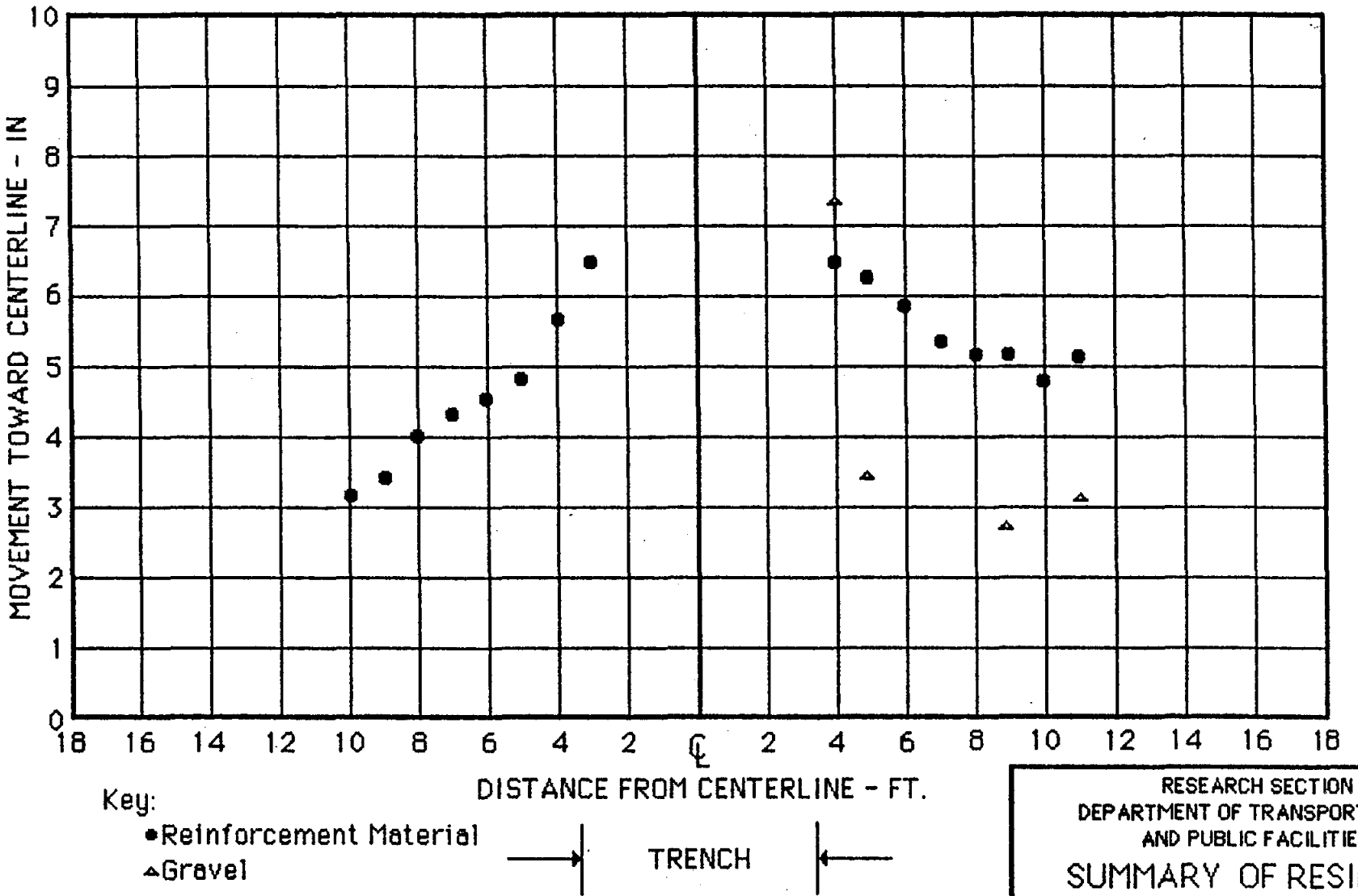
RESEARCH SECTION
DEPARTMENT OF TRANSPORTATION
AND PUBLIC FACILITIES

SUMMARY OF RESIDUAL REINFORCEMENT MATERIAL AND GRAVEL LOCATIONS SECTION 7

K-0617 MAY 1985

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

Fig. C-7



RESEARCH SECTION
DEPARTMENT OF TRANSPORTATION
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SUMMARY OF RESIDUAL REINFORCEMENT MATERIAL AND GRAVEL LOCATIONS SECTION 8

