TWENTY SEVENTH ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

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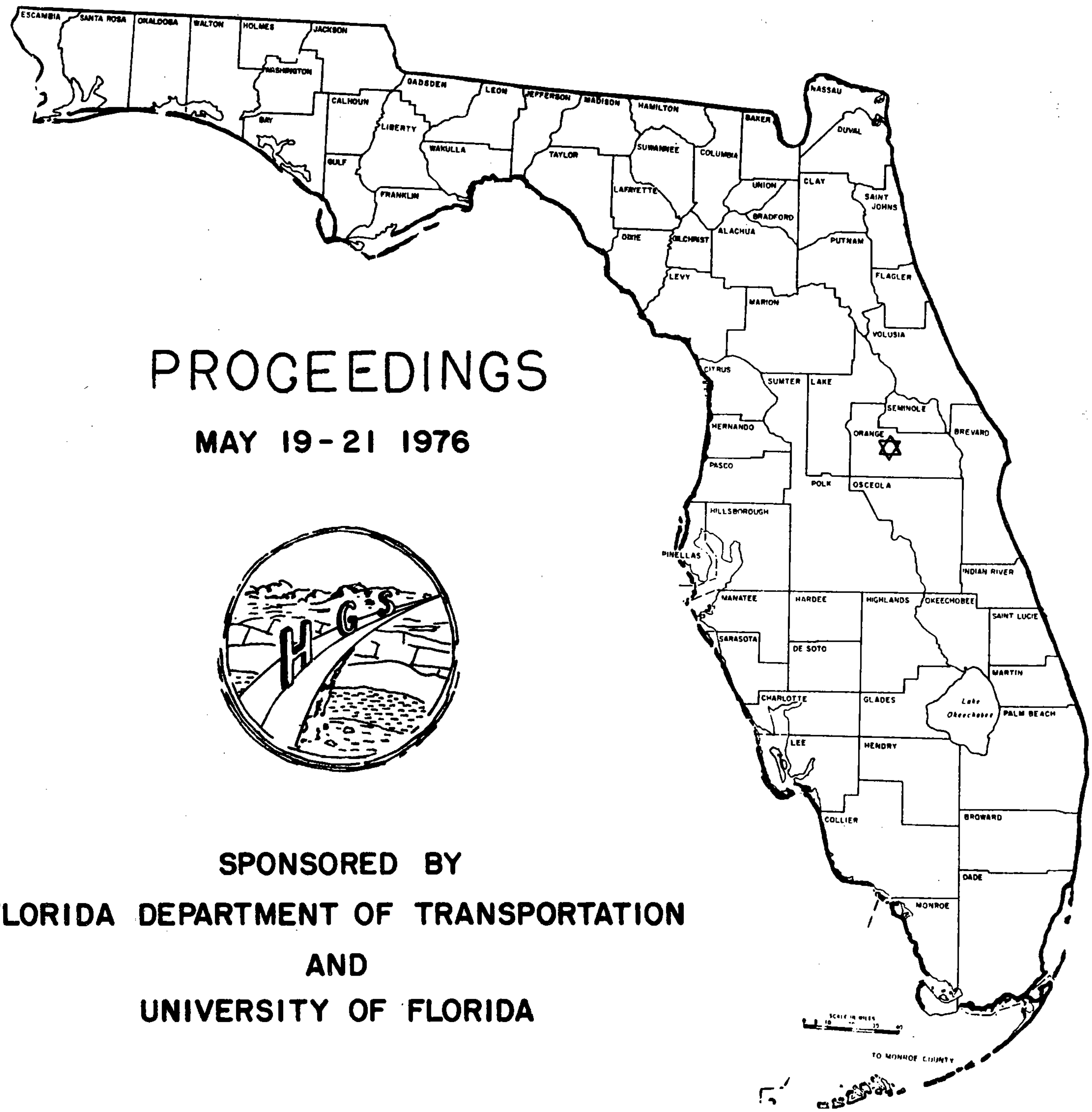

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Huckle, Horace F., United States Department of Agriculture Soil Conservation Service. A Resume' of a Study Between the Soil Conservation Service and the National Aeronautics and Space Adminstration to Determine if Remote Sensing could be Technically and Economically Developed as a Beneficial Tool to Assist Soil Scientists in Selecting Spots for Field Observations.

Kumar, Subodh, University of Arkansas. Shrinkage Factor for Fill Construction in Iowa.

McGrain, Preston, Kentucky Geological Survey. Lapies-Type Features in the Kentucky Karst Region.

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FLORIDA DEPARTMENT OF TRANSPORTATION

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A brief slide presentation giving Symposium participants a general view of Florida Geology and solution related problems in the State.

There is no Report.

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SUBSURFACE CAVITY DETECTION:
FIELD EVALUATION OF RADAR, GRAVITY AND
EARTH RESISTIVITY METHODS

BY

Lewis S. Fountain
Senior Research Engineer
Southwest Research Institute

San Antonio, Texas

SUBSURFACE CAVITY DETECTION: FIELD EVALUATION OF RADAR, GRAVITY, AND EARTH RESISTIVITY METHODS

By

Lewis S. Fountain
Senior Research Engineer
Southwest Research Institute
San Antonio, Texas

ABSTRACT

Gravity, ground-penetrating radar, and earth resistivity profiling were experimentally evaluated and compared as subsurface cavity detection methods. Tests were conducted in three different geological environments.

Verification tests showed that gravity measurements located large cavernous areas but did not detect mud-filled troughs; radar detected air-filled cavities at maximum depths of 4.6 metres (15 ft.) at one site, only penetrated three (3) metres (10 ft.) with inconclusive results at a second site, and could not resolve 0.6-metre (2-ft.) diameter vertical cylindrical cavities at another. Earth resistivity measurements using a pole-dipole electrode arrangement located cavities at all sites, indicating targets at depths of 24.4 metres (80 ft.). Both air-filled cavities, including vertical cylinders, and mud-filled troughs were detected using the resistivity technique, giving accurate depth and size resolution. A large mud-filled trough was detected at a 9.1-metre (30-ft.) depth that extended below 30.5 metres (100 ft.).

The earth resistivity technique was judged to be the most successful and practical underground cavity detection method in the environments tested. It was also found to be capable of delineating the irregularities of the bedrock at the soil-rock interface.

1. INTRODUCTION

Sudden collapse or subsidence above unknown cavities, results in extensive damage and property loss; and corrective action costs are very high and not always positive cures. This is a very serious problem in the construction and maintenance of highways and railroads. The public costs for damages, property losses and accidents would be greatly reduced if subsurface earth structural conditions along transportation routes and at building construction sites were known prior to final planning and construction.

Presently, the only reliable method for locating underground cavities of concern in highway planning and construction stages is by direct drilling. The time and cost of this method, however, is generally restrictive because of the close spacings required between borings in order to reliably detect and delineate possible underground cavities.

The subsurface cavities of concern in most highway stability and construction are those located within 15 metres (50 ft.) of the surface. They may of various sizes and shapes and may contain various amounts of air, water, or soil. Broadly speaking, conventional geophysical exploration techniques have been of only limited success in detecting subsurface cavities of this type because of size resolution difficulties and because of low contrast between the various observable cavity manifestations and the typical background conditions.

The presence of possible underground cavities may be noted from subtle indirect surface anomalies under ideal conditions using airborne sensing techniques. According to Warren and Wiechowsky (1) who used infrared photography, thermography, and side-looking airborne radar to study subsidence and collapse problems in several carbonate terrains, the results of aerial surveys can be a good beginning point for further geologic and hydrologic studies. Newton (2) also found remote sensing (multispectral photography andinfrared imagery) useful in delineating features such as water loss in streams, geologic structures, and vegetative stress related to sinkhole formation.

Surface-operated geophysical exploration methods are more intimately responsive to specific subsurface conditions related to underground cavities and offer the best potential promise for cavity detection success. Of these methods, improved versions of electrical resistivity survey techniques, a newly emerging electromagnetic technique utilizing a ground-penetrating radar concept and microgravity surveys appear to offer the most effective and practical approaches to underground cavity detection for highway survey applications.

Lakshmanan (3) and Colley (4) both reported successful cavity detection using gravity measurements in 1963. Neumann (5) also successfully demonstrated in 1972 that detection of solution cavities was possible by gravity measurements if extreme care was taken in making the measurements on a very tight grid and topographic features were carefully accounty for.

Ground penetrating impulse radar is one of the most recent instruments to show promise for subsurface cavity detection. (6, 7, 8, 9). Work done by Morey (9) for the Federal Highway Administration was probably the latest attempt to use this method. His results showed the detection capability but only to depths of about 2.4 metres (8 ft.) in moist clay-rich soils. Possibilities appeared to be good if instrumentation was improved.

A third detection method found worth evaluating was earth resistivity measurements. In 1966 Bristow (10) described a search method using a pole-dipole electrode array that could resolve small underground cavities. He used a graphical data display technique that allowed a bearing to be obtained on the location of the cavity as well as a prediction of the size and shape. Bates (11) in 1973 reported successfully detecting cavities using Bristow's method and modified the search procedure to allow more redundant data to be collected. The author in 1972 was successful in detecting shallow voids using a mobile equatorial dipole electrode array (12). Results from these studies were encouraging enough to warrant further evaluation.

The Federal Highway Administration still has work in progress evaluating subsurface cavity detection methods (13). Their earlier work led to the research reported in this paper and also to the search methods used (14).

The three (3) search methods selected for evaluation (gravity surveys, ground penetrating radar mapping, and earth resistivity surveys) were tested at sites in Alabama and Florida. All sites were sinkhole prone areas where sinkhole formation was known to be active. At each site all surveys were made over a common base grid pattern comprised of a rectilinear array of traverse lines equally spaced at a distance of 3 metres (10 ft.). The grid points were identified in matrix fashion by row and column number. In this manner a direct comparison of test results could be made.

Porous-pot electrodes were used as potential electrodes and metal electrodes were used as current electrodes. Porous-pot electrodes are non-polarizing electrodes used to eliminate problems and errors caused by galvanic action between metal electrodes and the soil. Contact resistance between the bottom of porous-pot electrodes and the ground can be as high as several thousand ohms; therefore copper-clad steel electrodes were used as current electrodes.

The main differences among the various electrical resistivity geophysical profiling methods are largely in the electrode array patterns used. The manner in which the electrodes are moved or scanned over the area being surveyed also differs with the different electrode arrays as do the methods of resistivity data analysis and interpretation. The pole-dipole array has had less previous use and has shown the greatest potential for detecting underground cavities and predicting their depths and locations. This method will be discussed in detail below.

2. Pole-Dipole Earth Resistivity Electrode Array

a. Theory and Method

The pole-dipole electrical resistivity survey method is based on a four-electrode, straight-line array configuration in which the current sink electrode is located at infinity and the potential electrodes are separated from one another by a fixed minimum distance proportional to the desired resolving power of the system. The potential electrode pair is located at various positions along the array line on both sides of the current source electrode as the means of vertically sounding the subsurface below the current source electrode. The current source electrode is moved ahead at suitable incremental distances to provide horizontal profile scanning. The pole-dipole array is illustrated in Figure 1.

In order that the equipotential surfaces be hemispherical and concentric about the source electrode, the sink electrode must be located at effective infinity which will generally be about 5 to 10 times the largest value of detection penetration depth of interest in the survey.

For clarity of explanation, the overlapping survey procedure is described as follows for a typical 30.5-metre (100-ft.) penetration depth survey:

(1) Place the current source electrode, C_1 as shown in Figure 1 at the first traverse station;
(2) Place the current sink electrode C_2 at a minimum distance of 152.4 metres(500 ft.) $(5r_2)$ and behind C_1 on the pre-established traverse line having already decided that the maximum potential electrode scan distance from C_1 will be 30.5 metres (100 ft.);

II. INSTRUMENTS USED AND SURVEY METHODS

A. Gravity Survey

The gravity meter used in the evaluation tests was a Lacoste Model "G". It has a vernier scale permitting gravity readings to the nearest 0.001 milligal. Repeatability is within 0.01 milligal.

At the test sites where gravity surveys were made, the elevation of each grid point was measured to the nearest 30 millimeter (0.1 ft.) so that elevation variations could be compensated for in the gravity readings. In the gravity survey a minimum of two measurements were made at each station, and the measured values then average. Gravity data analysis and display was in the form of a Bouguer gravity map compiled for each test site. Gravity values to 1 microgal and contour intervals of 20 microgals were plotted on these maps.

B. Radar Survey

The radar system used in the evaluation tests was manufactured by Geophysical Survey Systems, Inc. The van-mounted system transmits a base-band voltage pulse of approximately three (3) nanoseconds in time duration. The resulting quasigaussian pulse waveform is radiated into the earth by means of a broadband antenna. The radiated signal is an electromagnetic transient having a frequency spectrum with -3dB points at about 30 MHz and 120 MHz. The pulse peak power is 35 watts with an average power of 5.2 milliwatts. A two-way transmission loss of 110 dB is claimed for the system, indicating the ratio peak radiated power to minimum detectable received signal power.

Radar field data are collected in a continuous profile by pulling the sled-mounted antenna assembly across the area of interst. Real-time profile data are displayed both on an oscilloscope and on a graphic recorder while simultaneously being recoreded on a magnetic tape recorder for subsequent laboratory playback and analysis.

The data consists of signals reflected or scattered from subsurface anomalies such as voids. Depth to the target is indicated by the time delay between pulse transmit time and signal receive time.

C. Resistivity Survey

1. Instrument and Method

Earth resistivity surveys were made using a Keck Model 1C-69 earth resistivity instrument. The resistivity instrument is a dc system obtaining power for earth current from dry cell batteries having a total capability of 630 v. Resistance can be measured with the instrument over the range 0.001-1000 ohms, And the dial can be read to one part per thousand.

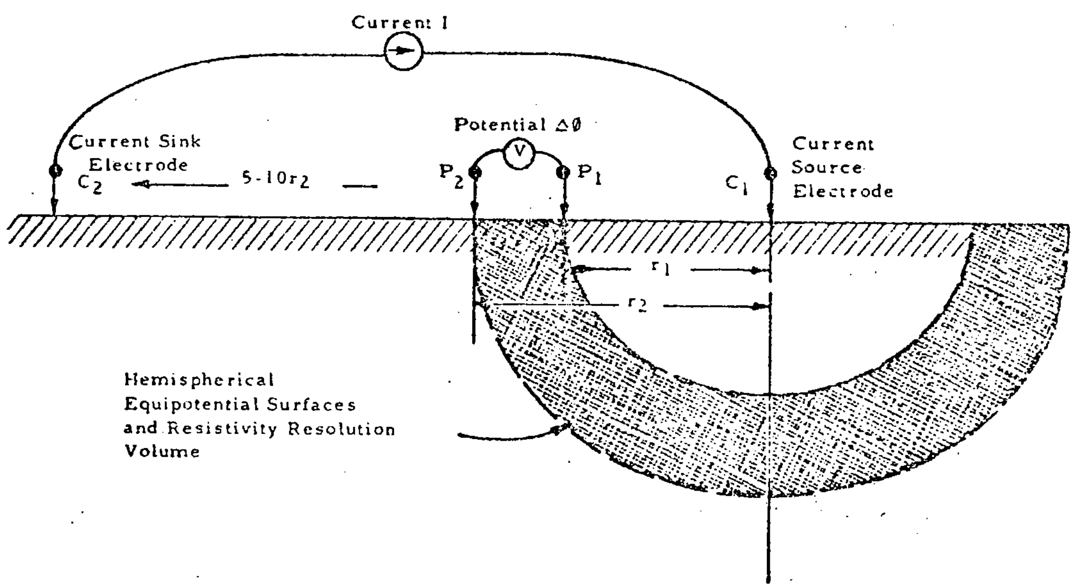


FIGURE 1. POLE-DIPOLE EARTH RESISTIVITY ELECTRODE ARRAY

the fourth column is the geometrical factor required to calculate apparent resistivity for the distances listed in the first column using the resistances recorded in the third column; and the fifth column is the calculated value of apparent resistivity in ohm-meters x 10 (ohm-centimeters x 10^3).

The pole-dipole geometrical factor in column four is calculated from the relationship,

$$KPD = \frac{2\pi r1r2}{r_2 - r_1} \qquad (1)$$

where r_1 and r_2 are the distances of the potential electrodes, P_1 and P_2, from the current source electrode, C_1, as illustrated in Figure 1. The basic principles on which the geometrical factor, KPD, are derived are discussed in many texts on geophysical prospecting. (15)

Graphs of the forward and reverse apparent resistivity profiles are plotted on the data sheet shown in Figure 2. Those points on the profiles that indicate high or low resistivity perturbations away from the average profile trends are next identified and marked for transfer to a scaled drawing used to graphically locate the anomalous underground resistivity structures. A simple and liberal interpretation of high and low resistivity perturbations is permissible since the redundancy of the data and a required multiplicity of perturbations associated with each underground anomaly will delete the improper interpretations.

An example of the scaled drawing used in the graphical analysis is set up as shown in Figure 2. Those points on the profiles that indicate high or low resistivity perturbations away from the average profile trends are next identified and marked for transfer to a scaled drawing used to graphically locate the anomalous underground resistivity structures. A simple and liberal interpretation of high and low resistivity perturbations is permissible since the redundancy of the data and a required multiplicity of perturbations associated with each underground anomaly will delete the improper interpretations.

An example of the scaled drawing used in the graphical analysis is set up as shown in Figure 3 (not related to the data of Figure 2). The distance along the ground surface representing the survey traverse is marked, and the consecutive positions of the current probe are shown by the arrows marked C_1. The perturbations interpreted from the forward and reverse resistivity profiles for each current electrode station, C_1, are denoted on the bracketed lines drawn above the ground surface line. The high resistivity anomalies in this example are labeled H_1, H_2, and H_3. With a compass centered at each C_1 location on the ground surface line, arcs are drawn at distances representing the bounds of the high resistivity anomaly with respect to the current probe position. The pairs of circular arcs are labeled H_1, H_2, and H_3 corresponding to the interpreted anomalies. The space where the three sets of arcs intersect is the graphically derived location of the underground structure responsible for the high resistivity perturbations. The single isolated low resistivity perturbation shown in the example of Figure 3 is insufficient to provide a graphical intersection with other low resistivity perturbations and is therefore ignored. A useful arbitrary guide for taking advantage of the redundancy of the overlapping field measurements is to require a minimum of three arcs to intersect at a common anomaly location before it is interpreted with any confidence as a probable underground anomaly. Moreover, as illustrated in

(3) Place potential electrodes, P_1 and P_2, at pre-established station markers at 3 metres (10 ft.) and 6.1 metres (20 ft.), respectively, on the sink electrode side of C_1 and obtain resistance reading;

(4) Repeat step (3) above with the potential electrodes, P_1 and P_2, at pre-established station markers at 6.1 metres (20 ft.) and 9.1 metres (30 ft.), respectively, on the sink electrode side of C_1 and obtain resistance reading. Continue movement of the P_1 and P_2 electrodes in this manner until the final potential reading is obtained at 27.4 metres (90 ft.) and 30.5 metres (100 ft.) from C_1;

(5) Next, place potential electrodes at pre-established 3-metre (10-ft.) interval station markers on the opposite side of C_1 from C_2 and obtain resistance readings at each station pair out to the 30.5-metres (100-ft.) limit. This completes the survey procedure for the first current station position for C_1;

(6) Move C_1 up the traverse line a dinstance of 12.2 metres (40 ft.) and obtain resistance readings over the 30.5-metre (100-ft.) scan zones on each side of this current station;

(7) Repeat step (6) above until complete traverse line is surveyed.

The above survey procedure gives four (4) overlapping resistance readings for each 3-meter (10-ft.) spaced potential electrode pair station to a depth of more than 15.2 metres (50 ft.) as the survey proceeds. The four-level data overlap aids in resolving the locations and resistivity contrasts of the various high- and low-resistivity anomalies which may be encountered along the traverse.

A straightforward graphical analysis method was devised capable of utilizing and displaying all of the field data as a means for locating the experimental best-fit positions and depths of detected subsurface cavities. The success of this analysis approach is largely achieved through the spatial redundancy of the field data with the result that target ambiguities and false interpretations are minimized and improved cavity size and shape indications are derived.

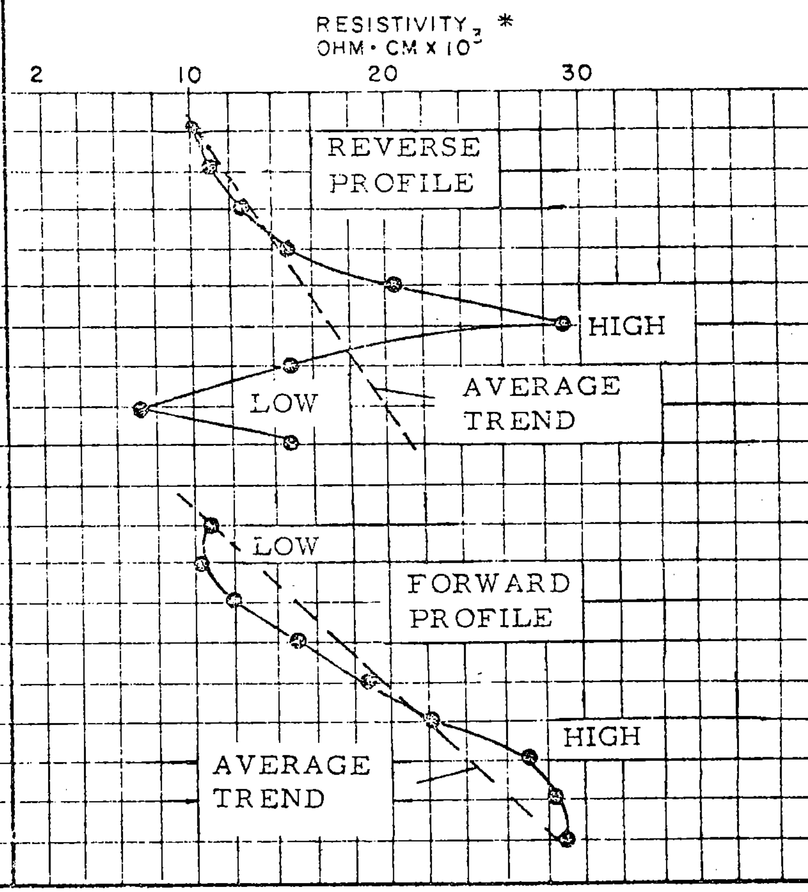
b. Graphical Data Analysis

The graphical analysis of the pole-dipole resistivity data can best be illustrated by first examining a sample of field data The basic field measurements as performed on this program were recorded on specially prepared data forms. Figure 2 is a sample data sheet showing recorded instrument readings, calculated resistivity values, and a graph of a derived resistivity profile. The first column on the data sheet is the potential-pair electrode distance from the current source electrode; the second column lists the distances from the current electrode to mid-point between potential electrodes; the third column is the resistivity instrument reading in ohms;

PROJECT SwRI 14-4013
TEST NO #7
DATE 28 Oct. 74 Monday 12:50 to 1:05 pm
WEATHER CONDITIONS Fair Warm

SITE I-59 ALA
AREA I-59, Birmingham, Alabama Grid Col. A
CURRENT PROBE POSITIONS +280 (Std. 387+30) +1000
OBSERVER H&K

P-P FEET	D FEET	READING OHMS	FACTOR 10^3	RESISTIVITY OHM·CM×10^3	INTERPRETATION
-10-20	15	2.67	3.83	10.23	
-20-30	25	0.969	11.49	11.11	
-30-40	35	0.550	22.98	12.64	
-40-50	45	0.386	38.30	14.78	
-50-60	55	0.394	57.45	22.64	
-60-70	65	0.365	80.43	29.36	Limestone
-70-80	75	0.138	107.25	14.80	
-80-90	85	0.048	137.89	6.62	Possibly Solution
-90-100	95	0.090	172.36	15.51	
+10-20	15	3.01	3.83	11.53	
+20-30	25	0.902	11.49	10.36	Possibly Solution
+30-40	35	0.511	22.98	11.74	
+40-50	45	0.388	38.30	14.86	
+50-60	55	0.321	57.45	18.44	
+60-70	65	0.275	80.43	22.12	
+70-80	75	0.252	107.25	27.03	Limestone
+80-90	85	0.203	137.89	27.99	
+90-100	95	0.165	172.36	28.44	