K-0617 MAY 1985

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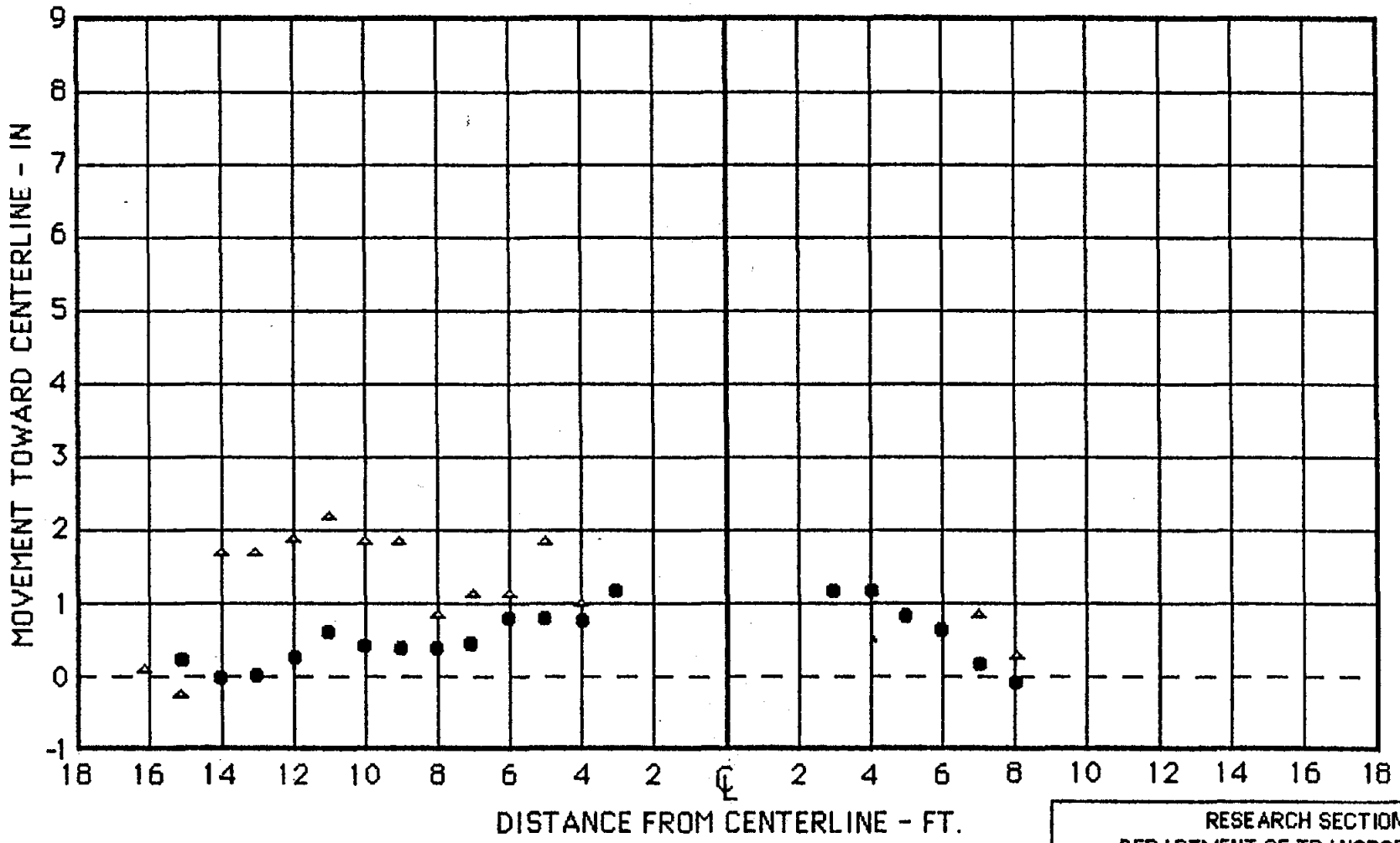
Fig. C-8