*1 OHM·CM×10^3=10Ω·m

FIGURE 2. SAMPLE RESISTIVITY TRAVERSE DATA SHEET WITH ANOMALIES MARKED

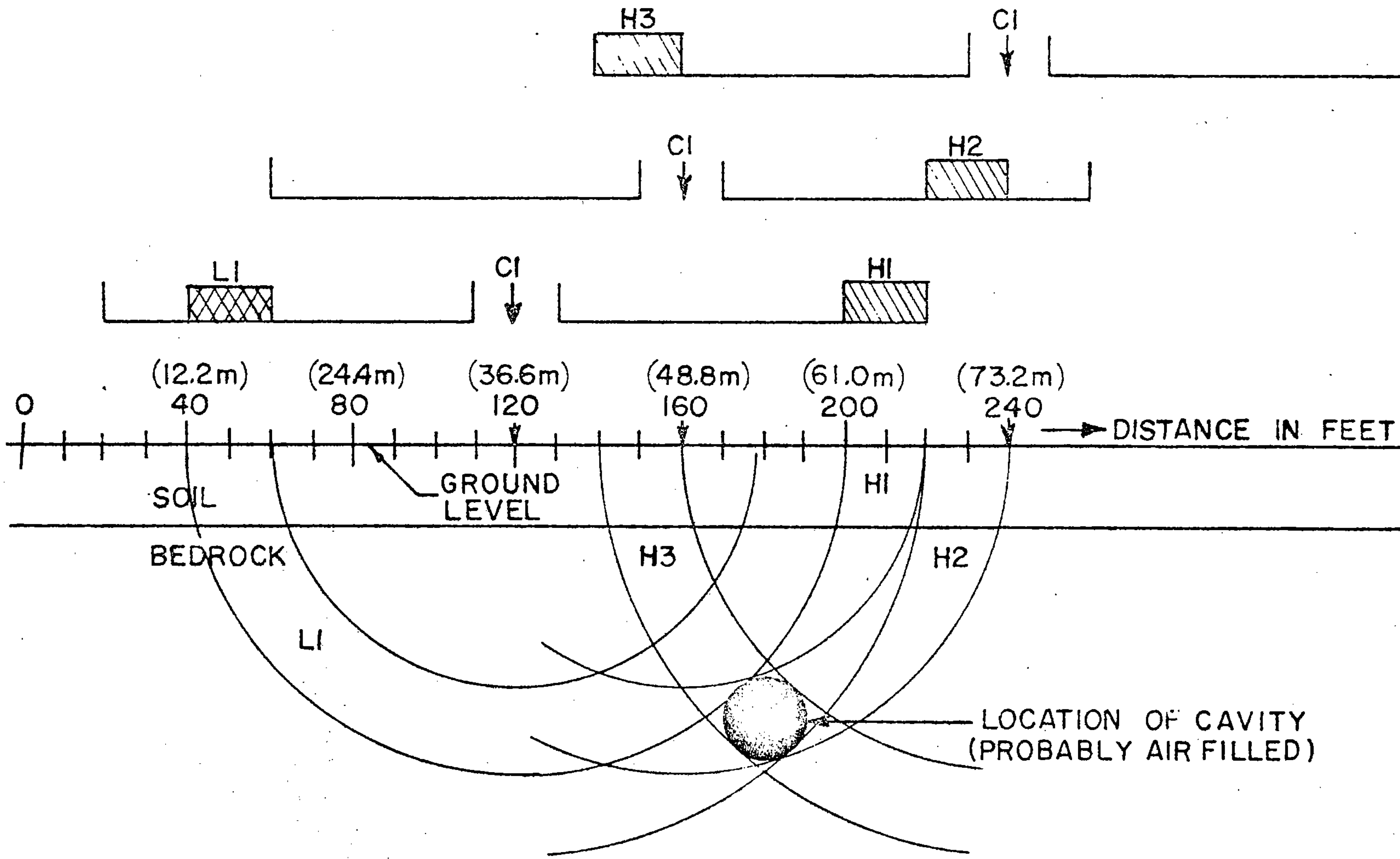


FIGURE 3. SIMPLIFIED SKETCH SHOWING GRAPHICAL METHOD OF LOCATING A RESISTIVITY ANOMALY

Figure 3 the arcs are drawn only in the 90-degree sectors corresponding to either the forward or reverse profiles containing the perturbations being used. The reason for this is that the distortions of the equipotential lines (represented in a first order manner by the circular arcs) are very weak if the perturbing anomaly is located in the opposite 90-degree sector.

It should be recognized that the pairs of arcs drawn for each resistivity perturbation describe not only the resistivity perturbations in the vertical plane along the ground traverse line but also apply, to some extent, to a three-dimensional spherical shell segment extending laterally on each side of the traverse line. It has been speculated by both Bristow and Bates that this apparent lateral field of view away from the traverse line is contained within an angle of about ± 25 degrees on each side of the traverse line relative to the source electrode location.

III. FIELD TEST RESULTS

A. Medford Cave Test Site, Reddick, Florida

One test site was selected near Reddick, Florida, south of Gainesville. It is on private property and has accessible air-filled underground limestone solution caverns known as Medford Cave. The underground caverns have some rooms ranging in size up to 12.2 metres (40 ft.) in width and many smaller passages. These caverns are primarily in the Miocene Hawthorne Formation limestone of the Alum Bluff Group but deeper portions are in the Eocene Ocala limestone. The subsurface cavities average about 3 to 9 metres (1o to 30 ft.) below the surface in most places, although some rooms reach depths down to 24.4 metres (80 ft.). The 0.9 to 1.8 metres (3 to 6 ft.) of soil on top of the limestone is sandy clay containing some phosphate and weathered limestone.

The first tests were made at this site because of the known features.

1. Gravity Survey

The gravity survey at this site was fairly successful. Major known caverns were detected, and a few other suspicious areas were noted. The survey consisted of gravimeter measurements at 563 stations on a 3-metre (10-ft.) rectilinear grid. As was discovered later when the cave was mapped, large near-surface joints and cracks were found to run in many directions from the main rooms of the cavern. This could have caused a very high level of "lighological noise" making recognition and interpretation of the gravity anomalies more difficult.

Two (2) gravity depressions were noted near cavities of relatively large volume. In these locations the cave was under about 6.1 metres (20 ft.) of overburden. In the vicinity of the largest of these anomalies, the roof thickness was about 3 to 4.6 metres (10 to 15 ft.).

Figure 4 is an outline map of the cave with the cavity detection results of the three (3) remote sensing methods superimposed. The principal gravity anomalies are shaded areas designated by the symbols A through E. Anomaly A was not verified. Anomaly B was drilled but no cavity was found. Anomaly C is probably a composite effect of both the large and small caves with probably an added effect from the large sinkhole near the small cave. Anomalies D and E are over the main cave complex which explains their cause. A number of smaller gravity anomalies were noted but not verified. Unexplained anomalies were very likely caused by cracks and fractures in the area.

In summary, it is concluded that the gravity method only detected the largest room of the Medford Cave complex with any certainty of results. Negative drill verification results on gravity anomalies of magnitudes comparable to those associated with the large room tend to reinforce this uncertainty of detection.

2. Radar Survey

Results of laboratory tests on soil samples from the area showed attenuation for the predominant frequency range of the radar to range from about 3.5 dB/m(1.1 dB/ft.) in the top 0.3 metre (1 ft.) of soil to 22 dB/m (6.7 dB/ft.) in the third 0.3 metre (1 ft.). Relative dielectric constant varied from a low of 3.8 to a high of 22.9 in the same depth range. Loss through the damp limestone is not known, but the above results appear to limit the radar performance to about 0.3-metre (10-ft.) penetration depths in this area for highly reflective underground targets. The final radar results verified this prediction.

The final results of the radar survey are shown in Figure 4. Six (6) anomalous subsurface areas are shown. One area is located over the main room of the cave and another over one edge of it. The largest of these was verified to be the cave ceiling at a depth ranging from 2.9 to 4.5 m (9.5 to 14.8 ft.). Another anomaly was noted in the vicinity of row 16, column 24 in the lower left of Figure 4. This anomaly is located near a fairly large room of the small cave section. The validity of this radar detection was not tested by drilling, but a fracture was observed in the roof of this room that extended in the direction of the indicated radar anomaly. The small anomaly near row 13, column 20, is over a large fractured area, a part of which was a man-sized passage. The surveyed depth below surface in this area was about 5.5 m (18 ft.). The large irregular anomaly centered on about row 12, column 15, was not drilled, but the depth below surface as predicted by the radar ranged from 2.7 to 5.2 m (9 to 17 ft.). The other sensing methods showed no significant anomalies corresponding with this location. The curved radar anomaly centered at about row 16, column 12 was drilled at grid row 15.5, column 11 to a depth of 15.8 metre (52 ft.). Rock was encountered at about 0.9 metre (3 ft.) that continued to a depth of 15.8 metre (52 ft.) where drilling stopped. The interpreted radar return was probably from the soil-limestone interface.

In summary, it is concluded that the radar technique was fairly successful at the Medford Cave test site. It detected the known or verified limestone cavities that were not more than about 4.6 metres (15 ft.) below the surface. It also detected soil-rock interfaces.

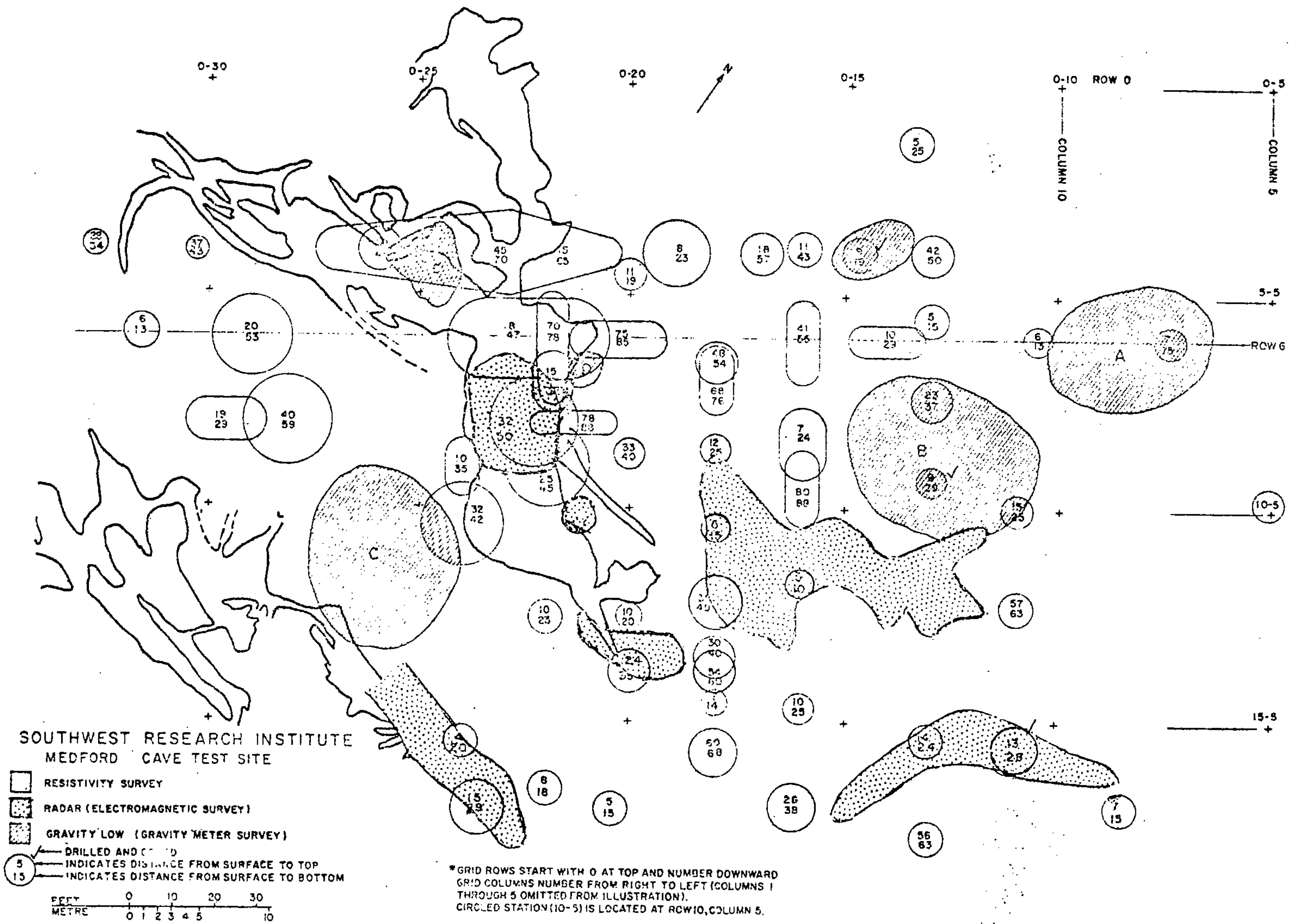


FIGURE 4. LAYOUT OF THE MEDFORD CAVE SITE SHOWING OUTLINE OF CAVE AND LOCATIONS OF UNDERGROUND ANOMALIES AS INDICATED BY THE THREE REMOTE SENSING TECHNIQUES

3. Earth Resistivity Survey

Eleven (11) lines were traversed at the Medford Cave test site using the pole-dipole electrode array with very good results. Over fifty-five (55) resistivity "high" anomalies were mapped, showing the interpreted depth below surface of top and bottom of the anomaly. A large number of these were located in the known cavernous area. Moreover, deeper subsurface cavities were indicated in at least one location under the floor of other voids. One of these deep voids was verified by the cave mapping crew. This deep room was narrow and interpreted to be approximately three (3) metres (10 ft.) high located at a depth of about 16.8 metres (55 ft.) under the main room whose floor was about 7.6 metres (25 ft.) below the surface.

The results of the survey are indicated on the map in Figure 4. The interpreted resistivity "high" indications are plotted as if they were spheres or cylinders. The dimensions shown for these anomalies are actually the predicted top-to-bottom dimensions of the anomaly as derived from the graphical analysis. In order to determine more detailed horizontal extensions of these anomalies, it would be necessary to obtain resistivity readings on a much tighter survey grid.

Figure 5 is indicative of the pole-dipole resistivity results derived for the Medford Cave test site. The data in this figure are from a traverse along Row 6 crossing over the main room of the cave. The circled numbers along the ground surface line in the figure indicate the current source electrode positions analogous to those illustrated previously in Figure 1. The interpreted high resistivity perturbations for each current station are indicated by the shaded blocks shown in the upper section of the figure. The lower portion of the illustration shows some of the measured apparent resistivity profiles from which the perturbations were derived. Only the reverse profiles are shown because of limited space.

Examples of high resistivity interpretations are identified on the lower section of the figure and labeled A3, A4, B4, etc. The high perturbations are transferred as arc radii to construct the subsurface zones of intersection as discussed earlier. The cavity labeled G in Figure 5 is the large cylindrical pattern on Row 6, extending from about Column 20 to Column 24 on Row 6 of Figure 4. The roof and floor of this cavity are interpreted to be at 2.4 and 14.3 metres (8 and 47 ft.) respectively.

Three (3) test borings at the Medford Cave site did not encounter additional cavities. Since an extensive verification description of most of the surveyed area could be obtained by mapping the cave, the various detection results indicated by the resistivity tests as well as by the other survey methods were verified by comparison with the cave map.

In summary, the resistivity survey was successful in detecting all of the known voids associated with survey paths traversing the Medford Cave test site. Many more anomalies were detected away from the mapped cave area and are predicted with good confidence to be other inaccessible and unmapped voids in the limestone bedrock.

B. U. S. Highway 19, Chiefland, Florida

A second field test site selected in Florida is located about 3.2 kilometres (2 mi.) north of Chiefland, Florida, along U.S. Highway 19. This area is underlain by the Ocala limestone of the Jackson age in the Eocene Series. It is generally covered with sand from the ground surface to a depth of about 3 metres (10 ft.) with a fairly heavy clay-sand mixture (argillaceous sand) down to about 4.6 to 6.1 metres (15 to 20 ft.). Soft limestone is generally found below the surface soil materials, and harder limestone exists at greater depths.

Sinkhole cavities in this area are generally in the form of vertical pipes a metre or two (a few feet) in diameter with depths of 15.2 metres (50 ft.) or greater. Some pipes are air filled, and some are filled with sand and clay mixtures.

Two (2) of the remote sensing techniques are evaluated at the field site at Chiefland, Florida, the ground penetrating radar system and the earth resistivity system.

1. Radar Survey

The results of the radar survey at U.S. Highway 19 showed good penetration depth but no verified cavity detection. This performance can be explained in terms of the measured electromagnetic properties of the soil encountered at the site.

The electromagnetic attenuation constant for the radar signal frequency range varied from less than 1.0 dB/m (0.3 dB/ft) to about 2 to 3 dB/M (0.6 to 0.9 dB/ft) from the ground surface to a depth of 1 metre (41 in.). Core samples were not taken below 1.0 metre (41 in.) but it was found later during verification boring tests that fairly dry sand extended to an average depth of more than 3 metres (10 ft.). These low values of attenuation rate will permit two-way radar signal penetration to depths of about 8.1 metres (30 ft.) assuming the presence of highly reflective subsurface targets and allowing for geometrical divergence of the radar energy.

The electromagnetic soil properties at the U.S. Highway 19 test site were such that radar could have been more successful at this location. A shorter wavelength radar system with an improved two-way transmission loss capability should find most of the piping in this Florida location

2. Earth Resistivity Surveys were conducted along each of the lettered grid lines in Figure 6 covering a total 213.4 metres (700 ft.) per line. The same spacing was used as before: 3-metre (10-ft.) potential electrode spacing moved in 3-metre (10-ft.) increments over 30.5-metre (100-ft.) forward and reverse traverses from the current electrode with current electrode moved in 12.2-metre (40-ft.) increments. Only the traverse lines running parallel to the highway were surveyed because of lack of space to run cross-grid traverses.

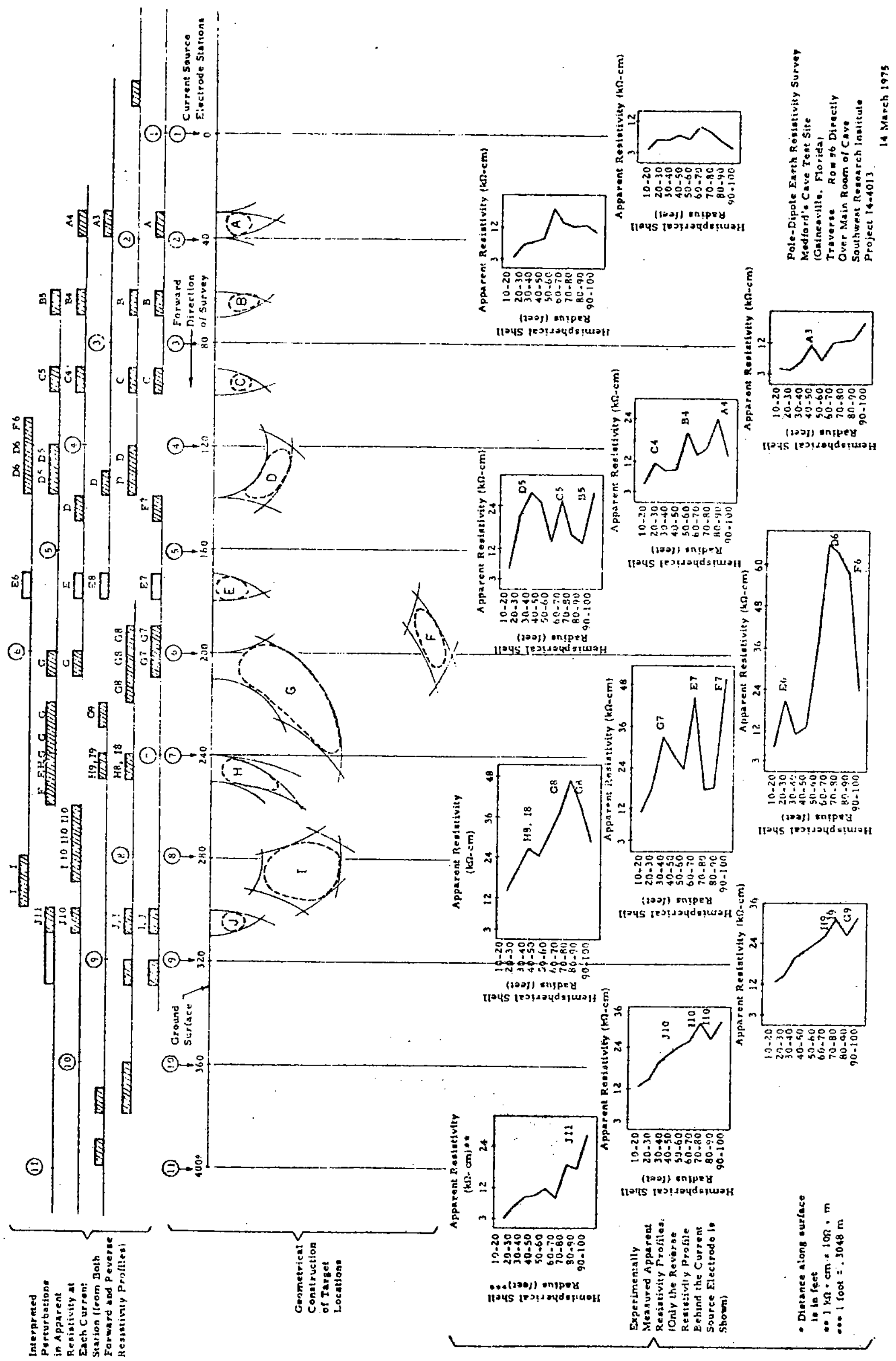


FIGURE 5. GRAPHICAL POLE-DIPOLE RESISTIVITY SURVEY RESULTS FOR TRAVERSE ROW 6 OVER MAIN ROOM OF MEDFORD CAVE

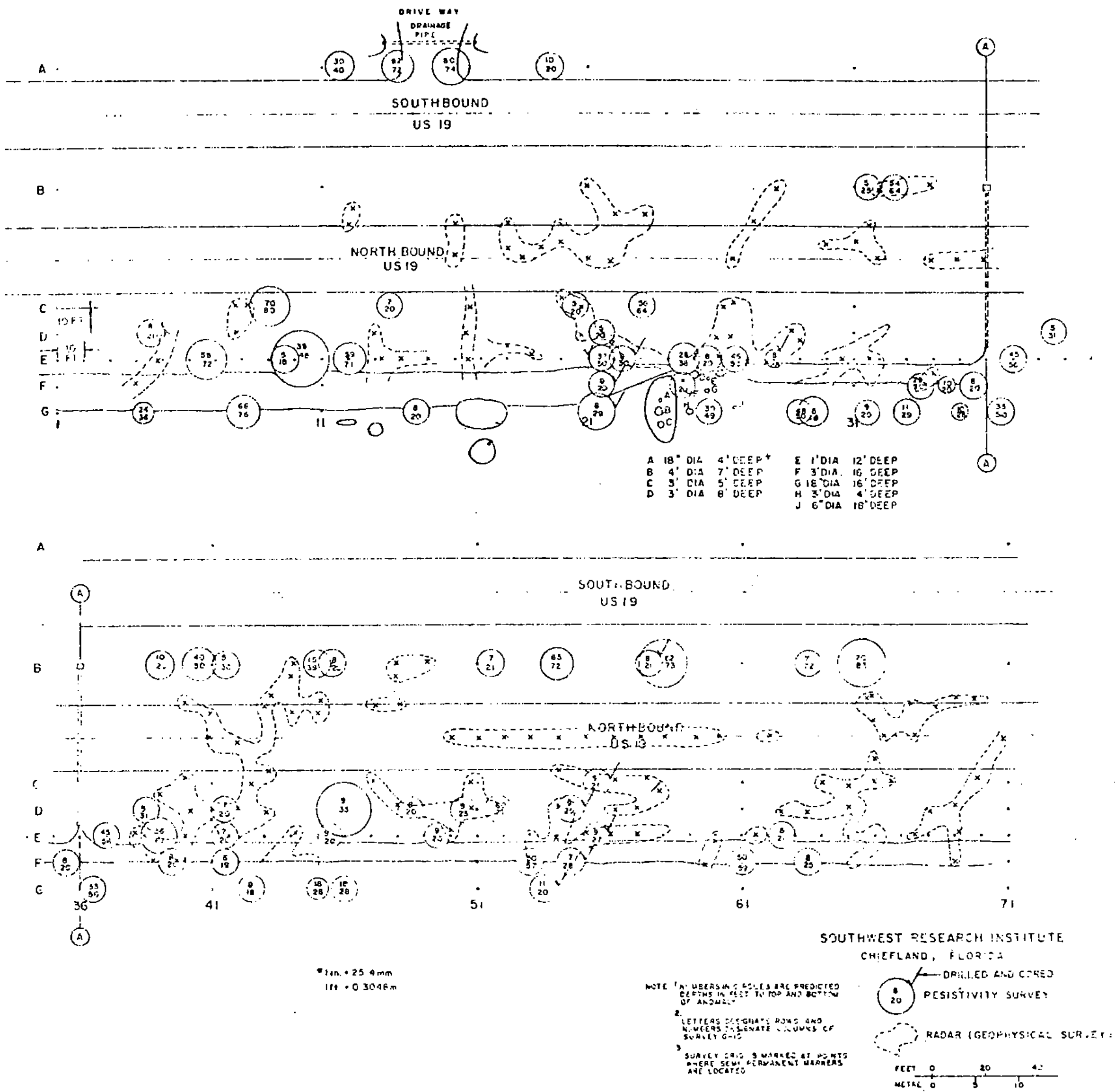


FIGURE 6. LAYOUT OF THE U.S. HIGHWAY 19 TEST SITE SHOWING LOCATIONS OF UNDERGROUND ANOMALIES AS INDICATED BY THE RADAR AND EARTH RESISTIVITY SURVEY TECHNIQUES

Particularly good results were obtained. Only the interpreted high resistivity anomalies are shown in the map, although some low resistivity anomalies were found in the moist sandy clay at some depths. Seventy-eight (78) high resistivity target areas were marked. Because of the limited time and availability of drilling equipment, only thirteen (13) verification holes were drilled. Most of the locations were selected in areas having both resistivity and radar anomalies. Out of the thirteen (13) holes, five (5) different air cavities were encountered in both the predicted depth range and at unpredicted depths. Some of the inaccurately predicted target depths resulted from errors in interpretation of the complex resistivity data because of the multilayer sub-structure. In those test borings where predicted cavities were not encountered, it is possible that because of the small diameter of the vertical-pipe voids, their localized position was missed by the drill.

The effectiveness of the pole-dipole resistivity survey technique is shown clearly by the verification tests at grid location (C, 55.5). The resistivity anomaly interpreted at this location was a high perturbation having a graphically indicated depth range 2.7 to 6.4 metres (9 to 21 ft.). The drill tests revealed a cavity extending from 4.3 to 7.6 metres (14 to 25 ft.). The absence of this cavity at positions of ±1.5 metres (5 ft.) on each side of grid position (C, 55.5) indicated the limited lateral extent of the detected cavity.

Similarly, another small cavity was interpreted as being near the surface at grid location (E 28). Drill verification of this anomaly revealed a 0.6 metre (2-ft.) diameter 2.7-metres (9-ft.) deep pipe concealed under only 0.3 metre (1 ft.) of surface soil.

C. Interstate Highway 59, Birmingham, Alabama

The third site chosen was in Birmingham, Alabama. It was a portion of the highway right-of-way along Interstate Highway 59 in the Roberts Industrial Park area located 73.0 metres (240 ft.) from a very large limestone quarry. The test area is mostly clay underlain by a light gray Ketona Dolomite having an irregular surface at depths in the range 9.1 metres (30 ft.). Much of the layered limestone is badly cracked and fragmented probably caused by shocks from explosive charges used in the quarry only 73.1 metres (240 ft.) away that runs approximately parallel to the highway. Water is continually pumped to the surface for run off to provide work access to the quarry. This causes a lowering of the water table in the surrounding area. The entire area is very active with sinkhole formations of various sizes and depths.

All three detection methods were evaluated at the I-59 test site with only the earth resistivity method being successful in locating potential subsidence areas. The I-59 test conditions caused problems for all three survey methods. The radar test experienced extremely high electromagnetic attenuations in the soil resulting in limited depth penetration. The gravity survey was eventually aborted because of ground vibrations caused by the heavy traffic flow at such close proximity to the measurement stations. The traffic flow on I-59 at this location averages about 75,000 vehicles per day, a great many of which are large trucks. Under these conditions, large amplitude ground vibrations prevented gravity measurements at distances closer than about 15.2 metres (50 ft.) from the pavement. The large nearby quarry excavation also had an effect on the observed gravity values.

Earth resistivity measurements at the site encountered two problems: (1) a chain link fence with ground-contacting metal parts ran parallel to the highway at distances of from 3 to 21.3 metres (10 to 70 ft.) from the desired resistivity traverse lines; and (2) very strong dc earth currents, apparently caused by the various manufacturing processes in the area such as a metal plating plant, caused occasional problems in making accurate resistivity measurements. At various periods during the day these slowly varying earth currents were so strong that the survey was interrupted until after the disturbances stopped.

The earth resistivity survey at I-59 was successful in spite of site obstacles.

Although no subsurface voids were found, a total of 92 high resistivity anomalies were detected. Of this number 33 were augered to rock and 2 were cored into the rock. All "high" anomalies except two (2) were caused by the soil-bedrock interface, some of which were thin rock strata over mud-filled slots. The two (2) "high" anomalies not caused by bedrock were interpreted as base structures of highway light poles near the traverse path. In all of the verified subsurface anomalies, the predicted depth was correct to within 0.6 to 0.9 metre (2 to 3 ft.). A number of low resistivity areas were also detected usually under a rock layer indicating a mud- or solution-filled slot or trough.

Three (3) areas having low resistivity anomalies beneath higher resistivity soil or rock were verified as having large mud-filled subsurface troughs. At one location a shattered rock layer about 0.61 metre (2 ft.) thick was encountered. The verification auger broke through the rock at about 10.4 metres (34 ft.) and hit very soft mud that extended to a depth greater than 30.5 metres (100 ft.) below the surface. No attempt was made to drill deeper.

A second mud-filled trough was verified in an area where clay soil was found to a depth of about 7.6 metres (25 ft.). At about 7.6 metres (25 ft.) very soft mud was encountered and extended to greater than 19.2 metres (63 ft.). No attempt was made to drill deeper. .

A third mud-filled trough was found in the area in the vicinity of an earlier construction fill. Very soft mud was found in the 6.1 - to 9.1-metre (20- to 30-ft.) depth range. Nearby very soft mud was found in another hole in the 9.1- to 15.2-metre (30- to 50-ft.) depth range.

From the larger scale interpretations of the resistivity anomalies, there appears to be a deep mud-filled channel crossing under the highway in this area. Sinkhole activity was found on the north side of the highway in the test site vicinity where there is one sinkhole near the fence. Available drilling logs also showed soft mud to depths greater than 32.9 metres (108 ft.) in the median in this area. On the south side of the highway a number of sinkholes were observed at the surface along the right-of-way in this same area. These sinkholes had funnel type vents in the bottom where water entered underground probably to feed larger underground channels extending under the highway.

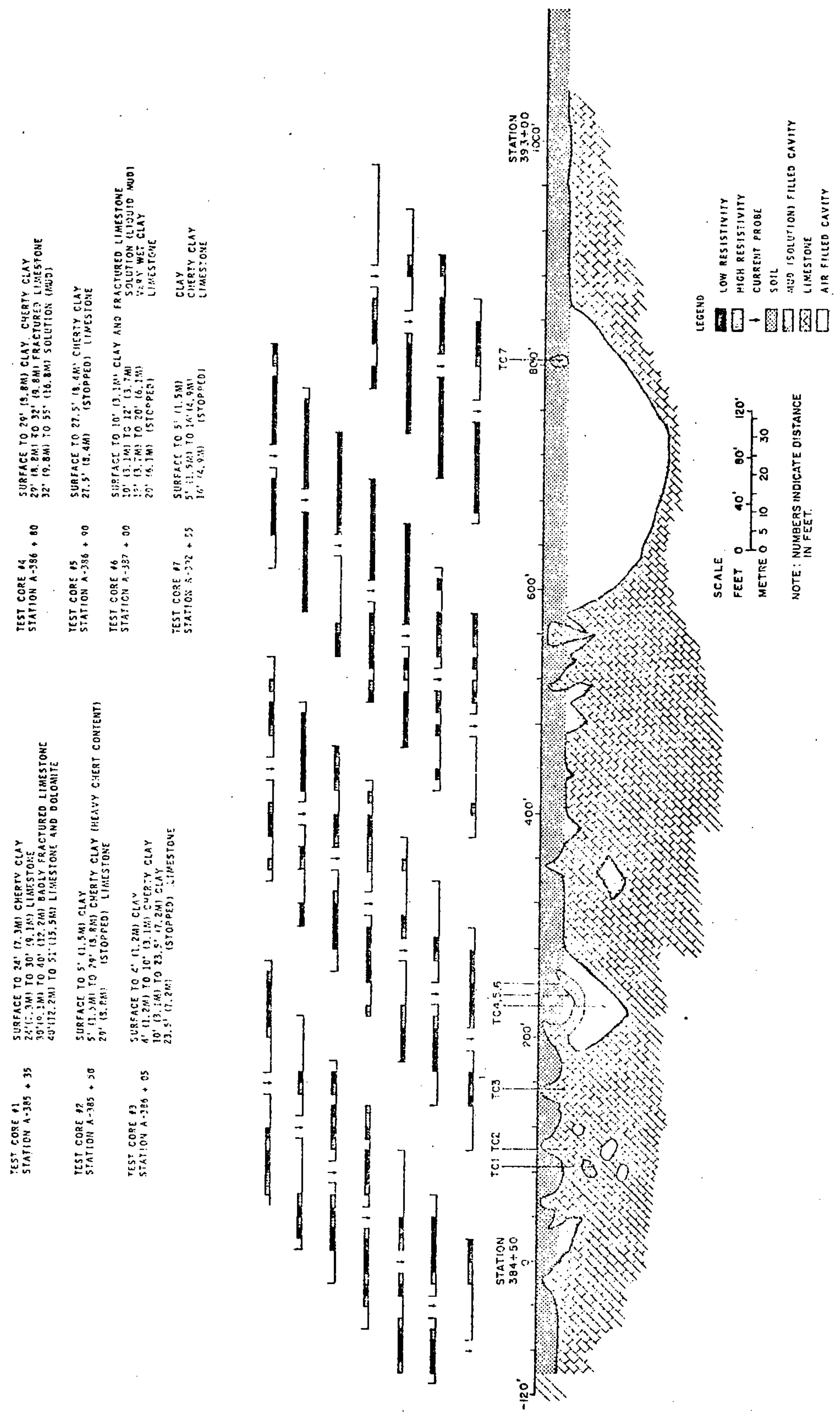


FIGURE 7. VERTICAL PROFILE UNDER SURVEY TRAVERSE A, INTERSTATE HIGHWAY 59 TEST SITE, BIRMINGHAM, ALABAMA

With the knowledge of average soil depth to bedrock, as gained from drill logs in the area, the close-spaced resistivity data can be more readily interpreted and understood. Figure 7 is the interpreted vertical resistivity profile along of the survey lines parallel to the highway about three (3) metres (10 ft.) from the pavement. The results of seven (7) of the test borings are also shown in this figure. The bars above the profile show the current electrode positions and the extent of the traverse on each side of the current electrode. The coded blocks on these lines show positions of high and low resistivity anomalies that enable geometrical constructions of the subsurface profile as explained earlier.

The three (3) small air-filled cavities were very strong "highs" on the data graphs, but they were not verified by drilling. The isolated low resistivity anomaly below the 118.9-metre (390-ft.) marker was not verified but was a strongly indicated anomaly. The undulating bedrock surface caused resistivity highs whenever a near-surface peak, "pinnacle", was encountered during a traverse, and numerous test borings verified their presence.

In summary, the earth resistivity survey was successful at the I-59 site, revealing not only soil-bedrock interfaces but also mud-filled slots, pinnacles, and overhangs that were verified by drilling. Enough information was gained from the data to allow a conceptual drawing of the complete subsurface profile structure down to a depth of greater than 15.2 metres (50 ft.) along a survey line 304.8 metres (1000 ft.) long.

V. CONCLUSIONS

Three (3) methods of surface remote underground cavity detection were field tested and evaluated on this program including: (1) gravity surveys; (2) electromagnetic subsurface profiling (ground penetrating radar); and (3) earth resistivity surveys. The following conclusions are drawn from the results of this work:

(1) Earth resistivity surveys were the most successful of the three (3) methods evaluated for detecting subsurface cavities, either air- or mud-filled. The method was successful at three (3) field sites all having different geologic structures and gave consistently good results. Both size and depth interpretations of detected cavities were generally good as verified by subsequent verification drilling.

(2) Earth resistivity methods were capable of locating both air- and solution- (mud) filled cavities in limestone.

(3) Good penetration depth and spatial resolution can be realized using the pole-dipole electrode array. Voids were detected and verified at depths of 24.4 metres (80 ft.). A small vertical air-filled pipe less than 0.6 metre (2 ft.) in diameter and 2.4 metres (8 ft.) in vertical length was also detected and verified.

(4) Information gained from close-spaced pole-dipole electrode resistivity surveys allows vertical profiles and subsurface structures to be interpreted for an area if basic geological information is known. Potential sources of surface problems caused by sinkhole activity can be detected.

(5) Currently available radar instruments can locate subsurface voids in some materials, but cannot reliably do so in all materials because of the variations in earth material electromagnetic properties. Penetration depth by the radar used in the tests ranged from a minimum of 2.4 to three (3) metres (8 to 10 ft.) in Alabama to about 9.1 metres (30 ft.) in Florida.

(6) Provided that penetration depth is adequate, radar can accurately measure the floor to roof dimension of an underground void since it receives signal echoes from both.

(7) The available radar could not detect 0.6- to 0.9-metre (2- to 3-ft.) diameter vertical pipe cavities in low loss earth materials characteristic of the sinkhole environment investigated in Florida.

(8) Present-day ground penetrating radar has potential advantages over other methods. The search rate is high, about 4.8-6.4 kilo-metres (3-4mi.) per hour. It has an immediate printout of data for quick field interpretation. If penetration depth is achieved, it can generally outline the shape, depth, and thickness of a void.

(9) Present-day ground penetrating radar, because of limited penetration depths associated with lossy soils, is generally inadequate for sinkhole surveys and other cavity detection investigations applicable to highway stability problems.

(10) Gravity perturbations caused by many underground voids are large enough to be detected using commercially available gravimeters. These voids generally must be fairly large and located at depths that do not exceed more than about one (1) diameter of a roughly spherical equivalent cavity. This detection capability depends upon the density of the surrounding earth materials and their variations. In practice, it might be generally found that a gravity anomaly of about 0.02 milligals can be detected, but this is probably the state-of-the-art limit. It has been found that normal gravity variations caused by the variations in the subsurface structure are greater than 0.01 milligal. As an example of the size of gravity anomalies that are to be expected, the calculated gravity perturbation from an idealized air-filled cylindrical void having a diameter of 12.2 metres (40 ft.) a floor to roof dimension of 4.3 metres (14 ft.), and a distance from ground surface to center of 11.3 metres (37 ft.) is 0.058 milligals. This is for an air-filled void having a density difference of -2.7 from its surroundings.

The gravity survey had only marginal success on this program.

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The State of Alabama Highway Department and the Florida Department of Transportation provided excellent field support in field site preparation and in test verification drilling.

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Appreciation is expressed to Katherine W. Taylor, owner of the Medford Cave near Reddick, Florida, for allowing tests to be conducted on the property.

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CURICULUM VITAE

LEWIS S. FOUNTAIN

EDUCATION

Universities	Location	Degrees
Arkansas State University	Jonesboro, Arkansas	---------
Trinity University	San Antonio, Texas	BS in Math MS in Physics

OCCUPATIONAL HISTORY WITH PRESENT COMPANY

1956-58	Sr. Electronics Technician
1958-62	Research Engineer
1962-71	Sr. Research Engineer & Project Manager
1971-75	Sr. Research Engineer and Acting Manager of an Earth Science Applications Sections
1975-	Sr. Research Engineer and Project Manager

Most work has been in special geoscience and earth science related problems including global soils studies and chemical, physical, electro-magnetic, and magnetic analysis of soils, but a good deal of work has been in non-destructive testing electric, seismic, ultrasonic, and acoustic search and detection methods.

DETECTION OF SUBSURFACE CAVITIES BY GROUND PROBING RADAR

By

REXFORD M. MOREY
GEOPHYSICAL SURVEY SYSTEMS, INC.

DETECTION OF SUBSURFACE CAVITIES BY GROUND PROBING RADAR

Rexford M. Morey
Geophysical Survey Systems, Inc.

"In a continuing effort by the Federal Highway Administration to evaluate the efficacy of ground penetrating radar for detection of subsurface cavities, an experimental program was undertaken during the Fall of 1974 at selected sites in Florida and Alabama. This paper outlines the results of the test program, describes the radar equipment, and presents theoretical considerations in the use of radar for subsurface cavity detection."

No Report Available for publication.

CURRICULUM VITAE

REXFORD M. MOREY

EDUCATION

Mr. Morey received a Bachelor of Science degree in Electrical Engineering from Northeastern University in 1963 and has taken additional graduate and executive development courses since that time.

RESPONSIBILITIES

Mr. Morey was the founder and the President of GSS. Presently he is the Chairman of the Board and Director of Research and Development. He conceived the Electromagnetic Subsurface Profiling (ESP) system and directed the design, development and implementation of the system. Along with certain responsibilities for the administration of the Company, Mr. Morey directs the Company's Research and Development activities.

EXPERIENCE

At GSS, Mr. Morey has participated and directed several ESP surveys in the Canadian Arctic. These surveys included both permafrost surveys to determine the location of large masses of ground ice and sea ice surveys for profiling ice thickness. Mr. Morey has written several technical reports and presented technical papers on various aspects of Electromagnetic Subsurface Profiling.

Mr. Morey was employed with EG&G, Inc. from 1967 to 1970, first as a Senior Engineer and then as Scientific Executive. His last position was one of three Technical Staff Members of the Engineering Directorate of a major division of the Company. The Directorate had a complement of approximately 100 engineers who were engaged in a variety of research and development projects. Additionally, his technical experience included Mission Manager for the EMP Mission and the Time Domain Radar Mission, being responsible to the Division for all aspects of the business development of these missions. He has been Proposal Manager for several Nuclear Weapon Effects Proposals, one of which was valued at over three million dollars. In this capacity, he was responsible for the development of the technical, cost and schedule portions of the proposals.

From 1963 to 1967, Mr. More was involved in numerous research and development projects dealing with electromagnetic propagation, antenna theory and design, and the design and fabrication of various radar antennas and components.

PROFESSIONAL AFFILIATIONS

Etta Kappa Nu.

A SHALLOW REFRACTION SEISMIC STUDY OF FOUNDATION CONDITIONS FOR THE SEVEN MILE BRIDGE, MONROE CO., FLORIDA

By

H. K. Brooks
Professor, Geology
University of Florida

A SHALLOW REFRACTION SEISMIC STUDY OF FOUNDATION CONDITIONS FOR THE SEVEN MILE BRIDGE, MONROE CO., FLORIDA

H. K. Brooks
Professor, Geology
University of Florida

The need for exploring foundation conditions between Knights' Key and Little Duck Key for the proposed new route of the oversea Seven Mile Bridge provided an opportunity to utilize one of the oldest of remote sensing techniques, refraction seisomology. Because of shallowness of water and the non-stratified, heterogeneity of the underlying reef rock, the reflection technique was of no value in characterizing the underlying rock, or even in locating problematic sites such as sinkholes.

Offshore test drilling is expensive and sediment masks the underlying rock relief and lithologic characteristics. To properly evaluate the overall geological and rock mechanic characteristics of the Seven Mile Bridge route would require a large number of test borings. Based upon seismic data, the number of exploratory cores has been reduced to thirty-eight(38). The simple twelve (12) channel refraction seismograph was adapted to the problem. A sixty-five (65) pound cannon, in which the source of explosive shock was placed directly on the bottom, was developed. A hydrophone array with each of the twelve (12) receiving units precisely spaced on the sea floor was utilized. Spacing of twenty (20) ft. intervals provided information over a linear test distance of 240 feet and to depths of sixty (60) feet below the sea floor. Problematic areas were later reinvestigated utilizing a ten (10) ft. spacing which provided greater detail.

The seismograph records velocity of travel of compressional waves through the underlying earth materials. There is a direct relationship between the velocity of compressional seismic waves, Young's modulus, Poisson's ratio and bulk density in homogeneous rock. These relationships have been evaluated. In the vuggy, heterogeneous reef rock at the Seven Mile Bridge site, the compressional wave velocity is probably a better test of the overall foundation conditions than are the test borings. Both tests, the velocity of travel and core recovery, are biased toward the hardest rock. In the 115 seismic tests performed, not only could the degree of hardness and heterogeneity of the underlying rock be determined, but significant offsets in the time-distance curve suggested anamalous areas that are either fracture zones or sinkholes.

<u>NO REPORT AVAILABLE FOR PUBLICATION</u>

APPLICATION OF GEOPHYSICAL METHODS

TO THE

DETECTION OF SHALLOW KARSTIC CAVITIES

by Gildas OMNES
Geoterrex Ltd.

Preprint of paper presented at the 27th Annual Highway Geology Symposium, Orlando, Florida, May 1976.

APPLICATION OF GEOPHYSICAL METHODS TO THE DETECTION OF SHALLOW KARSTIC CAVITIES

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ABSTRACT

Shallow karstic cavities may be defined as natural cavities where the shallowest part is not more than 15 feet below the ground surface and which have been created by dissolution phenomena in limestones. These Cavities may be presently filled with air, water or loose sediments.

Though good correlations may be observed locally between remote sensing results and the distribution of shallow cavities air photographs, aerial infrared surveys and microwave surveys lack sufficient depth of investigation and resolution.

Among ground electromagnetic methods pulse radar generally lacks sufficient penetration because dielectric looses in the overburden, or even in the limestone, are generally too high. Resolution falls sharply when penetration is increased through the use of lower frequencies.

Direct Current Resistivity methods have been used successfully in a large number of cases. Cavities filled with air give rise to resistive anomalies, cavities filled with salt water of clayey mud give rise to conductive anomalies. Unfortunately cavities filled with fresh water or loose sediments may be undectable.

Seismic methods do not have enough resolution for the detection of shallow relatively small cavities.

Gravity has been used continuously in Western Europe since 1963 for the detection of cavities. It has almost completely replaced the use of resistivity methods since 1971. Until 1969 it was necessary to survey each station at least twice in order to obtain the required accuracy with a standard gravimeter. Since 1969 it is possible to obtained an accuracy of 5 microgals (5×10^{-3} milligals) with the Lacoste-Romberg Model D "Microgal" gravimeter without repeating all stations. The production of a three man crew is generally about 60 stations per working day. Microgravity problems require an approach somewhat different from classical structural studies with an emphasis on accuracy control.

Experience has shown that a cavity with one horizontal dimension of the same order of magnitude as the depth causes a measurable gravity anomaly whether it is filled with air, water or loose sediments. On the other hand all negative anomalies are not due to cavities, some may be caused by lows of the bedrock topography or a higher density of dissolution joints within the limestone.

Examples of negative gravity anomalies due to shallow karstic cavities in Western Europe and Florida show frequently observed features such as the merging of negative axes near cavities and subquandrangular contours. It is particularly interesting to note that the actual amplitude of an anomaly due to a karstic cavity is generally more than twice the amplitude of the

theoretical anomaly calculated on the basis of the size of the void.

In the case of very shallow cavities and homogeneous surface conditions the measurement of the vertical gradient of the gravity field is an interesting technique because no accurate levelling is required, and the interpretation of results does not necessitate the separation of residual and regional anomalies.