Key:
- ● Reinforcing Member
- △ Gravel

TRENCH

RESEARCH SECTION
DEPARTMENT OF TRANSPORTATION
AND PUBLIC FACILITIES

SUMMARY OF RESIDUAL REINFORCEMENT MATERIAL AND GRAVEL LOCATIONS SECTION 9

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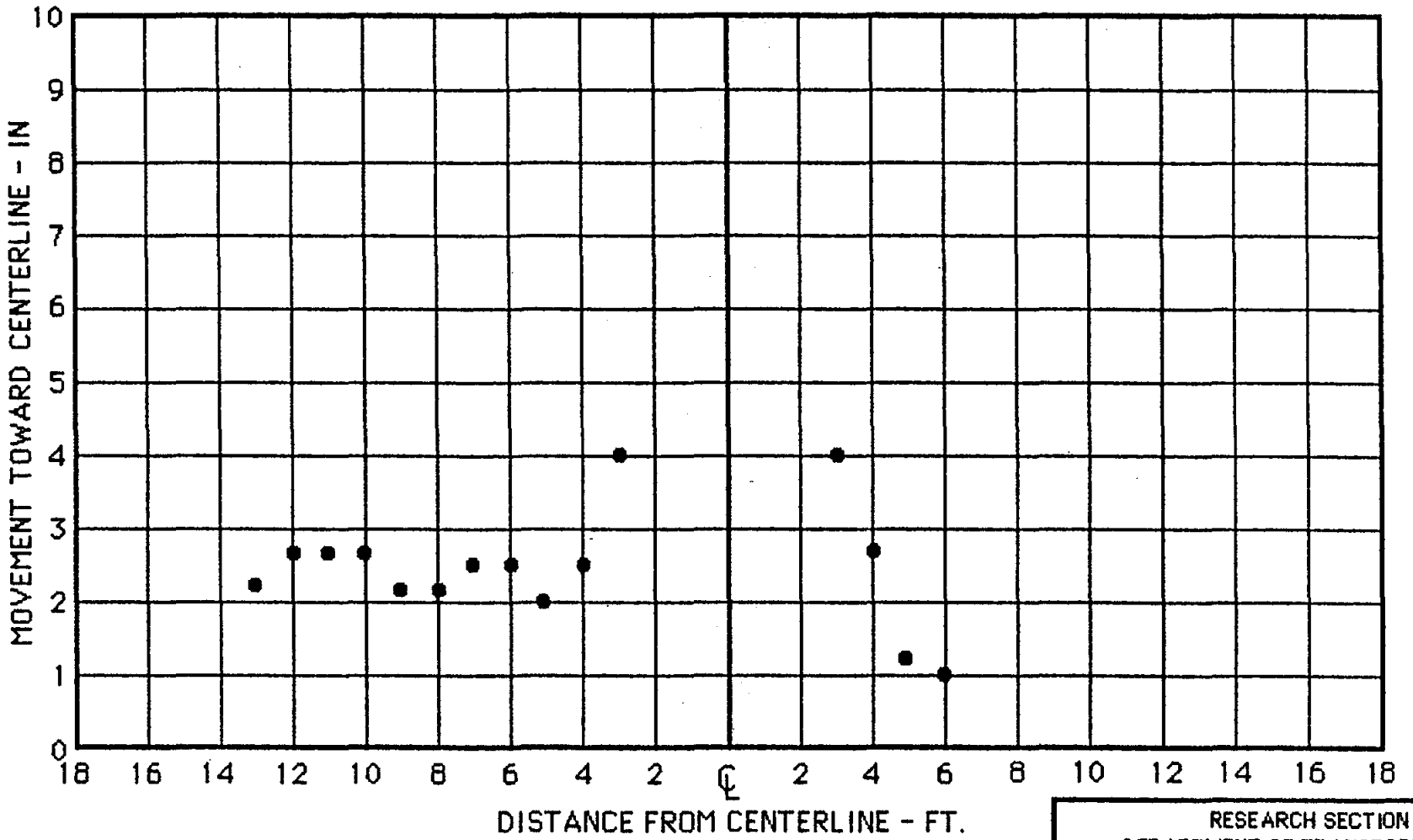
Fig. C-9



Key:
- ●Reinforcing Material
- △Gravel

→ TRENCH ←

RESEARCH SECTION
DEPARTMENT OF TRANSPORTATION
AND PUBLIC FACILITIES

SUMMARY OF RESIDUAL REINFORCEMENT MATERIAL AND GRAVEL LOCATIONS SECTION 10

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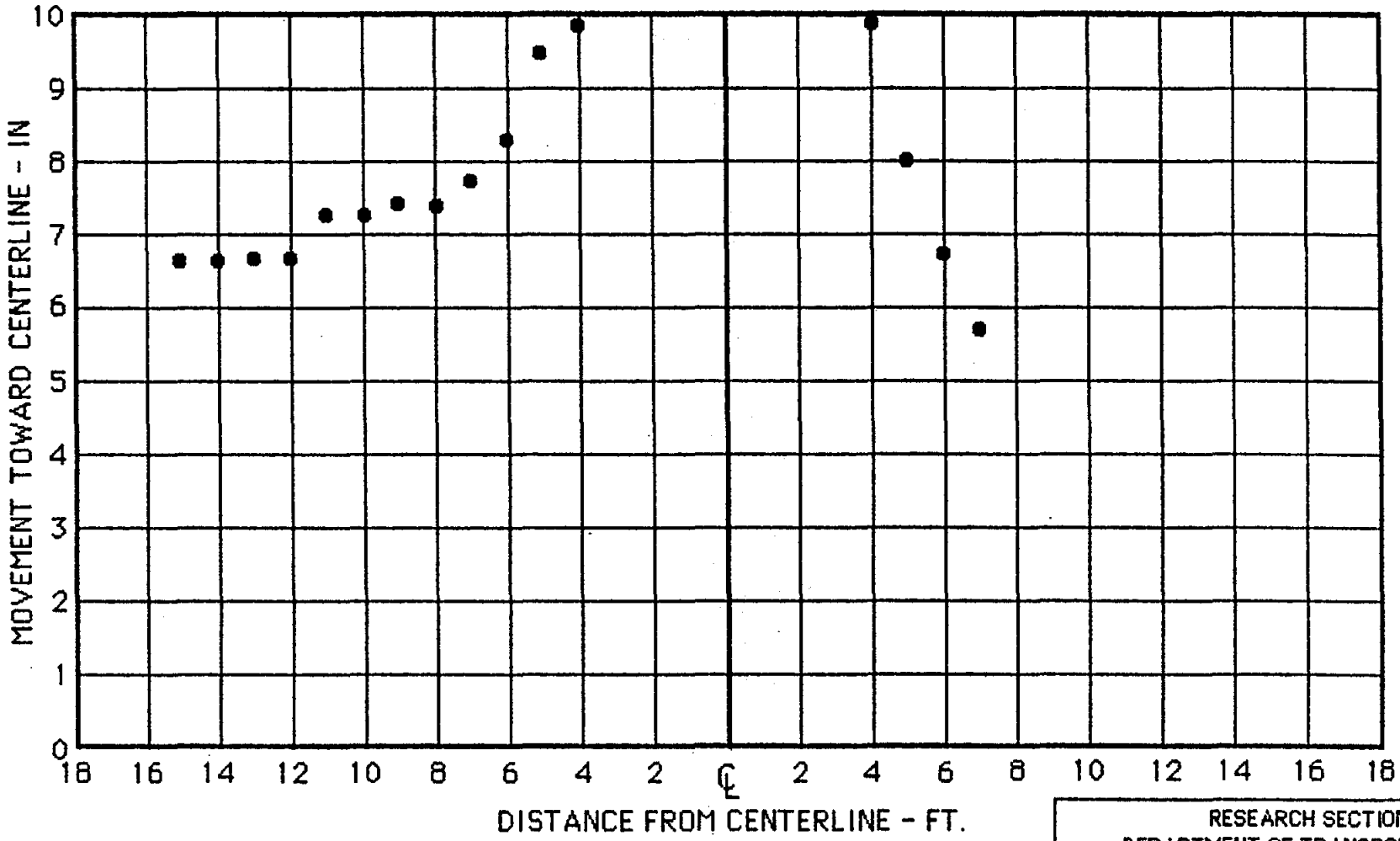
Fig. C-10