INTRODUCTION

Shallow karstic cavities may be defined as natural dissolution cavities where the apex is not deeper than 15-20 feet below the ground surface. Most marstic cavities have been created by dissolution phenomena occurring above the water table in limestones. Karstic cavities may be presently filled with air, fresh water, sea water or sediments, generally sand and clay.

The detection of shallow karstic cavities is a problem of particular interest for highway construction. Shallow karstic cavities may cause collopses of bridges, causeways or the highway itselt. An obvious detection method is drilling shallow holes on a regular grid, although there are many drawbacks to this approach.

- drilling amounts to exploration along an axis, a dangerous cavity or a narrow dissolution joint may correspond to equivalent intersections on a drill hole log.

- because cavities a few feet in diameter and a few feet deep represent hazards for highways a thorough reconnaissance by drilling is costly.

- at the reconnaissance stage working with a light drilling rig across cultivated land causes damages and adds to the total cost of the reconnaissance.

These remarks underline the attractiveness of geophysical methods, particularly remote sensing methods, as possible reconnaissance tools.

Remote sensing comprises all methods where signals from a portion of the electromagnetic spectrum are received on-board of an aircraft. On the ground geophysical methods applied to the detection of cavities include electromagnetic and D.C. electrical methods, seismic and gravity. Visual observations on the ground may be considered as a particular case of remote sensing taking place near the surface of the earth, though the distance between the observer and the particular spot under study may vary considerably.

THE USE OF ELECTROMAGNETIC SIGNALS

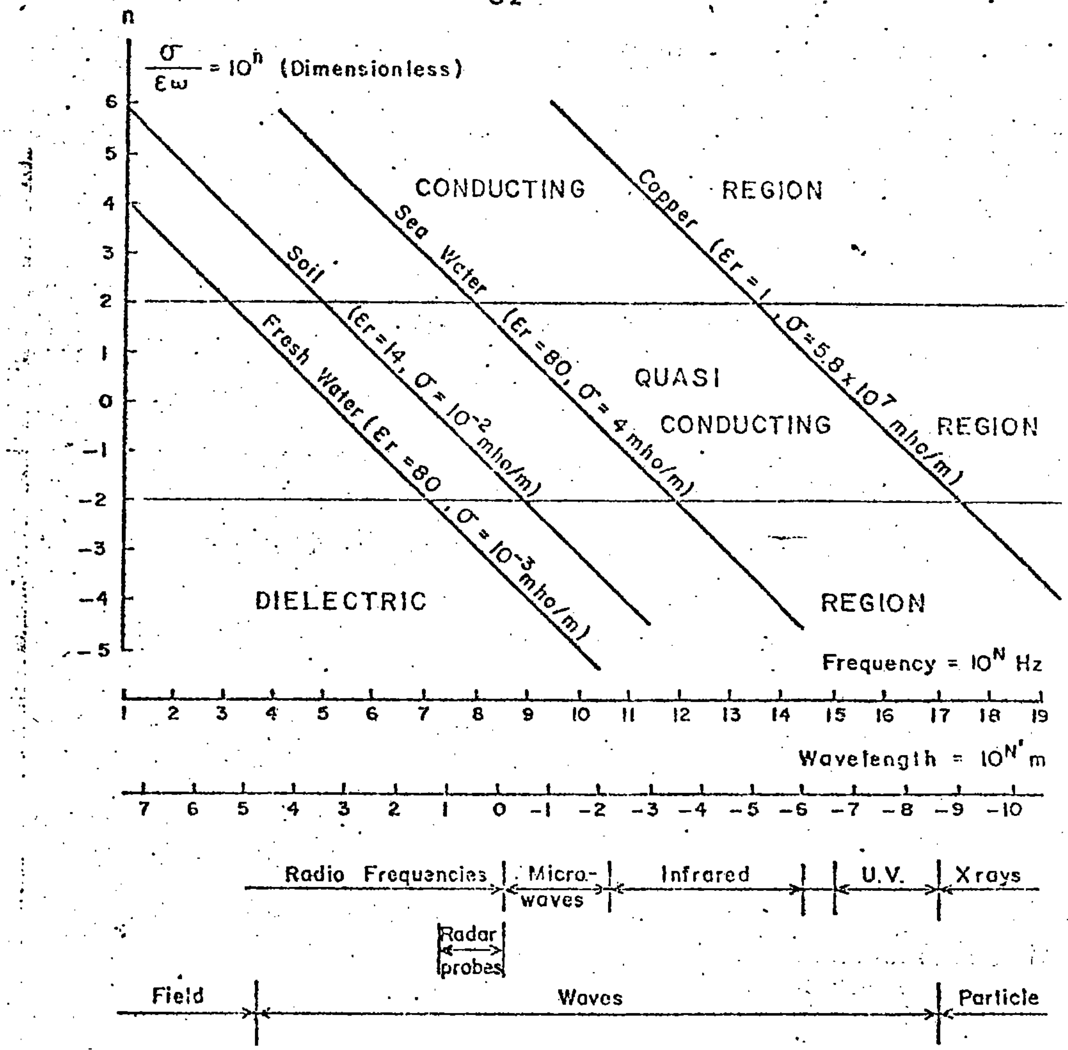
Figure 1 sums up in a semi-quantitative way the behaviour of electromagnetic waves in conducting media for frequencies ranging between 1K Hz and 10^{19} Hz.

The equation expresses the relation between the magnetic component Hx and the elctric component Ey for a plane wave where Hx and Hy are harmonic functions of time. is the conductivity in mhos and Ey represents the conduction current density while $j \hat{\omega} \varepsilon$ Ey represent the displacement current density.

(W = 2 x Frequency; is the permittivity in forads/meter. Thus it may be said that the space rate of change of Hy is equal to the sum of the sum of the conduction and displacement current densities.

FIGURE 1 : THE ELECTROMAGNETIC SPECTRUM

$$-\frac{\partial Hx}{\partial z} = \sigma Ey + j\omega \varepsilon Ey$$



The three (3) regions of Figure 1 where $/\varepsilon\omega$ is plotted in ordinate and the frequency is plotted in abscissa, correspond to the following cases.

- : The displacement current is much greater than the conduction current, the medium behaves as a dielectric.

- : The conduction current is much greater than the displacement current, the medium behaves as a conductor.

- : The conduction and displacement currents are of the same order of magnitude, the medium is said to behave as a quasi conductor.

It can be seen that depending on the frequency a medium may behave as a conductor or as a dielectric.

In the visible and upper infrared part of the spectrum all geological formations are in the dielectric region. The detection of shallow cavities is possible only where surface features are related to the cavities. This may be the case, aerial photographs and infrared pictures may show alignments of dolines filled or not with clay. These remote sensing methods are interesting because they are relatively unexpensive and may give positive results at the preliminary reconnaissance stage.

From the lower infrared to the upper part of the radio frequency range (100K Hz) most of the geological formations are in the quasi conducting region of Figure 1. Penetration is reduced by both the electrical skin effect and dielectric losses. In the electrical skin effect penetration is proportional to the square roots of the reciprocals of conductivity and frequency. Dielectric losses increase with frequency and the dielectric constant (1). Because the dielectric constant of water is particularly high the moisture content has a strong effect on the probing depth of pulse radar probes. Most pulse radar probes work in the 15-500M Hz range. In a very resistive material with a low dielectric constant such as salt, the probing range may exceed 1000 feet (2). Where moist clay or soil covers the bedrock the penetration of radar or microwave signals is generally less than 5 feet (3).

Except in very favorable conditions methods based on the use of microwave and radar signals cannot be considered as reliable tools for the detection of shallow karstic cavities, at least in the present state of technology.

At frequencies lower than 100K Hz most geological formations are in the conducting region of Figure 1, the main factor in the reduction of penetration is the skin effect, wave lengths are longer than 3000 feet and it may be said that anomalous bodies cause deformations of the electromagnetic field rather than radiation reflections. On the other hand, resolution decreases as the wave length increases. The penetration of a plane wave is the depth at which the amplitude of the signal is divided by e, meaning that the amplitude decreases exponentially with depth.

= 500 meters

: conductivity in mho/m

f: frequency in hertz

For example in the case of a VLF signal with a frequency of 20K Hz on a 1 ohm-m clayey overburden p = 3.5 meters or 11.5 feet. p should actually be considered as an upper limit for the probing depth, thus enven with VLF signals exploration may remain very shallow.

D. C. RESISTIVITY

In the case of Direct Current or low frequency AC transmitted by a grounded dipole the depth of investigation increases with the distance between current electrodes and potential electrodes.

Cavities filled with air are evidently very resistive bodies. Fig. 3 shows an example of apparent resistivity map obtained near Amiens in France where artificial and natural cavities exist in chalky limestone. The survey was warred out with a 187 foot current line and three (3) adjacent 10 foot potential dipoles. The middle of the current dipole was a symmetry center for the electrode array. All the drill holes indicated by triangles intersected voids at depths ranging between 10 and 23 feet. The association of a conductive anomaly with a resistive anomaly, as in the case of axes R3 and C2 on lines HY and G, is frequently observed above karstic cavities. The conductive anomaly is probably due to residual clay.

Conditions in the case of Figure 3 were particularly favorable. Another favourable case would be the case where cavities are filled with salt water and thus give rise to conductive anomalies. On the other hand cavities filled with fresh water, sand or rubble may not show any resistivity contrast. This is an important obstacle to the extensive use of resistivity in areas where cavities are deeper than the water table.

Resistivity results may be affected by buried pipes and stray currents.

Another drawback of resistivity methods is the use of grounded dipoles. Light damages may be caused in cultivated areas.

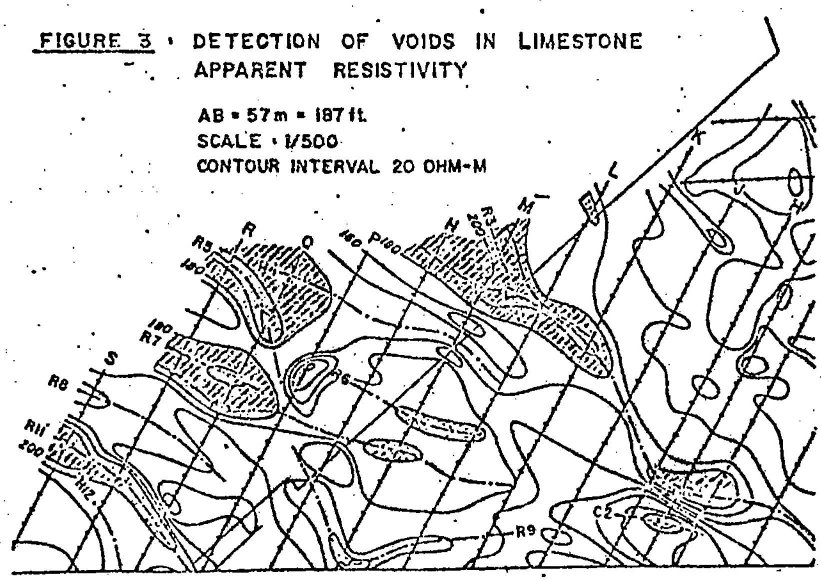


FIGURE 3 · DETECTION OF VOIDS IN LIMESTONE
APPARENT RESISTIVITY

AB = 57m = 187 ft
SCALE : 1/500
CONTOUR INTERVAL 20 OHM-M

Application of Geophysical Methods to the Detection of Shallow Karstic Cavities.

SEISMIC METHODS

As a general rule the velocity of seismic waves is lower in cavities than in the country rock. This is true whether the cavities are filled with air, water or loose sediments. Velocities of longitudinal waves in cavities range between 1100 and 5900 ft/s against 3000 and 9800 ft/s in karstic limestones. Longer travel times are frequently observed over karstic cavities on refraction profiles, although similar anomalies may be due to deepenings of the limestone bedrock. Moreover, because of the relatively small size of the cavities, waves travelling in the country rock around the cavities may overtake the waves travelling across the cavities and mask the travel time anomalies. Even where anomalies are observed the method lacks resolution.

GRAVITY

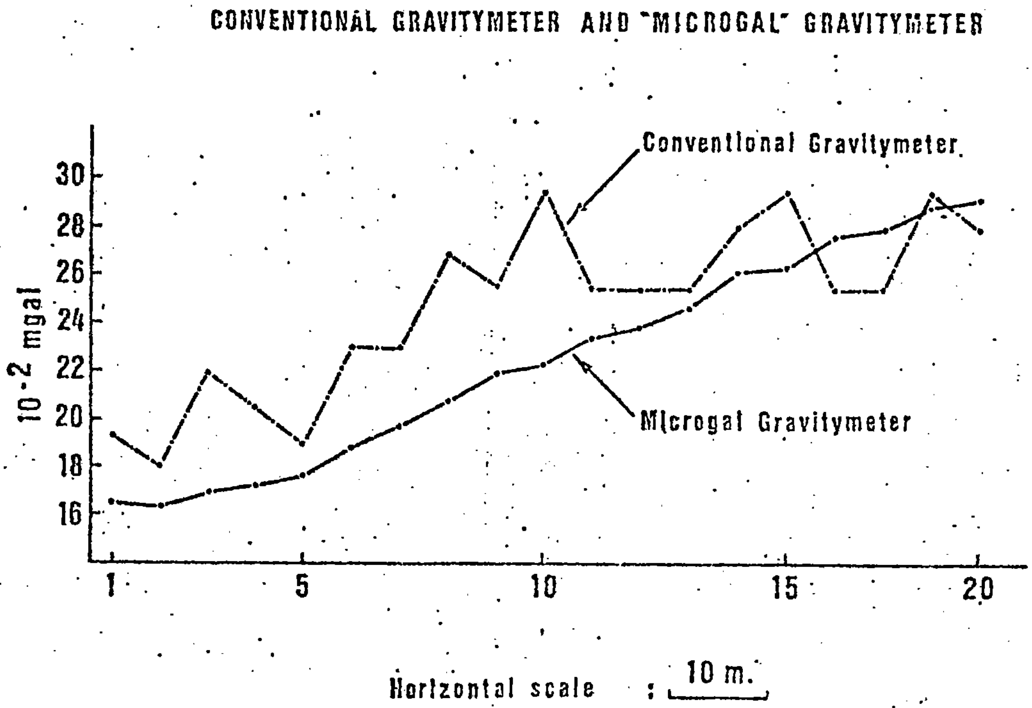
Many underground quarries on the outskirts of European cities are centuries old and their exact location, or even their existence, is unknown. The same remark applies to old coal mines and karstic cavities. The need for a geophysical method capable of detecting these cavities was felt particularly strongly after 1950 when European cities went through a phase of rebuilding and expansion. In the case of CGG (Compagnie Generale de Geophysique) activity in the domain of cavity detection in Europe has been continuous since 1958. Between 1958 and 1963 the main method was D.C. resistivity while research and some tests on the application of seismic were carried out. In 1963 a first gravity survey was carried out on a freeway route near Paris. Between 1963 and 1969 both resistivity and gravity were used though the share of gravity increased continuously. Because the density of stations and the required accuracy is much higher than in the case of ordinary gravity surveys, this particular application came to be named micro-gravity.

Until 1969 good standard gravimeters were used. The necessary accuracy of 0.02 milligal could be obtained only through repetition of measurements. The first "Microgal" Model D gravimeter, built by Lacoste Romberg Inc., was delivered at the end of 1968. Figure 4 illustrates in a striking way the technological leap separating a conventional gravimeter from a "Microgal" gravimeter. Measurements obtained with a "Microgal" gravimeter are affected by errors at most one-fifth of the error affecting the results obtained with a standard gravimeter operated in the same conditions. The "reading accuracy" in the case of the "Microgal" is about 2 microgals, although the instrumental drift is relatively strong than for a standard gravimeter and the "measuring accuracy" is about 5 microgals.

Because the use of the "Microgal" increased the production of microgravity crews drastically, microgravity completely replaced resistivity as a method for the detection of cavities in Western Europe since 1971. The production of a three (3) man crew is generally about 60 stations per working day against about 25 for a crew working with a standard gravimeter.

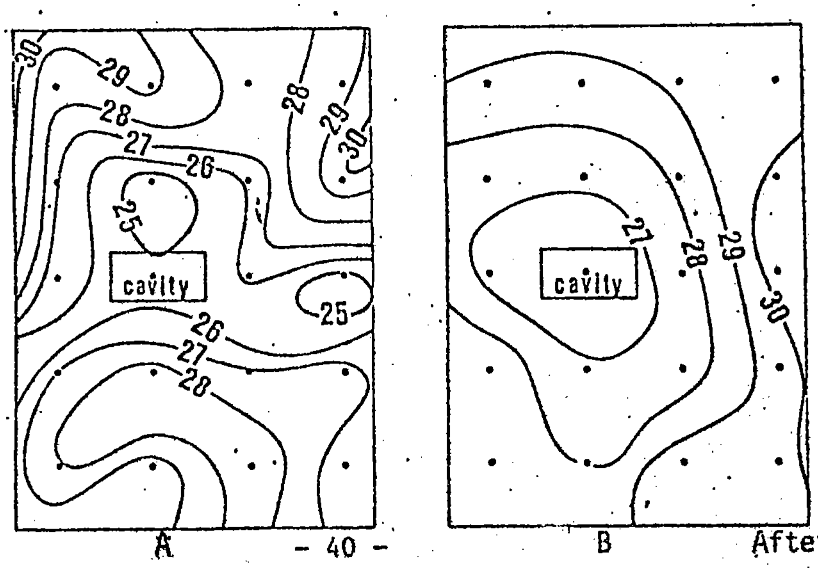
It should be emphasized that the subtitution of a "Microgal" gravimeter to a standard instrument does not automatically provide a much higher accuracy. All procedures should be adapted to the special requirements of microgravity.

Fig. 4



The operator must watch continuously the behavior of the instrument, temperature, light shocks and drift. Figure 5 shows an eloquent comparison between results obtained by an operator who did not know more than "taking readings" (A) and results obtained by an experience microgravity operator. An equal care should be taken during the levelling operations, an accuracy of one (1) centimeter is required.

Fig 5 COMPARISON OF RESULTS OBTAINED BY TWO DIFFERENT OPERATORS – Grid 2x2m



After R. Neumann

The anomaly due to a shallow anomalous body is not much wider than the anomalous body itselt. Therefore the grid should not be larger than the projection of the cavity on the surface. Figure 6 shows the result of a test over a known cavity. In the case of the larger grid the existence of a negative anomaly is apparent, although the exact location of the cavity cannot be pinpointed.

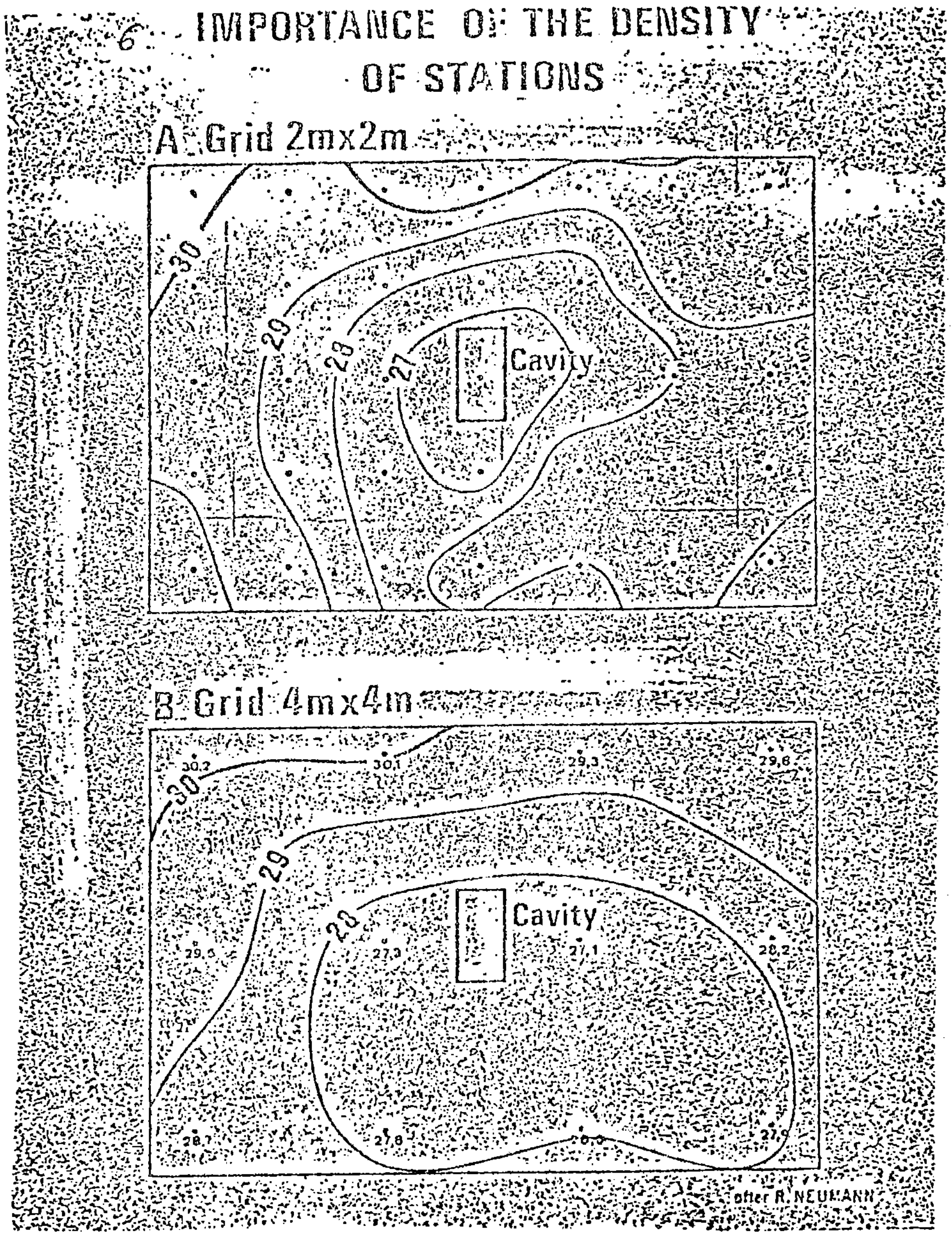
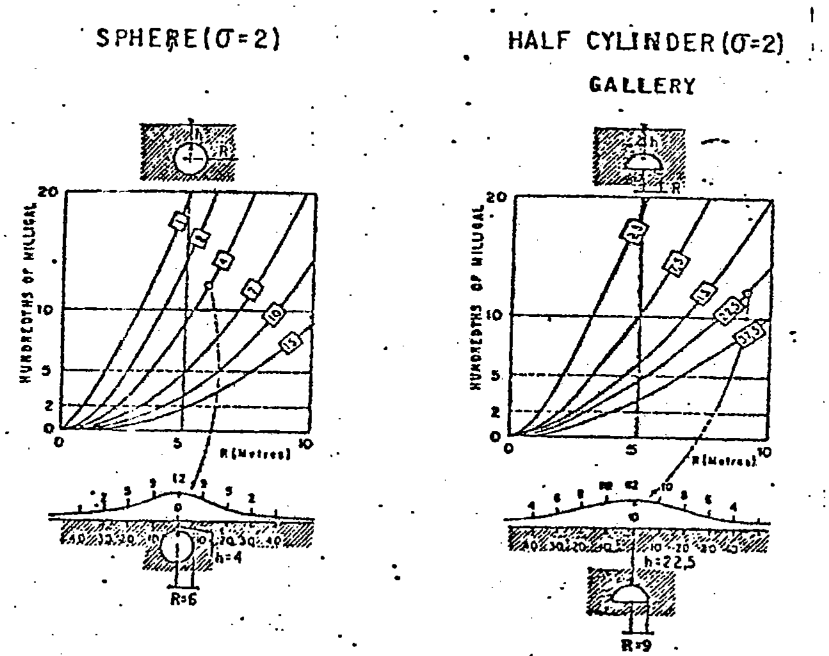


Figure 7 shows examples of theoretical anomalies due to a sphere and a half cylinder. It appears that for equivalent depths the amplitude of the anomaly due to the long half cylinder is more twice the amplitude of the anomaly due to the sphere.

It is interesting to note that the acutal amplitude of an anomaly due to a karstic cavity is generally more than twice the amplitude of the theoretical anomaly calculated on the basis of the size of the void. This phenomenon is explained by the presence of a higher density of dissolution joints within a certain distance from the cavity. Thus anomalous density contrasts exist within a volume much larger and closer from the surface than the cavity itself.

Fig.7

DETECTION OF CAVITIES
THEORETICAL STUDIES
(EXAMPLE)



after R.NEUMANN

Figure 10 is another negative anomaly detected in the same area as the anomaly shown on Figure 9. Drill holes and complementary work including resistivity and refraction seismic showed that the karstic cavity is filled with loose detrital material.

Figure 9

DETECTION OF KARSTIC CAVITIES

Contour Interval 0,02 mgal

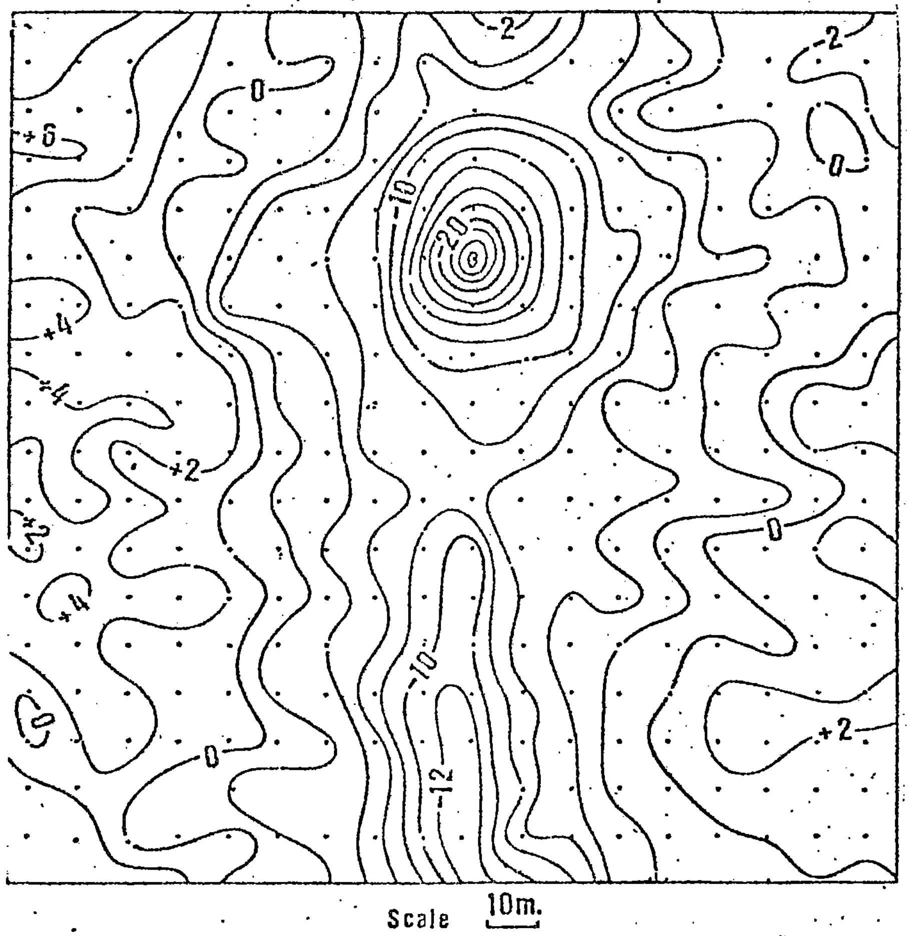
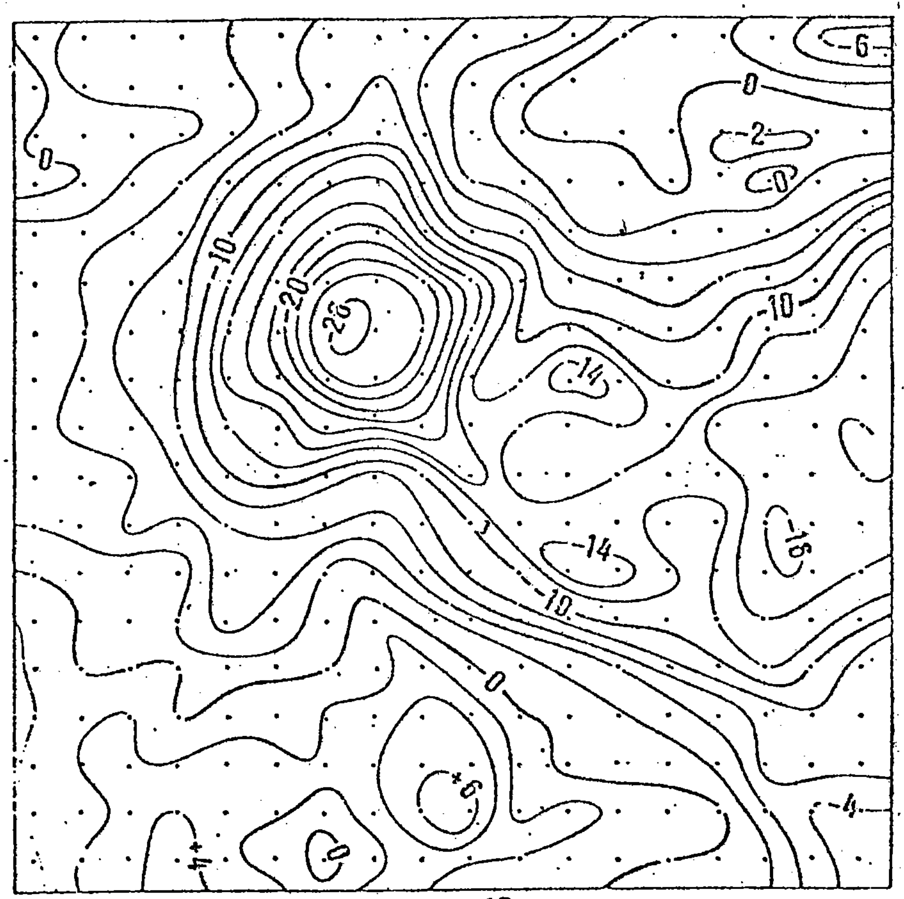


Figure 10

DETECTION OF KARSTIC CAVITIES

Contour Interval 0,02 mgal



Scale 10m.

Figure 11 shows a 30 microgal anomaly detected on a freeway route in Belgium. The small sinkhole appeared after completion of the survey when a light post was being set up.

Figure 11 DETECTION OF CAVITIES (FREEWAY)
Contour interval 0.01 mgal

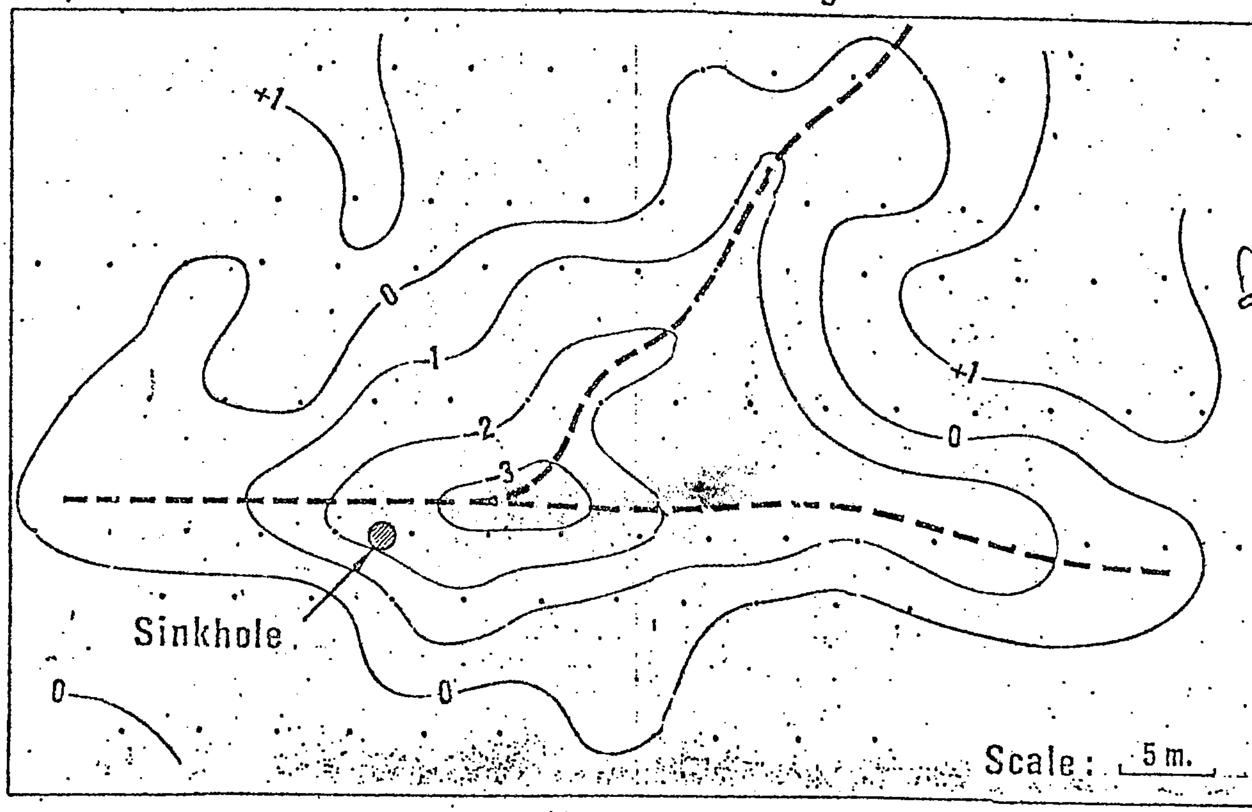


Figure 12 shows a 480 microgal anomaly detected on a freeway route in the Jura in France. It is an interesting example because it was detected while a followup microgravity program was being carried out over a series of small areas where surface visual observations had indicated that the existence of sinkholes was most probable, or even certain. The purpose of the survey was to outline the extension of the strong dissolution phenomena rather than to detect the cavity itself. In the present case the residual anomaly extends over an area of 33,000 square feet, much more than the area of the surface showings. A 82 x 82 ft. grid was used and not more than 36 stations were needed.

FIGURE 12 :

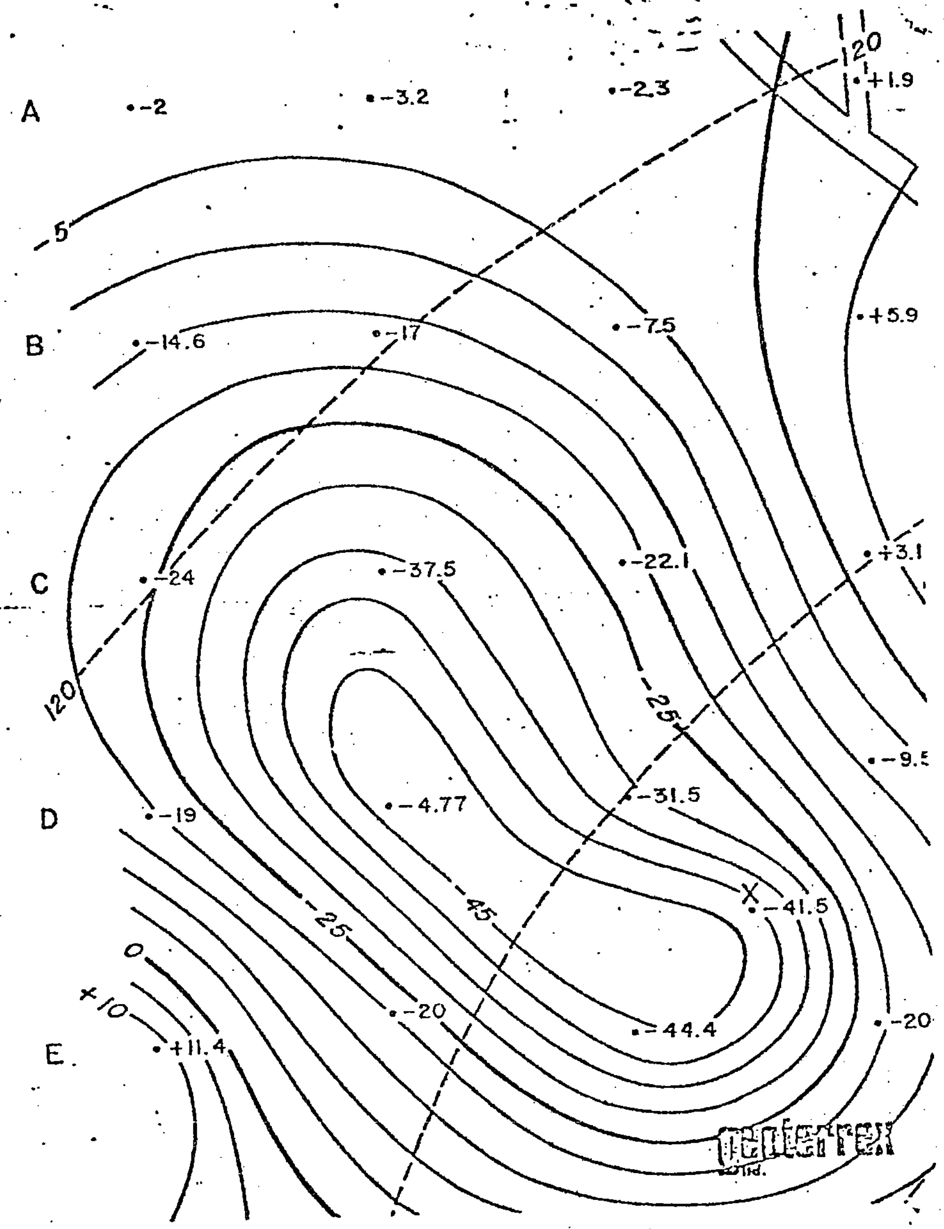

A
•-2
•-3.2
•-2.3
120
•+1.9
-5
B
•-14.6
•-17
•-7.5
•+5.9
C
•-24
•-37.5
•-22.1
•+3.1
120
-25
•-9.5
D
•-19
•-4.77
•-31.5
-25
45
•-41.5
0
+10
E
•+11.4
•-20
•-44.4
•-20

Figure 13 shows results from a survey carried out in Florida. Patterns already shown on previous examples from Western Europe may be recognized. Gravity countours of residual anomalies over karstic cavities are frequently subquandrangular and stronger anomalies are generally located where several narrow negative axes merge.

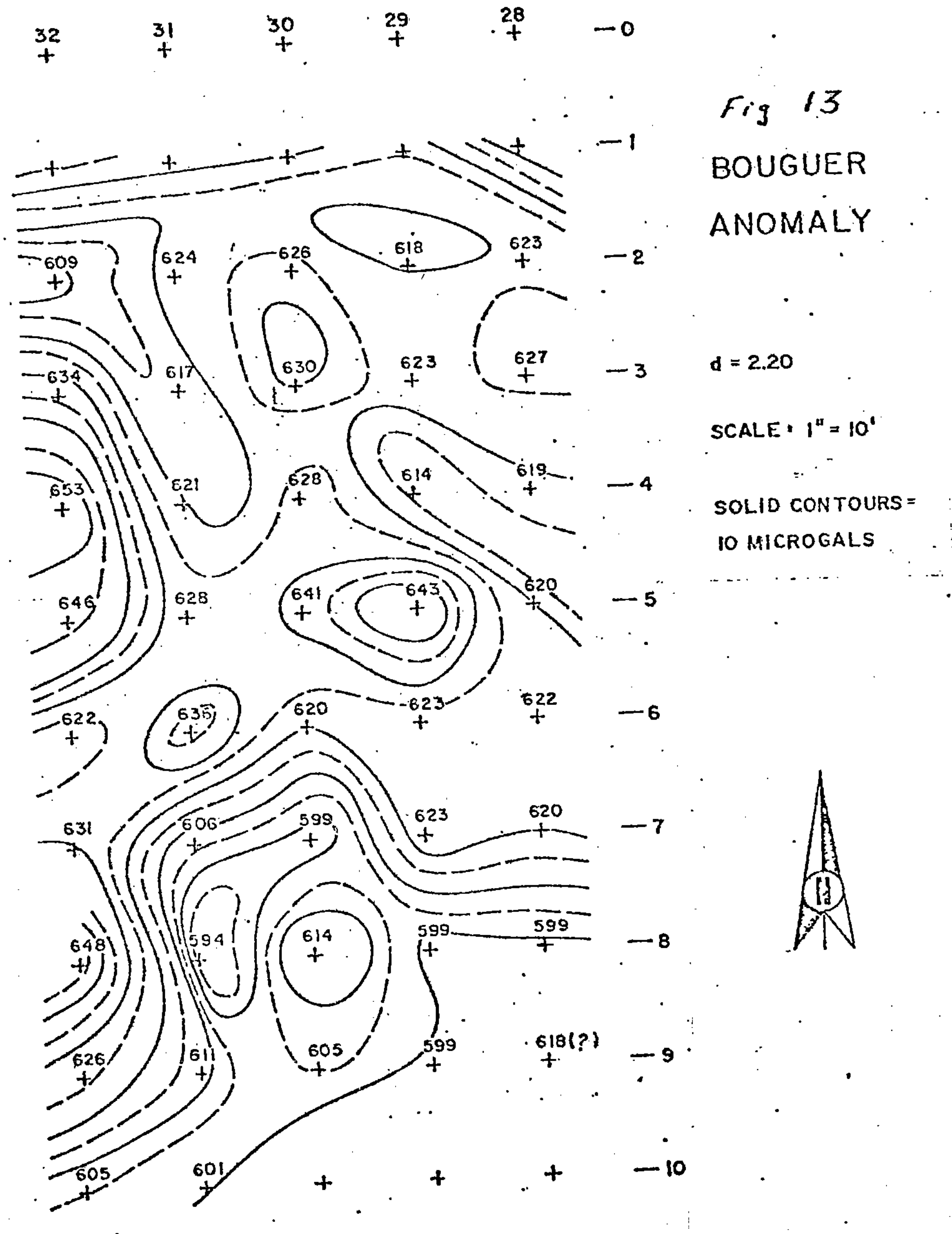


Fig 13
BOUGUER
ANOMALY

In the case of very shallow cavities and homogeneous surface conditions, the measurement of the vertical gradient of the gravity field is an interesting technique because no accurate levelling is required, and the interpretation of results does not necessitate the separation of residual and regional anomalies moreover a better separation of adjacent anomalies is possible. An application is the detection of collapse zones under concrete slabs, highways or landing strips. Readings are taken with a gravimeter at two (2) different elevations above a given station. The difference between the two (2) measurements is divided by the elevation difference. The difference between the two (2) elevations is generally larger than 3 feet and smaller than 7 feet.

The method has a few drawbacks even in the case of very shallow cavities. Measurements are not possible during windy days, surficial heterogeneities affect the results.

Figure 15 shows a comparison of a measured vertical gradient profile and a Bouguer Anomaly profile obtained over a known artificial cavity about 1.7 ft. deep. The elevation difference between the gravity readings at a station ranged from 4 to 5 feet. Readings were taken under, and at the top of a strong tripod.

FIGURE 15 : COMPARISON OF THE BOUGUER ANOMALY AND A MEASURED VERTICAL GRADIENT

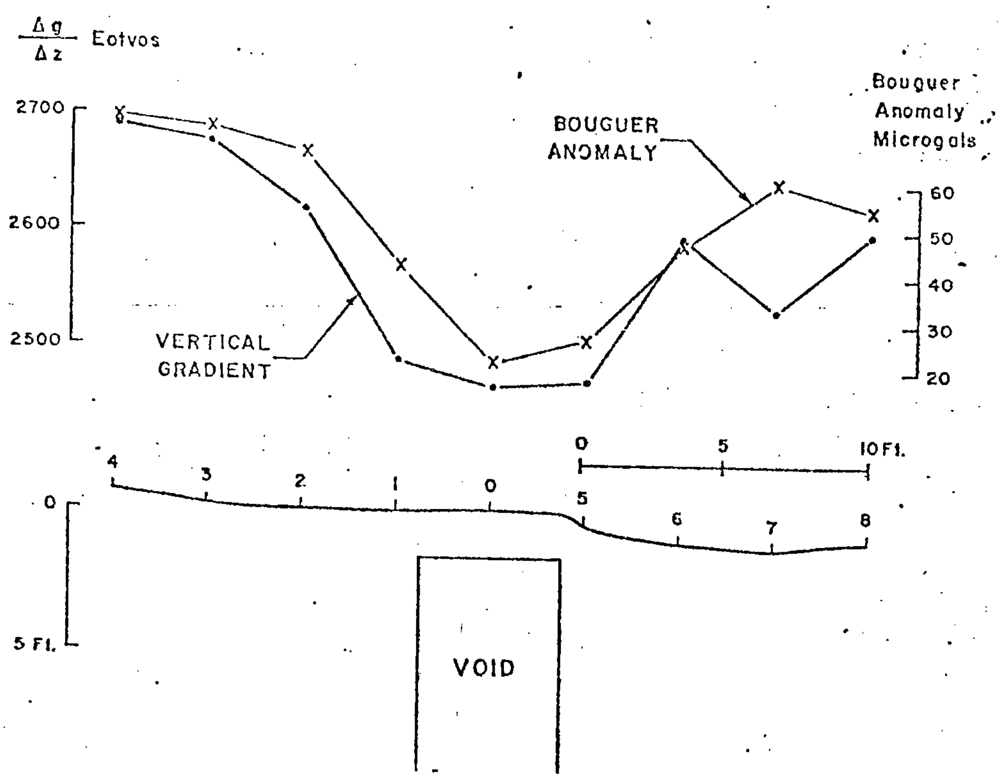


Figure 15 shows that the relative amplitude of the vertical gradient anomaly is slightly larger. The flat bottom of the vertical gradient anomaly coincides almost exactly with the width of the cavity. A secondary anomaly centered on station 7 is most probably due to a surficial cause without effect on the Bouguer Anomaly

CONCLUSIONS

More than eighteen years of experience in the field of cavity detection, particularly the detection of shallow karstic cavities, indicate that microgravity is the most versatile and reliable geophysical method among those tested and used in this field.

The best microgravity survey might not detect some cavities although experience shows that undetected cavities have yet to be proven as representing hazards for construction.

The main advantages of microgravity are the following,

- whether filled with air, fresh water, salt water, mud or rubble, a cavity corresponds to a negative density contrast.
- a gravimeter is a small portable instrument which can be operated almost everywhere without causing damages, or roads, on building sites, on landing strips, in gardens, even in basements.
- gravity measurements are not affected by buried metal pipes, or stray currents as resistivity measurements are.
- the depth of investigation of microgravity is not reduced by moisture and conductive soils as it is the case for radar probes.

It may be argued that the relatively large number of stations needed for a thorough exploration makes the method expensive. This remark underlines the interest of a selective use of microgravity where geological studies and remote sensing results indicate that the existence of shallow karstic cavities is probable.

ACKNOWLEDGEMENTS

I wish to thank Robert Neumann for all the documents taken from his previous papers and presented here.

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LAND AND MARINE APPLICATIONS AND ECONOMICS OF ENGINEERING GEOPHYSICS

By

Richard C. Benson, President

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ABSTRACT

Geophysical techniques have been utilized for "small sale" problems over the years. Siginificant advances in electronics and their application to these small scale problems have brought about a new breed of sophisticated "suitcase" instrumentation. Many areas of application have been neglected due to bad experiences with older equipment and lack of familiarization with the state of the art.

The various common methods of engineering geophysics are briefly introduced. A few case histories are given and include: cavern detection, economic resources, and marine applications. Examples are evaluated both technically and economically by comparing the results of techniques used vs. possible options available.

Introduction

This paper is presented as a brief introduction to some basic engineering geophysical methods used on land and water. Some advantages, disadvantages, examples of their application, and economics are discussed.

Engineering Geophysics is commonly thought of as concerning itself with the upper few 100 feet of subsurface or subbottom materials. More recent interest in seismic design and fault/fracture aspects has extended the depths of interests in these cases.

The methods which we will be discussing have originated within the fields of geophysical oil/mineral exploration and deep-sea oceanography.

Only within the past decade have the marine methods been applied with any regularity to engineering problems. The land methods have been used for a few decades. The techniques and equipment have been vastly improved and adapted to shallow works where most engineering takes place.