Key:
- ● Reinforcement Material
- △ Gravel

TRENCH

RESEARCH SECTION
DEPARTMENT OF TRANSPORTATION
AND PUBLIC FACILITIES

SUMMARY OF RESIDUAL REINFORCEMENT MATERIAL AND GRAVEL LOCATIONS SECTION 11

K-0617 MAY 1985

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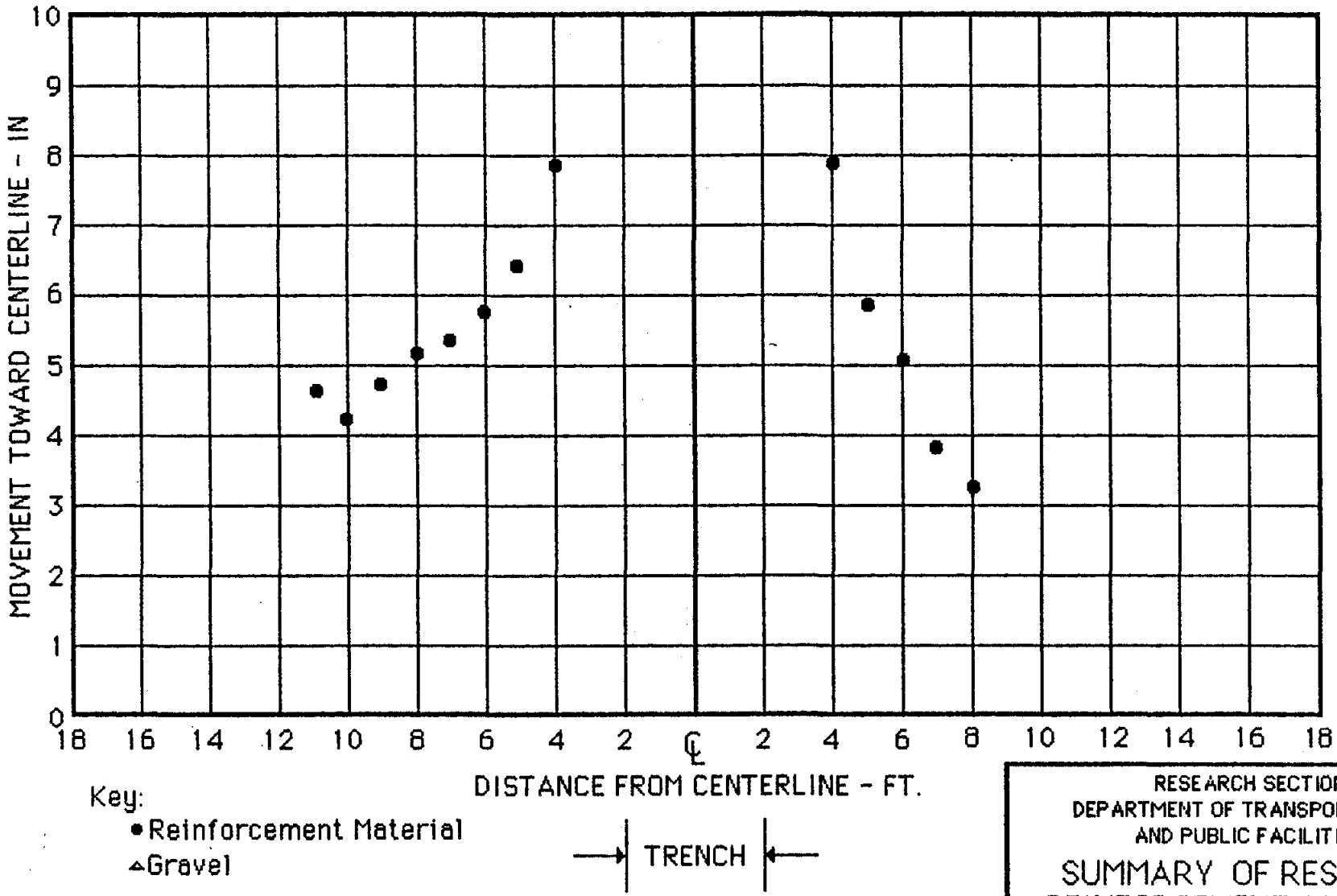
Fig. C-11



RESEARCH SECTION
DEPARTMENT OF TRANSPORTATION
AND PUBLIC FACILITIES

SUMMARY OF RESIDUAL REINFORCEMENT MATERIAL AND GRAVEL LOCATIONS SECTION 12

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Fig.C-12