Note: Numerous photos are not included within the figures attached to this paper.

A most significant improvement has taken place in the electronics via semi-conductors, integrated circuits, and digital techniqes.

An example is the enhancement now utilized on portable seismic systems. Here the analog signal is sampled and bits stored in memory. Repeating source impulses adds to the memory increasing the coherent signal to noise ratio. This step has provided a significant advancement in engineering seismics.

A very recent product provides for digitalization of marine subbottom and/or side scan signals and playback on a linear sweep recorder. These digital advancements also add to reduced power and size along with increased ruggedness and reliability.

The geophysical methods are <u>indirect</u> sampling techniques and while they can grossly reduce the amount of direct sampling required, they are not intended to totally replace direct sampling which provids the necessary ground truth. However, once the gound truth is provided one can usually continue to expand survey lines with a high level of confidence.

Figure 1 A and B illustrate how discrete point data can easily miss critical features. Figure 2 C compares the "Voulume of Sampling" for drilling and geophysical methods. Assuming some Diameter "D" of lateral homogeniety for the drill hole, we can likewise assume a similar width "D" to the seismic cross section. Since seismic lines are extended to four times the depth "d" of interest we have the following relations:

$$\text{Drill Volume} = \Pi \left(\frac{D}{2}\right)^2 \times d$$

$$\text{Seismic Volume} = 4d \times d \times D$$

$$\text{Using } D = 5'$$

The ratio of $\frac{\text{Seismic Volume}}{\text{Drill Volume}}$ is approximately equal

to the (depth) d.

To achieve a ratio of 1 at depths of 20, 50, and 100 feet, D (Driling, Diameter Homogeneity Assumption) must be approximately 100', 250', and 500' respectively.

This assumption might be perfectly okay in layer cake geology conditions, but why gamble when it will cost less to utilize a more powerful bulk sampling method.

Figure 3 shows the same situation in the marine environment--here the trade offs are even more significant.

Not only are we bulk sampling, but we typically are sampling "continuously" vs. discrete, and we have the capability for on-line analysis--all for equal or less costs.

Figure 4 shows some features that can be determined utilizing some of the geophysical methods.

Project Requirements

Once a requirement for subsurface data is established, numerous factors must be considered to produce reasonable optimum results. Some group of persons or possibly a single person must consider all of the parameters shown in Figure 5 to formulate an operational requirement. Field operations also require inputs and outstanding coordination of all members. The operations manager must understand all interacting functions as well as fully understanding the original project requirement.

We prefer, to have a degree of flexibility in such operations. By performing on-line, or at least daily first order data reductions/quality control, and having some flexibility, the operations manager can best optimize the overall program results. Optimum data reduction also requires knowledge from all of the various skills and people involved.

There are numerous technical factors which go into choosing the best equipment to do a job. Likewise, in planning requirements, executing the field operations, and data reduction, many aspects must be considered.

Larger and complicated projects may require an extensive team effort whereas small projects might be completely handled by a single person. No matter which, all factors must be considered to produce reasonably optimum results.

Methodology

Classic on-land engineering geophysical methods include seismic refraction and reflection, resistivity and gravity. A host of specialized methods and technology is available such as shear wave measurement, hole to hole, well logging, etc.

We restrict our discussion to the former. These methods have been utilized on land for a few decades and have typically undergone considerable improvement in instrumentation and methodology.

The seismic refraction method is shown in Figure 6.

The method involves time measurement of the first seismic arrivals as the distance between the geophone and hammer are increased.

The method is based upon wavefront refraction at the boundary of layers with increasing wave velocities in subsequent layers. The resulting plot of this data yields considerable subsurface information.

The resistivity method is based upon an electrical resistivity contrast in subsurface layers. Its influence is detected on the surface by an array of electrodes (many configurations can be used). Typically the outer electrodes supply a current and the inner electrodes sense the surface potential distribution.

Variation in subsurface materials cause a redistribution of the surface potential enabling indirect measurement of the structure. A simplified diagram is shown in Figure 7.

Two general methods of application are available:

Sounding consists of expanding the electrode array about a fixed center point. As the array is expanded the influence of materials at depth is increased and sensed at the surface.

Profiling consists of a fixed electrode spacing array which is moved along the survey line as shown in Figure 8.

Figure 9 shows typical refraction and resistivity plots for a given two layer subsurface condition.

Figure 10 shows typical data due to "bed rock" irregularaties. Some typical values of seismic velocity and resistivity for Florida are shown in Figure 11.

The basic Seismic Reflection technique is illustrated in Figure 12. Note the enhancement of reflecting horizon. R_2 and R_3 in subsequent traces.

R_1 is obscured due to large surface waves. This system can be as simple as a single channel system (as shown), or a sophisticated multi-channel system whose output is digitally processed and shown in Figure 13.

The gravity method consists of assuming stations along a profile line or more typically in two dimensional grid. Station spacing is related to the anomaly size expected. Data corrections are required and as a minimum, good elevation control is a must. Fig. 14 illustrates some of the basic ideas involved. Two methods of plotting data are shown in Figure 15.

The marine geophysical methods include precision bathemetry, subbottom profiling, side scan sonar, marine refraction, magnetometry, marine resistivity, and gravity. We will consider the first five methods.

In some cases (primarily the acoustic methods) the marine methods amount to continuous real time visual displays which makes their use very advantageous.

Marine Echo Sounding

Figure 16 illustrates the basic principle of reflecting acoustic signals from the bottom. Here we measure the travel time interval (divided by two) which yields a depth (assuming we know the acoustic velocity within the water column). Typically a mean water velocity is assumed (4800'/sec or 1500m/sec) and the resulting record is interpreted directly.

Sound speed corrections may be applied within the recorder itself if it has such accommodations. Or, they may be applied to the interpretation of the final record.

Figure 17 shows an Echo Sounding Record made on an oceanographic (PDR Precision Depth Recorder). These systems are designed for operation to the ocean depths. This particular record was made departing Government Cut. Note the multiple reflection occurring on the upper richt hand side of the record. Another PDR record, Figure 18, offshore Miami, Florida, illustrates the occurrence of multiples and their use in interpretation. Note that on the original and multiple trace the ridge occurs more intensified indicating a better reflector than surrounding areas. This would indicate a rock outcropping on the ridge. Considerable information can be obtained from a simple but well executed echo sounding record (scale lines are 20 fathoms).

Individual fish can be detected and sometimes bottom materials can be assessed as shown in the previous record (based upon reflectivity). Limited structure can be inferred based upon specific record characteristics such as the occurrence of Hyperbola, geometry, and reflectivity.

Marine Reflection Subbottom Profiling

Figure 19 illustrates the basic idea of acoustic signals reflecting from subbottom interfaces as well as the bottom. By utilizing higher power and lower frequencies, than in echo sounding, acoustic energy penetrates and reflects from subbottom interfaces. As with echo sounding, a recorder is designed to run at a speed which corresponds to standard water column velocity. Hence the actual record is in fact based upon an average water column velocity. In many cases this is quite adequate and records can be read directly without significant error. (One assumes the subbottom materials to have the same sound speed as the water). Where rock velocities are encountered they may range up to four times that of water and corrections may be required forquantitative results.

Figure 20 shows a typical shallow penetration subbottom record.

Marine Side Scan Sonar

Figure 21 illustrates the basic principle of side scan sonar. Here a "fish" (containing the transmitter and receiver elements) is towed from the surface vessel. It produces a fan shaped acoustic pattern which covers an area either side of the ship. Any reflectors such as the ridge on the lower left bottom wil reflect back increased energy and show as a dark spot on the record with a light spot (acoustic shadow) behind it. Hence we have a means of producing an "oblique acoustic photographic" presentation of the bottom.

This is a rapid means of providing coverage of a large area. Small details such as a 1/2" cable or small sand waves have been observed. The technique is frequently used in location of downed aircraft, ships, etc. It provides extensive information in surveys related to environmental and bilogical work as well as engineering geology site evaluations. Figure 22 is a typical side scan record of a reef/sand site along Florida coasts.

Marine Refraction

Figure 23 illustrates the basic idea of a refraction survey technique. Here the sound source emits its energy which refracts through the interface, then travels along the horizontal interface to be refracted some distance away to the receiving hydrophone.

With this technique, arrival times of "first arrivals" are noted, just as in the case of land refraction work. The method is slower, and provides less detail than reflection. It also suffers from the hidden layer problem (thin layers) and lack of detecting velocity inverted layers.

It does, however, yield direct velocity of the materail which can be utilized to correct reflection work as well as "Identify" materials. Work can be conducted in very shallow water, swamps, etc., as well as interface zones such as beaches where the contiuous reflection methods presently fail.

A second diagram, Figure 24, will provide a better understanding of the refraction method. In this case six (6) hydrophones are used. Note that the resulting Time vs. Distance Plot is drawn here to the actual distances of the hydrophone array.

Note that the same data can be acquired by a single hydrophone by changing its position. Engineering refraction/reflection systems vary from single channel to 24 (or more) channel systems. Modern systems commonly utilize signal enhancement to improve performance and data confidence.

Magnetometry

Magnetometers respond to magnetic anomalies within the earth's magnetic field. These anomalies may be caused by natural or manmade concentrations of "iron deposits." The method is commonly utilized in Florida as a means of conducting an archeological survey required by the state previous to any offshore dredge or construction. It is commonly used to locate ferrous pipes, cables and well heads. A magnetic record produced by an old anchor is shown in Figure 25 (850 lbs. early 1900's).

Field Examples

The following will illustrate some typical applications of general interest. They will include typical land and marine case histories upon which production rates and costs have been well established. In a number of cases we have been fortunate to have cost and production figures from a variety of jobs on which both classic "direct sampling" as well as the geophysical methods were used.

The examples cited include:

Land   Exploration for limerock road fill.

Cavern detection.

Land and Marine Applications and Economics of Engineering Geophysics.

<u>Marine</u> Pipeline/cable routing

Pipeline magnetic surveys

Environmental work on Lake Okeechobee

Offshore assessment of sand deposits.

A. Exploration for Limerock Road Fill

A residential development (Marion County) required considerable limerock for road building. The nearest known source was a number of miles away. In using this remote source, the developer was facing the increased cost of trucking, but he also had to pay for the limerock.

A reconnaissance seismic refraction survey was conducted on the property in hopes of finding a shallow (less than 20') source of limerock. This project was a natural for seismic refaction as the water table was generally greater than 20' and a distinct contrast existed in velocity between overlaying sands and the limestone.

We were initially instructed to examine a large hill which was "obviously" bedrock controlled. The first seismic line provided the data shown in Figure 26. The 2200 F.P.S. Velocity was too slow for limestone, but the 4,900 F.P.S. Velocity was within the range of local weathered limestone.

To become familiar with the area a resistivity sounding was made and provided the data shown in Figure 27. It was interpreted as a four (4) layer case using master curves.

Interpretation of the resistivity data alone would have been a bit difficult as one would at first glance see a three (3) layer case. The seismic data however revealed a two (2) layer situation in the upper 20' and saved considerable time and improved occuracy of the resistivity interpretation.

This same site was subsequently drilled as the client could not believe bedrock to be at 60'. He very much wanted it at 10-20'. The auger record is shown on Figure 28, along with the seismic cross section and resistivity cross section.

This is an excellent example of two (2) instruments providing back up for one another.

A few selected sites were drilled on and around the hill and it was quickly abandoned as a source of limestone.

The production rate of an auger system on a good day (using spin up) is approximately 400' penetration/day for 20-30' holes. This results in about 16 holes per day at a cost of $400.00 to $800.00 per day.

From subsequent tables we find production rates for refraction of (16-20') 100' lines/day. While this is comparable with the auger production rates and would cost approximately the same, it provides data over the length of the line about 100'. Further, the auger rig was restricted to work near roads or in clearings.

Another area was selected within a slightly different geologic setting. Again a ridge with much less relief. Here we found bedrock at economic depth.

Subsequently a "short" refraction line was used to map the rock interface over the area of interest.

The technique consisted of selection of two (2) stations before the break and about three (3) stations after. We could do this because from station to station we knew roughly where the limestone should lie. A two (2) man team can operate with equipment set up on the back of the field vehicle. One man drives to the next site, the other rides by the equipment. When the vehicle stops the hammer man runs out to the first station, then 2nd, 3rd, 4th and 5th. He then picks up while the instrument man plots the data and determines the limestone depth. They then move to the next station (approximately 5 minutes per station). A high density survey with 50' station spacing would yield rates of 600'/hour. At 100' station spacing we get 1200'/hour etc. All with bulk sampling.

Figure 29 shows the short-line refraction data and the profile mapping of the limerock.

Figure 30 shows a two layer resistivity curve taken in the immediate area.

B. Cavern Detection

The cavern collapse problem has been one of the most challenging field problems to date. This example will illustrate some of the geological background and field techniques we have used successfully.

COLLAPSE MECHANISMS

Two types of collapse mechanisms seem to exist. We are referring to a sizeable hole developing at the surface as apposed to a gentle depression (which may also have very damaging effect on structures).

Land and Marine Applications and Economics of Engineering Geophysics.

Roof Collapse

Figure 31 illustrates such a roof collapse on Grand Bahama Island.

The slow natural spalling of the limestone roof, plus the increased load due to loss of buoyancy, and the annual drying of the roof where applicable, add to take their toll in collapse of the limestone roof.

Solution Pipes

Figure 32 illustrates a typical solution pipe collapse at Guest Sink, Hernando County, Florida. Three (3) vehicles of total value in excess of $100,000 were lost on this site in September 1974.

Figure 33 shows one of numerous satellite sink holes near the main sink.

Figure 34 shows a proposed cross section of this site.

Vertical pipes or chimneys develop in the limestone between sizeable cavities and the surface of the rock. These pipes form a channel for the transfer of the over-burden. Again the annual change in water table induces structure stress and buoyancy loads while percolation of ground water carries away material.

Eventually structural failure of the over-burden occurs and the slumping material is carried through the pipe and into a larger cavity. Note that by this mechanism the limestone roof itself does not necessarily "collapse".

Most observers are of the opinion that the solution pipe or chimney mechanism is predominate in recent Florida sinks.

A few other examples of collapse are shown.

Figure 35 1st observation, Marion County, about 6' diameter.

Figure 36 8 hours later, Marion County, about 30' diameter.

Figure 37 Hillsborough County. Here the major cavity is greater than 100' in diameter.

Figure 38 Cross section, Hillsborough County. Note the intersection of the three (3) horizontal tunnels above the main cavity.

Cave divers have provided first-hand information from observation mapping and photos. This information is useful when working in a given area as it provides typical geometry, size, depths, etc. which are of importance in planning field work.

Figure 39 Represents a cross section of a typical open Florida sink.

Figure 40 Largest known "Blue Hole", British Honduras, shows typical round character of sink.

Figure 41 Breakdown. Note sizeable block fallen roof.

Figure 42 Solution of Bedding Plane to lower left.

Figure 43 Bedding in sizeable tunnel.

Figure 44 Enlargement of vertical fracture.

Figure 45 Vertical Solution Pipe.

(Cave Diving photos by Rick Freese and Tom Mount)

Figure 46 Local linear sink patterns are revealed upon conventional aerial photos. These patterns, if well established, can become an important part of field planning. They may indeed display regional trends which may be verified on the ground.

The Engineering Problem

The site requirement may be for a major structure, road, industrial site, airport, or simply residential housing.

Traditional direct sampling methods (drilling) have been commonly applied to site assessment along with aerial photos. Obviously an extensive drilling and/or geophysical program for cavern detection can be very expensive. Hence, only in unusual cases is a cavern/sink assessment made in site development and planning stages. Generally, this end of the project is omitted for a variety of reasons, and only when a problem occurs, does it receive attention.

An economical and technically viable approach is to incorporate an integrated program of site evaluation, using indirect geophysical methods, backed up by a less extensive but improved direct sampling drilling program. Here, one could obtain the usual information necessary for site development along with first order cavern/sink assessment. It has been our experience that the cost of such an integrated site assessment is always equal or less than other approaches on a piece meal basis, at the same time providing a significant increase in data.

Some requirements deal with relatively small and near surface structure such as Honeycomb Limestone, others with depths of 200 to 300' and proportionately larger structure.

One of the first steps in arriving at a field program is to define the limits and scope of the work both technologically and economically.

From both technological and economical points of view we are faced with resolving the physical size and lateral distribution of the unknown structure. We have used Figure 46 as a guide to planning field activities in terms of depth of burial vs. cavity size related to probability of detection.

THE FIELD WORK

Phase I - Site Familiarization and Calibration

Geologic knowledge is essential along with local information concerning the site. Calibration is carried out on local sites selected to be "free of anomalies". This calibration provides the basic Seismic Velocities (Vp) and Resistivities (Pa) related to the site.

Phase II - General Reconnaissance

A Reconnaissance plan is based upon expected anomaly size and depth, desired probability of detection, and economics. Generally, selected sites will be sampled, using resistivity sounding, seismic refraction, and gravity lines. Large areas will be examined using multiple spacing resistivity profiling. These selected reconnaissance data allow a larger area to be tied into the calibration sites and to note general trends in geology.

Phase III - Detailed Examination of Anomalies

To examine the potential cavern anomalies a number of detailed seismic, resistivity and gravity techniques as well as borings may be employed. These detailed techniques will depend upon many factors including resolution, local geology, economics, and the developed (buildings and other structures) state of the site.

We have found that careful application of some of the classical engineering geophysical methods, along with state of the art equipment, has produced technological as well as economic results. We rely heavily upon various resistivity methods as a means of economic reconnaissance and detailed location. Seismic techniques usually provide supplemental structural information rather than a means of direct detection. However, larger structures have been observed seismically. Gravity techniques are used when technically feasible. They provide another means of confirmation. Adaptation of a singular field technique and equipment does not normally yield effective results. An integrated multiple technique approach is usually essential to success.

Economics

The geophysical methods offer a considerable savings in cost and time, along with significant increase in data.

Figure 47 shows cost per acre as a function of cavern size.

The assumptions are that cavern diameter equals depth of burial to top of cavern, drill costs and geophysical costs and geophysical costs are both $4.00/foot, and 100% coverage is assumed for either method.

For geophysical techniques cost advantages of 2 to 10 prevail along with an increased volume of sampling of 2 to 20%. This, plus the additional homogeneity of sampling and geometrical features obtained, make these techniques an excellent approach to the problem.

Now for two more examples of comparison:

Example 1

First consider a small plot of land--say a one acre site. To obtain subsurface information over this site, let's consider two approaches and tabulate results.

Auger Sampling. To provide some degree of reasonable coverage, let's drill five holes. Let's also make the assumption that for a distance of say 5' radius around the hole, we have absolute homogeneity. This will give us a sampling volume to work with. Let's also consider the options of being required to have information to 10', 20', 40' and 80' depths.

The geophysical lines and layout were chosen to give approximately the same distribution as the 5 hole Auger array.

Geophysical Sampling. Seismic lines with a length 4 times depth of interest are used. (We also assume a + 5' lateral homogeneity). The results are tabulated below: (Figure 48).

Depth of Interest (feet)	Total Site Volume (cu.yds.)	% Volume from Auger	Total Drilling (feet)	Drilling Costs @$2.'	% Volume from Seismic	Total Line Ft.	Seismic Costs @$2.'
10	16,000	0.9	50	$100	4.6	200	$100
20	32,000	0.9	100	$200	7.4	320	$160
40	65,000	0.9	200	$400	7.4	320	$160
80	129,000	0.9	400	$800	14.7	640	$320

CONCLUSION

Percentage of Volume Sampled. At a sampling depth of 10' the geophysical leads by a factor of 5. At greater depths the geophysical method for out performs the auger techniques (factor of 16) and can continue to work to depths of a few hundred feet or more.

Cost. At shallow depths, say less than 20', the two techniques converge. For deeper investigations, the geophysical is cheaper by a significant margin (factor of2.5).

Example 2

For a second example, consider the comparison of drilling vs. dual profiling resistivity for coal seam/ground water evaluation.

Figure 49. Tabulated results for various sampling station distances.

The economic advantages of the geophysical techniques are quite obvious. Even so one would require some direct sampling for original ground truth and quality control. It becomes obvious that one could afford the geophysical program at any level of density and still include 1,000, 480 or 200 drill holes respectively and not exceed the cost of a drilling program alone. This is an excellent example of combining technologies to increase data level and confidence at equal or less cost.

The following are some examples of shallow water work within Florida:

1. A pipeline/cable routing survey is a typical application (Figure 50). Here depth to bedrock was required to locate a site which would require minimal removal of consolidated material. The record reveals a channeling in the limestone cap rock which is covered by clastic sediments.

   Figure 51 Shows a post installation survey at this site. This is a combination record incorporating both subbottom and side scan.

11. Approximately 2 man weeks were expended by divers attempting location of a valuable length of dredge pipe (underwater visibility was zero).

   Figure 52. Shows the magnetic anomaly on the first pass.

Subsequent runs and buoys were used to locate the center of the pipe. Center location was accomplished in less than one hour on site and the pipe was recovered immediately.

III. Pollution/Environmental Assessment of organic deposits or sediments within canals and marina basins can be aided by shallow water profiling. A first order survey was run at a Miami marina to determine bottom sedimentation thickness and distribution (Figure 53). Similarly the technique is also applicable to assess shifting bars, or silting canals. Assessment of bottom deposits will also enable improved future designs with improved flow/flushing characteristics and identify sources of sediments.

IV. A predredge survey was run through a local marina to assess depth to bedrock.

Figure 54 Subbottom record is shown.

V. Environmental work being conducted on Lake Okeechobee required assessment of relatively thin mud and peat deposits. Their distribution and total volume was required along with resolution to 6". This lake is relatively shallow, 14-15' maximum depth. Approximately 90 miles of preselected line was run to delineate the muds. An additional 30 miles of line was used to fill in gaps in the data for a total of 120 nautical miles of line. Both bathemetry and subbottom data were taken. Figure 55 illustrates the muds recorded as a semi-transparent on the subbottom record.

A total of 82 coring stations were used to provide ground truth. These samples were taken with a hand-held and driven piston core. The samples themselves were not retained, but simply used to determine accurate thickness and material content. This system of obtaining "bottom truth" proved most effective. While providing excellent quantitative control, each core station was typically completed in less than one minute (Figure 56).

These data were compiled to produce a 1st order approximation contour map of the mud and peat deposits within the area surveyed. Figure 57 shows the overall sediment type and distribution. Figure 58 shows the contouring of major mud deposit. This contour map then provided a means of approximating the total volume of bottom deposits within the surveyed area.

The entire operation was conducted from a 21' runabout with only two (2) persons required. This illustrates the lower operation costs and portability of the methods. Figures 59, 60 and 61 show a typical small boat installation.

Subsequent ERTS photos, Figure 62, utilized to study lake nutrients also reveal the outline of the charted muds which have been churned up by high winds. The total project costs ran about $65.00/nautical mile.

VI. An offshore sand deposit site required extensive investigation. A typical subbottom profile reveals ridges and troughs with clastic sediments trapped within the troughs (Figure 63). These are excellent sources for mining sand or for beach replenishment. The vertical scale is 25'/division.

The next two Figures illustrate the variability that can be encountered. The two records were run just 100' apart (Figures 64 and 65). Note the submerged ridge developing, detectable in the first Figure and just beneath the surface on the second Figure.

A few miles away the outer ridge which is shown in Figure 66 disappears completely (Figure 67) leaving no major sand trap.

VII. Side scan sonar is also very useful in assessing marine sites. The first Figure 68 shows a continuous ridge to the left, and a vast area of "sand" deposits to the center and right. We can correlate this photo with subbottom records. The ridge shown in Figure 69 is the same one shown in the side scan record. The area covered represents approximately 25 acres. Note how one also obtains a depth indication on the subbottom record. The distance between transmitted pulse center and the first bottom return either side (the depth over the ridge is less than over the sand).

Dredging operations are another very useful application for this tool, both in site evaluation and post dredge inspection (Figure 70). Note the continuity of the reef to the left, then a patch of sand, a dredged area (note arcing sweep marks left by the cutter), then another patch of sand to the right, and another reef.

Economics

Usually there is much more to economics than dollars. Data quality and overall effective results are what really must be measured per unit cost. In one example of offshore site assessment we found a cost savings factor of more than 2 to 1 in direct costs. At the same time providing three (3) times the sampling line length and at least a factor of ten (10) in data confidence and density.

The following diagrams show some typical production rates for specific engineering geophysical methods.

This data was developed from actual infield experience and fixed contract prices. It is based upon a 10 hour field day and a typical field team of three (3) people. It further assumes accessibility to the site. The data is typically shown as days per line mile for a given function and geometry.

Figure 78 Refraction

Figure 79 Reflection

Figure 80 Resistivity Sounding

Figure 81 Resistivity Profiling

Figure 82 Gravity

Figure 83 Marine Surveys.

This data should be considered as a general guide only as each field problem may vary due to its uniqueness.

The basis of the figures can be thought of as having enough detail to be used for cavern detection and coarse enough to be used for "fine reconnaissance."

Customers generally like to think in terms of cost per foot because they are used to direct sampling (drilling, etc.).

Figure 84 Provides a quick cost comparison on this one to one basis. What does not show up here of course is the number of other advantages provided by the geophysical methods.

General Comments on Techniques and Application.

There is no singular technique or methodology which will provide answers to all requirements.

First order problems (seismic refraction depth to bedrock in layer cake geology) are readily handled and the entire process can be taught to inexperienced persons in a matter of a few hours. On the other hand, as the problem becomes more complex, a broader technological approach may be required along with considerable experience.

All too often a person new to the field will expect to solve a more complex problem and be disappointed by "strange results." Subsequently, the equipment is placed in the closet to rest and conclusions drawn to the effect that that method isn't applicable or doesn't work under these conditions. Therefore, many newcomers to the methodology walk away in dismay stating that the method is not reliable, inappropriate, etc.

Site assessment was required over approximately one-half square nautical mile (Figure 71). Washborings were taken over 20,000 feet of line @130' spacing with a density of approximately 2.7 acres/boring.

Further definition was ultimately required and a seismic reflection survey was run. This survey covered 63,000 feet of line with continuous sampling. This coverage is equivalent (when considering beam width) to actually sampling 5 to 10% of the area involved.

To yield equivalent coverage from washborings, would require an assumption of approximately 100' diameter lateral homogeneity. Under layer cake geological conditions such assumptions are reasonably safe. However, under variable geologic conditions this assumption can be catastrophic. At the same time it's an assumption that doesn't have to be made technologically or economically.

Another positive aspect is that additional functions can be added to such surveys for relatively small additional costs. They might include:

Bathemetry
Side Scan Sonar
Magnetics
Electronics Positioning
On Line Data Reduction

This mode of running simultaneous data acquisition is extremely powerful in site assessment.

After such surveys have been made it is relatively easy to place a few well selected core or drill holes so as to optimize their sampling. Here then, the direct sampling is essentially 100% effective and is not being wasted as a reconnaissance tool.

For just a few holes (cores to 20-40' or drilled holes to 50') in shallow water this problem can be handled by recently developed portable systems. This alleviates the expense of a large vessel/barge with cranes and drill rigs.

The geophysical techniques and the shallow direct sampling methods mentioned possess another major economic advantage. This is their portability which allows easier access to difficult sites and reduces shipping problems to remote sites.

Figure 72 Gravity/Piston coring from a wire.

Figure 73 and 74 Vibracore which takes 3" cores up to 40' long in water depths to a few hundred feet.

Figure 75 and 76 The hydra core is our development as a smaller more portable system for cores to 20' plus the system can be operated out of a small runabout.

Figure 77 Underwater diamond drilling.

Land and Marine Applications and Economics
of Engineering Geophysics.

Integrated programs of subsurface investigation have yielded significant cost savings.

Our experience dictates a range of 1/4 to 1/2 being typical. At the same time data density will commonly be increased from 10 to 100 or more. Portability, along with lower mobilization costs are also major economic benefits.

It is not suggested that the geophysical methods replace the direct sampling methods, but that they be used in conjunction with them to obtain a much higher percentage of sampling volume with decided economical advantage. The suggested approach in many cases is to utilize the geophysical methods for preliminary reconnaissance, then drill a few selected reference holes for ground truth calibration and continue with geophysical as required to provide the lateral desired data density and confidence level.

If subsequent direct sampling is required it can be done with a high level of confidence avoiding wasted drilling time to perform reconnaissance.

Proper choice of appropriate methods will provide maximum information return at minimal cost. Frequently this choice is improperly made for many reasons.

So often one is expected to do miracles with his black box. Or, a new user wants to or is forced into tackling jobs beyond his present capability level. Maybe he did choose the wrong instrument and technology for the job. Maybe it should have been a multi-instrument approach or possibly other disciplines contained the best method of approach.

Choice of the proper technology, instrument, personnel, field procedure, and interpretation are essential to produce results in more than simple situations.

A good example is detection of subsurface cavaties. Here we not only consider seismic, resistivity and gravity, but also any additional source of information as to the local site. Many times ground truth by drilling is ruled out on the basis of potential collapse. Here then the multiple technological approach is even more important to provide strong correlation to confirm a find.

Each site will have its particular characteristics and field procedure will change accordingly. Subsequently a field approach using resistivity might vary considerably from job to job.

The optimal choice can seldom be made without gathering considerable back-ground information and possibly after some initial site work.

Other sources of information are vital to success. Federal, State and County records are obviously reviewed. Discussions with property owners, local drillers and anyone who is very familiar with the setting is important. Leading questions can extract considerable relevant information from lay persons familiar with the area.

Summary

Many classical geophysical methods are applicable to the field of engineering geophysics. We believe successful application depends upon a number of factors:

Quality (State of Art) equipment must be utilized.

Sound Geophysical knowledge and field techniques must be employed.

A variety of techniques and equipment must be available to fit the particular job requirements. Commonly a multiple interdisciplinary approach is necessary to produce results.

Field work must include "calibration" and some means of direct sampling (Outcrops, cuts or borings).

On site data reduction and preliminary interpretation are essential to produce economic results.

ENGINEERING GEOPHYSICAL METHODS

LAND	MARINE
REFRACTION	PRECISION ECHO SOUNDING
REFLECTION	SUBBOTTOM PROFILING
RESISTIVITY	SIDE SCAN SONAR
GRAVITY	REFRACTION
MAGNETICS	MAGNETICS
	RESISTIVITY
	GRAVITY
BLAST & VIBERATION MEAS.	SPECIALIZED SEDIMENT & ROCK CORING
BORE HOLE METHODS	IN-SITU " SOIL MECHANICS"
WELL LOGGING	
SHEAR WAVE MEAS.	

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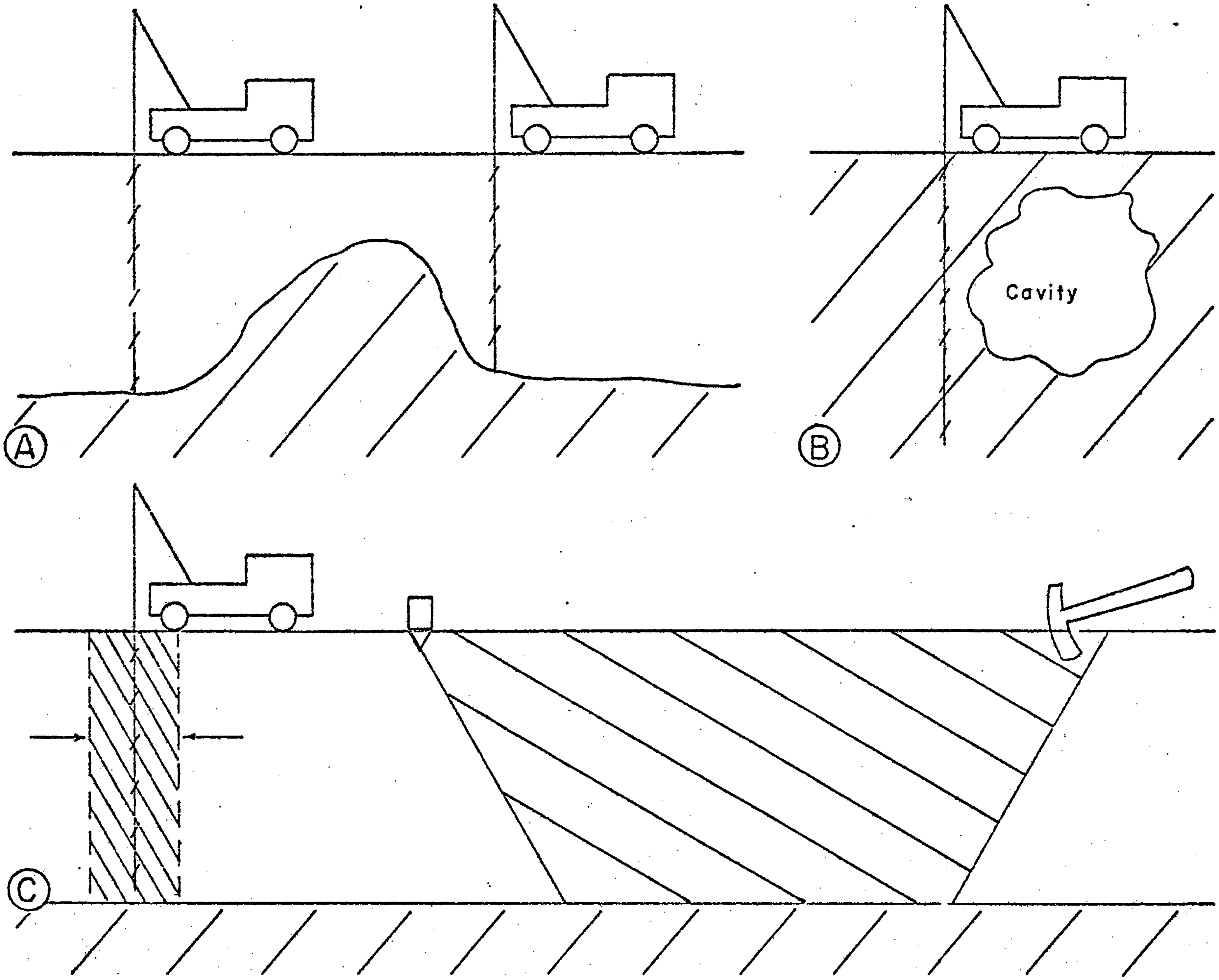
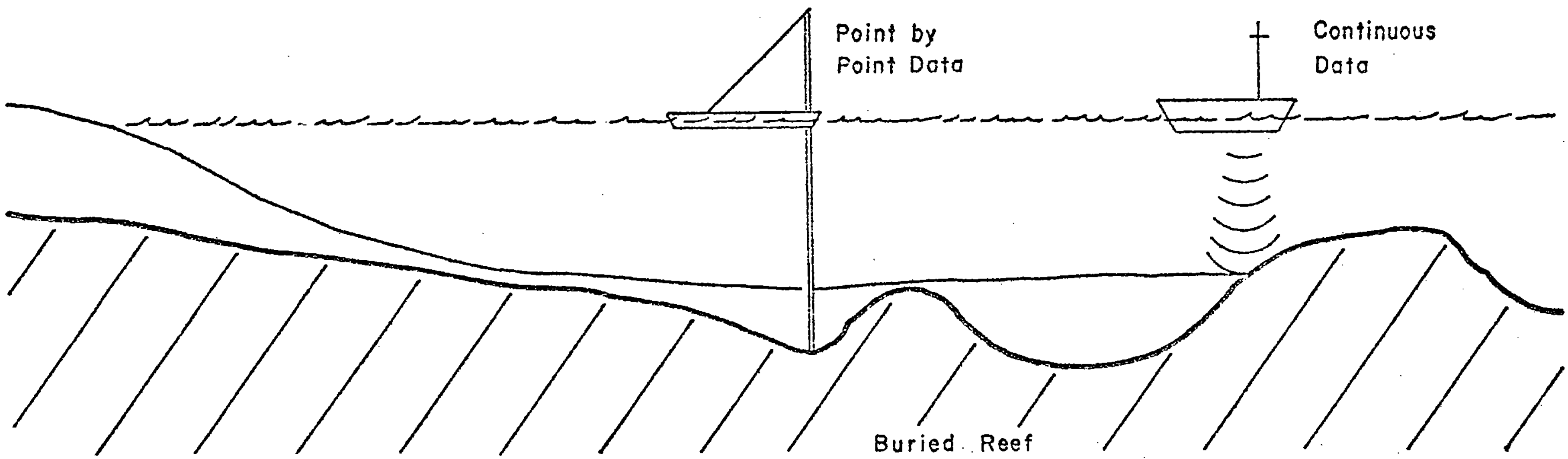
APPLICATIONS

FOUNDATIONS SOIL MECHANICS
GEOLOGIC STRUCTURAL
CAVERN DETECTION
ECONOMIC RESOURCES
SITE PLANNING
CONSTRUCTION & BIDDING
GROUND WATER SALTWATER INTRUSION
DETAILED MATERIAL STRUCTURAL
ASSESSMENT
DREDGE AND FILL
SEISMIC SITE - EVALUATION AND DESIGN
ENVIROMENTAL
SCIENTIFIC



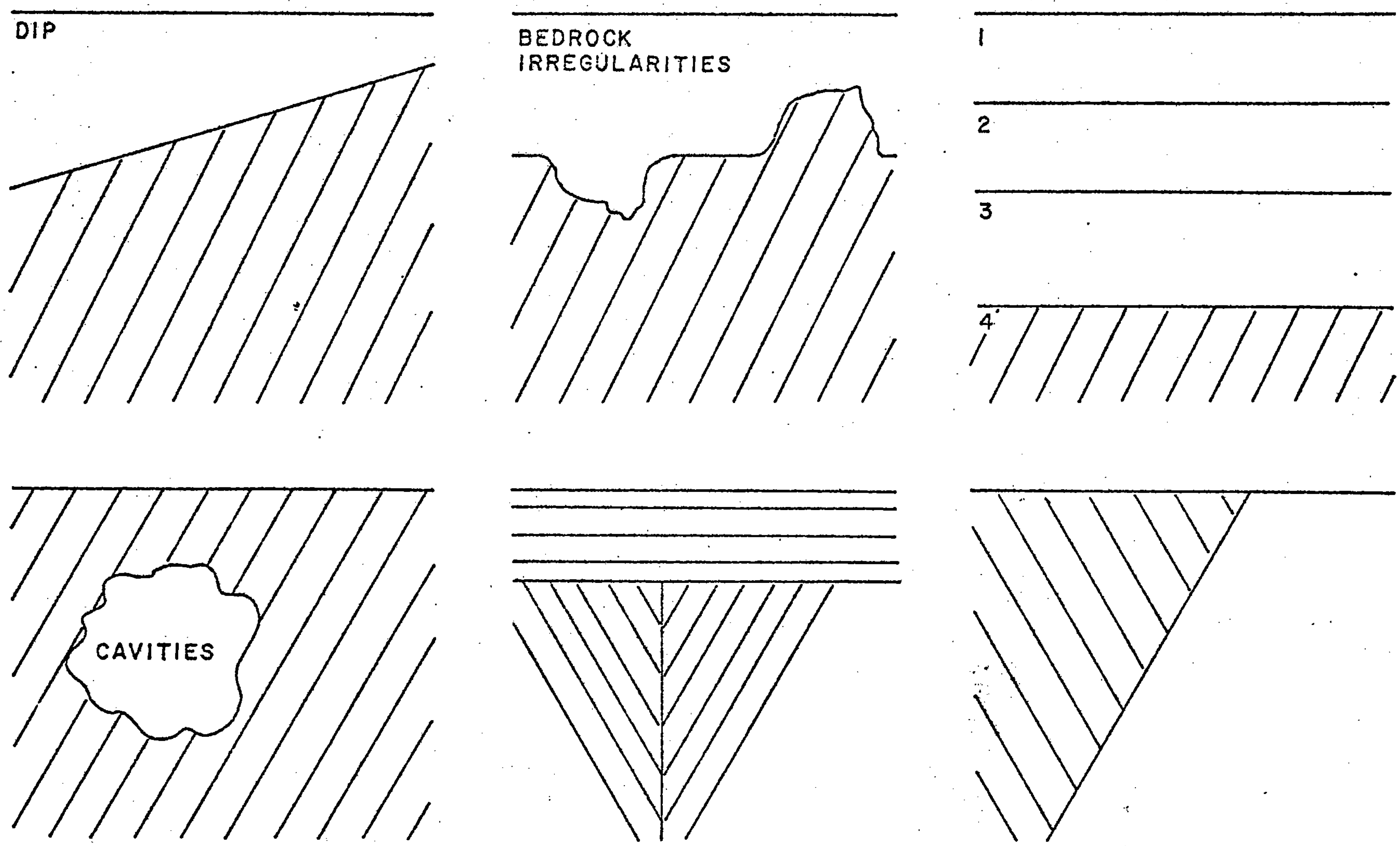
1 DISCRETE DRILL SAMPLING VS BULK GEOPHYSICAL METHODS

OUTFALLS AND PIPELINES
DREDGING AND MINING OPERATIONS



MARINE SITE EVALUATION
DISCRETE POINT DATA
VS CONTINUOUS
GEOPHYSICAL METHOD

3



4

MULTI LAYER STRUCTURES
AND STRUCTURAL FEATURES

FIG. 5

TECHNICAL AND ORGANIZATIONAL ASPECTS IN PLANNING

SUBSURFACE REQUIREMENT

PROJECT MANAGER

FORMULATE REQUIREMENT

PROJECT MANAGER - GEOPHYSICIST

GEOLOGIST - SOIL MECHANIC - OPERATIONS

FIELD OPERATIONS

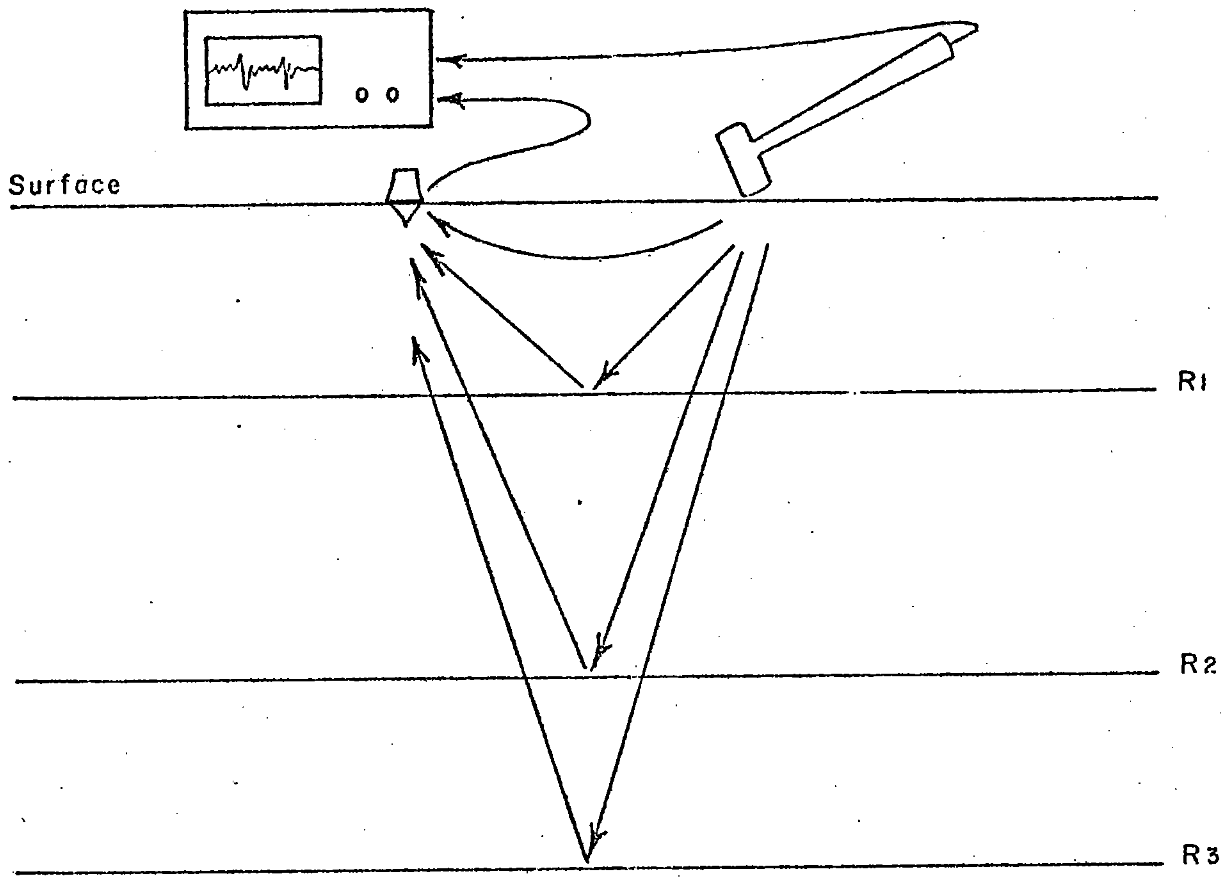
EQUIPMENT	OPERATIONS MANAGER	VESSEL AND HELMSMAN
OPERATOR	QUALITY CONTROL	SURVEY/ POSITIONING
GEOLOGIST		EQUIPMENT AND TEAM
GEOPHYSICIST		

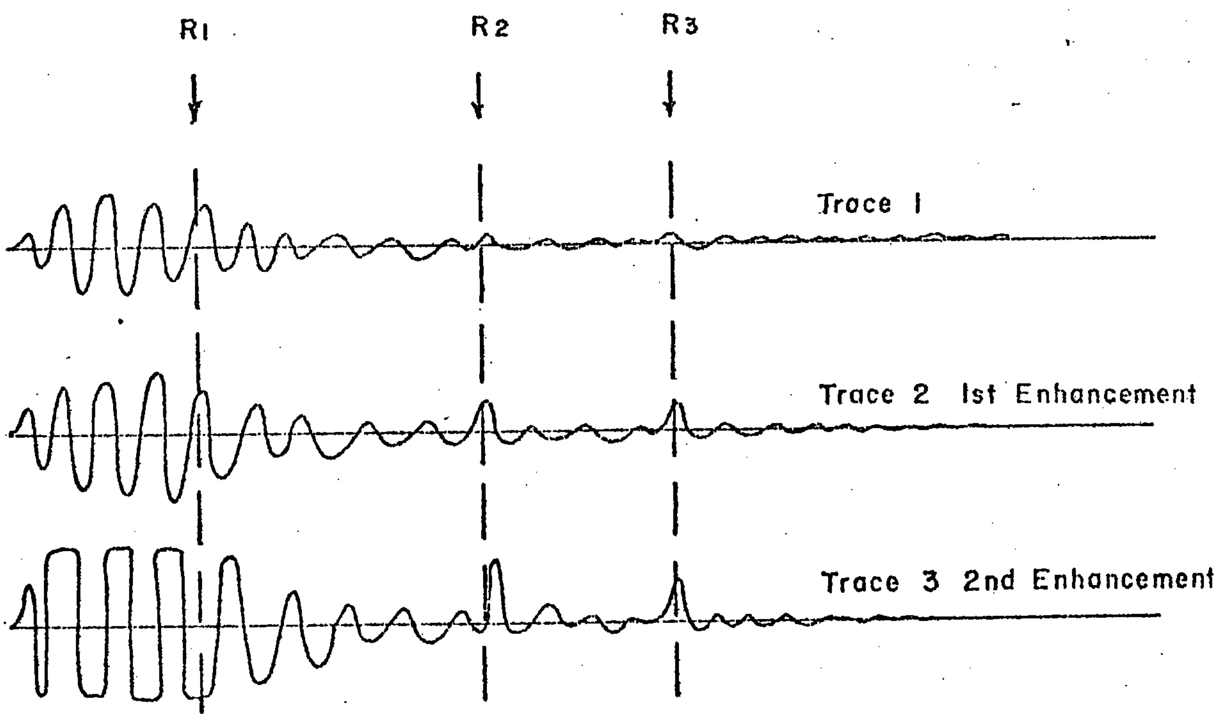
DATA REDUCTION

INPUTS FROM ALL ABOVE

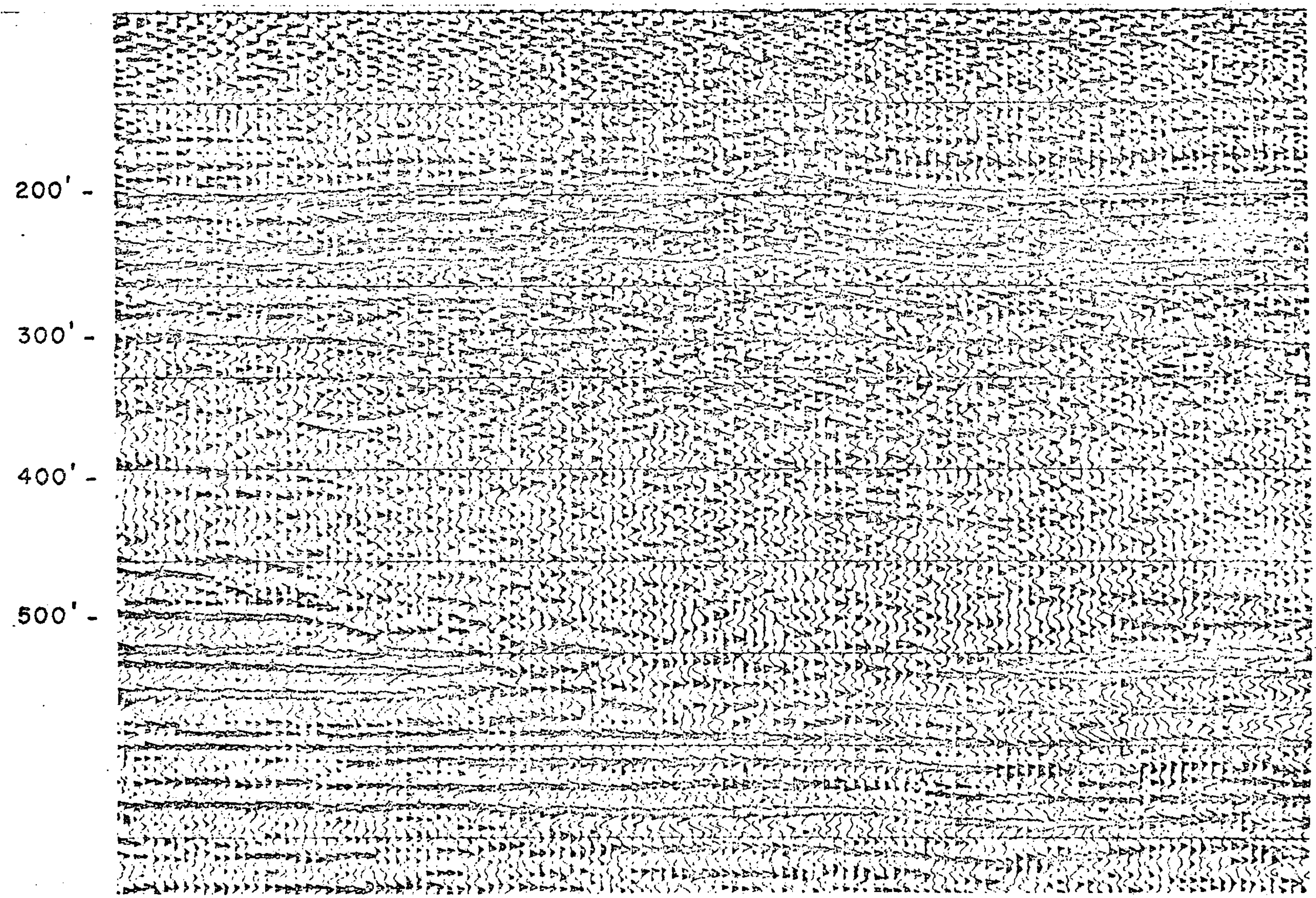
FINAL REVIEW AND

PROJECT REVIEW

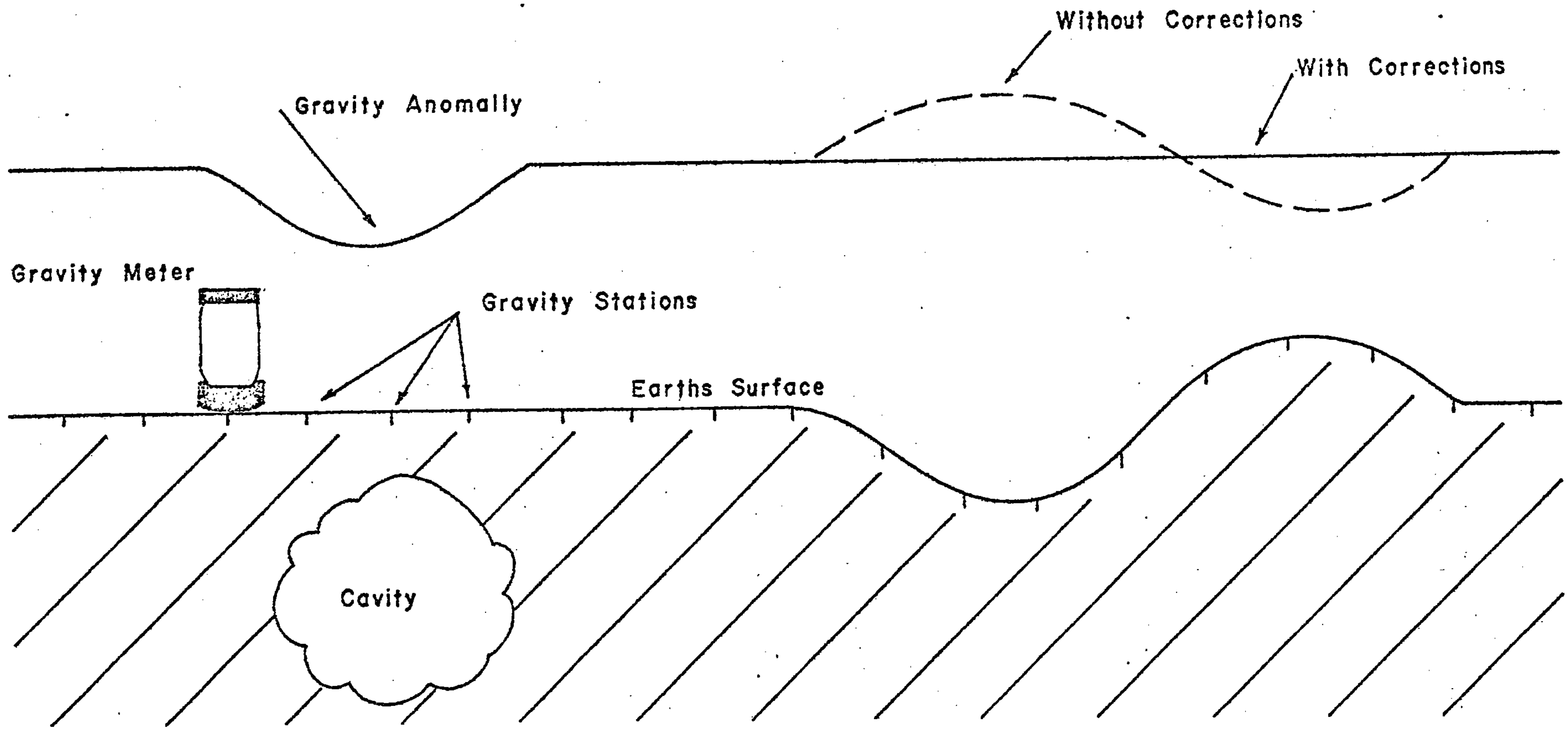




12 BASIC SEISMIC REFLECTION TECHNIQUES



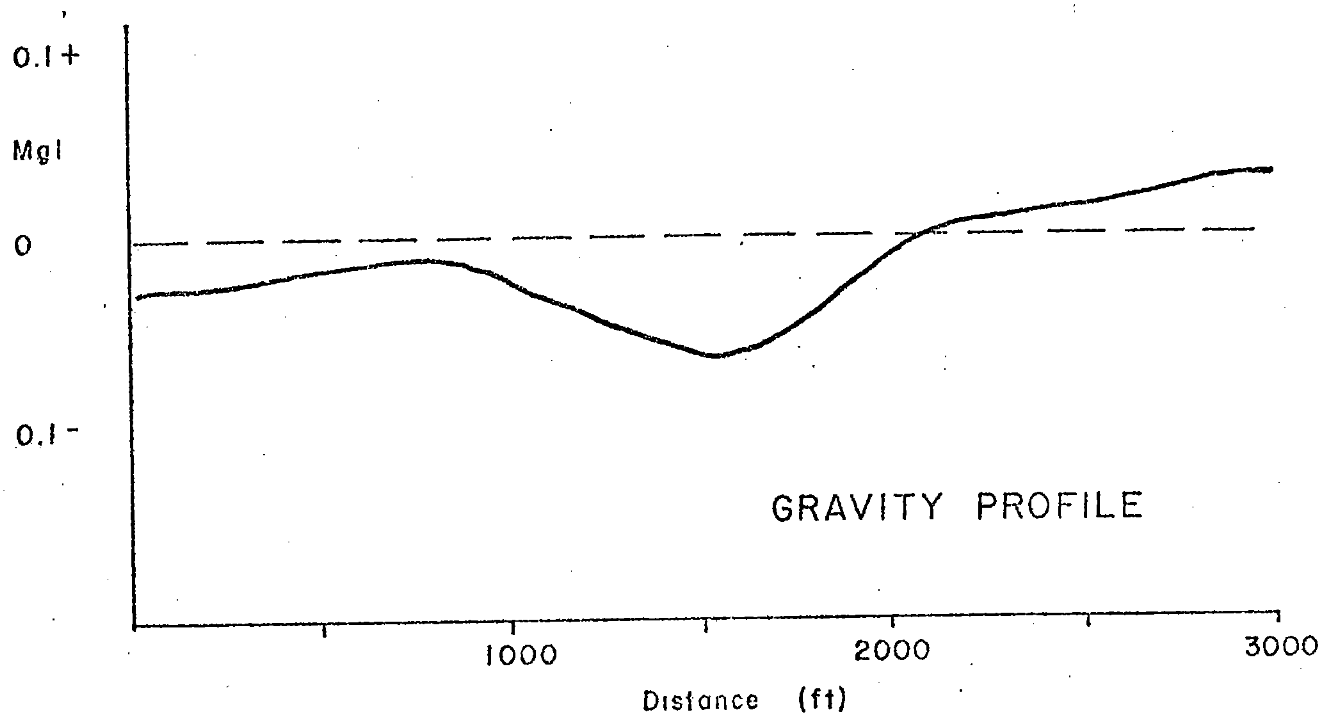
13 HIGH RESOLUTION SEISMIC REFLECTION

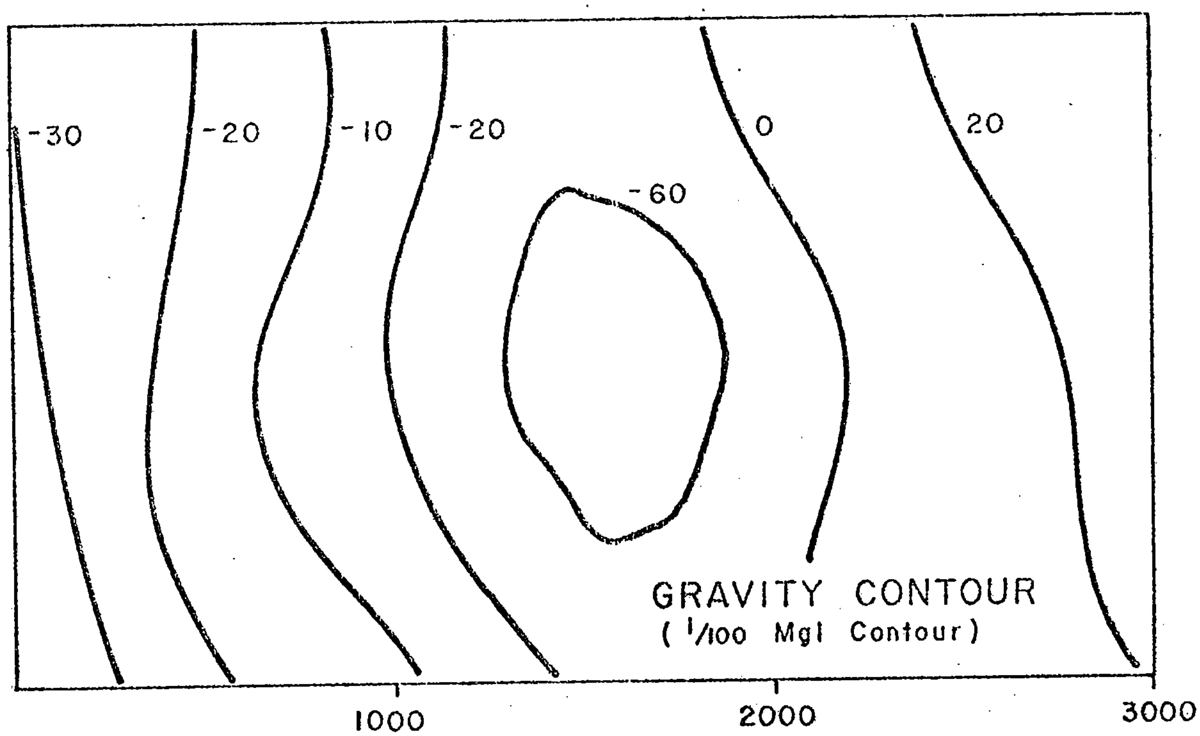


Instrument Reading + {Latitude Elevation Bouguer Terrain} + Instrument Drift = Gravity Profile

19

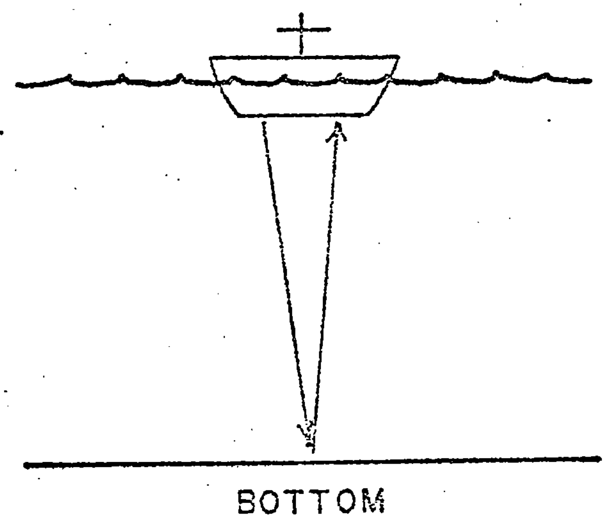
MICRO GRAVITY SURVEYING


0.1+
Mgl
0
0.1-
GRAVITY PROFILE
1000
2000
3000
Distance (ft)


-30
-20
-10
-20
0
20
-60
GRAVITY CONTOUR
(1/100 Mgl Contour)
1000
2000
3000

15

FIG. 16 MARINE ECHO SOUNDING



USED FOR

CHARTING DEPTHS (BATHYMETRY)

SITE EVALUATION

SEDIMENTATION / DREDGING

ENVIRONMENTAL / BIOLOGICAL / GEOLOGICAL SURVEYS

SOME MATERIAL IDENTIFICATION BASED UPON REFLECTIVITY

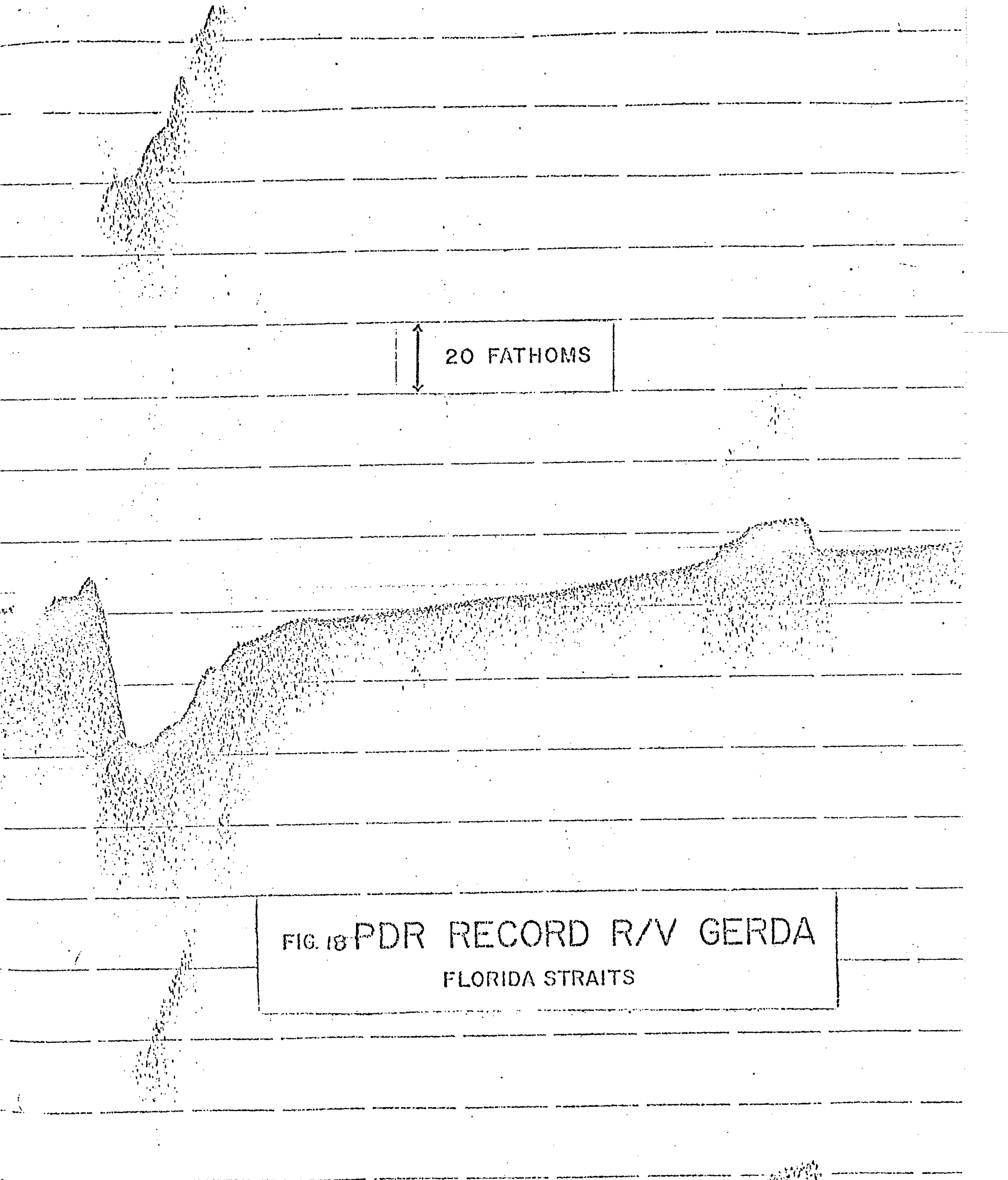
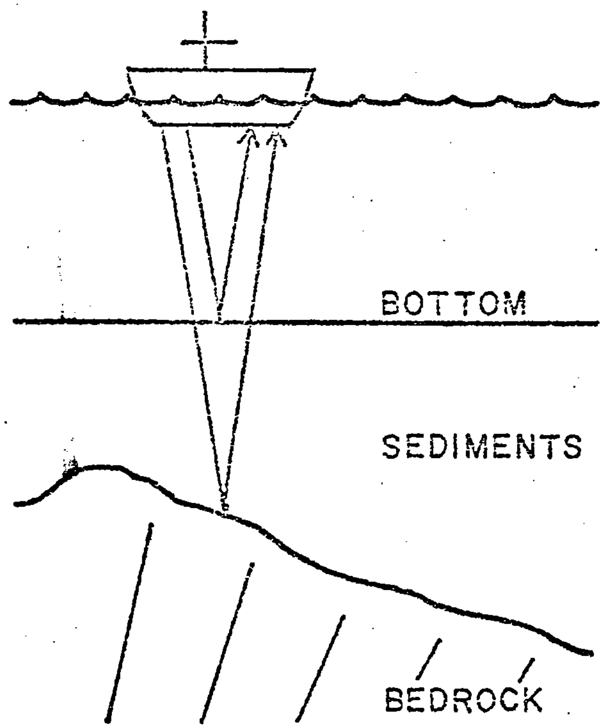


FIG. 18 PDR RECORD R/V GERDA
FLORIDA STRAITS

FIG. 19 MARINE REFLECTION SUB-BOTTOM PROFILING



USED FOR

ASSESSING SUB-BOTTOM MATERIAL AND STRUCTURE.

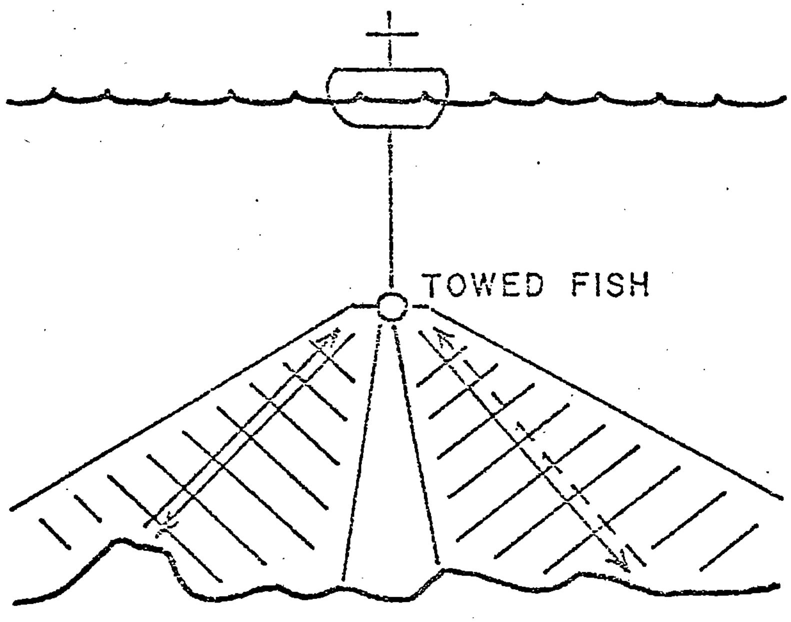
THICKNESS OR DEPTH TO AN INTERFACE LATERAL EXTENT AND VARIABILITY.

LOCATION OF ECONOMIC DEPOSITS.

BIOLOGICAL/GEOLOGICAL SURVEYS.

SOME MATERIAL IDENTIFICATION BASED UPON REFLECTIVITY.

FIG. 21 MARINE SIDE SCAN SONAR



USED FOR

LARGE AREA BOTTOM COVERAGE.

PROVIDES AN OBLIQUE "PHOTOGRAPHIC" PRESENTATION OF BOTTOM FEATURES.

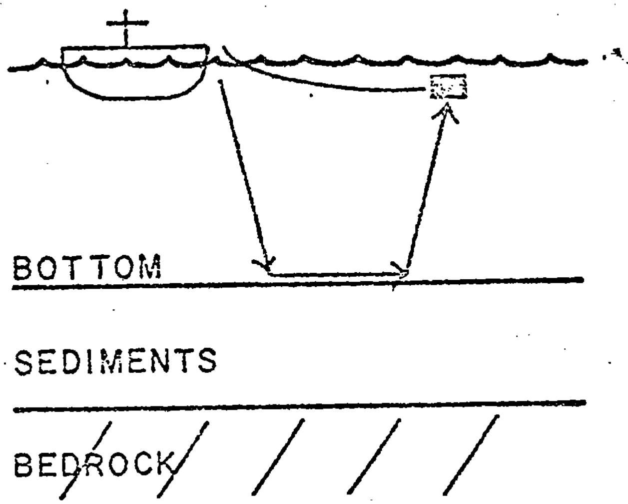
OBSERVATION OF VERTICAL IRREGULARITIES AND THEIR LATERAL EXTENT (CONTINUITY)

BIOLOGICAL/GEOLOGICAL SURVEYS

SOME MATERIAL ASSESSMENT POSSIBLE BASED UPON REFLECTION CHARACTERISTICS.

VERY USEFUL FOR SEARCHING LOST OBJECTS, PIPELINES, CABLES ETC

FIG. 23 MARINE REFRACTION SUB-BOTTOM PROFILING



USED FOR

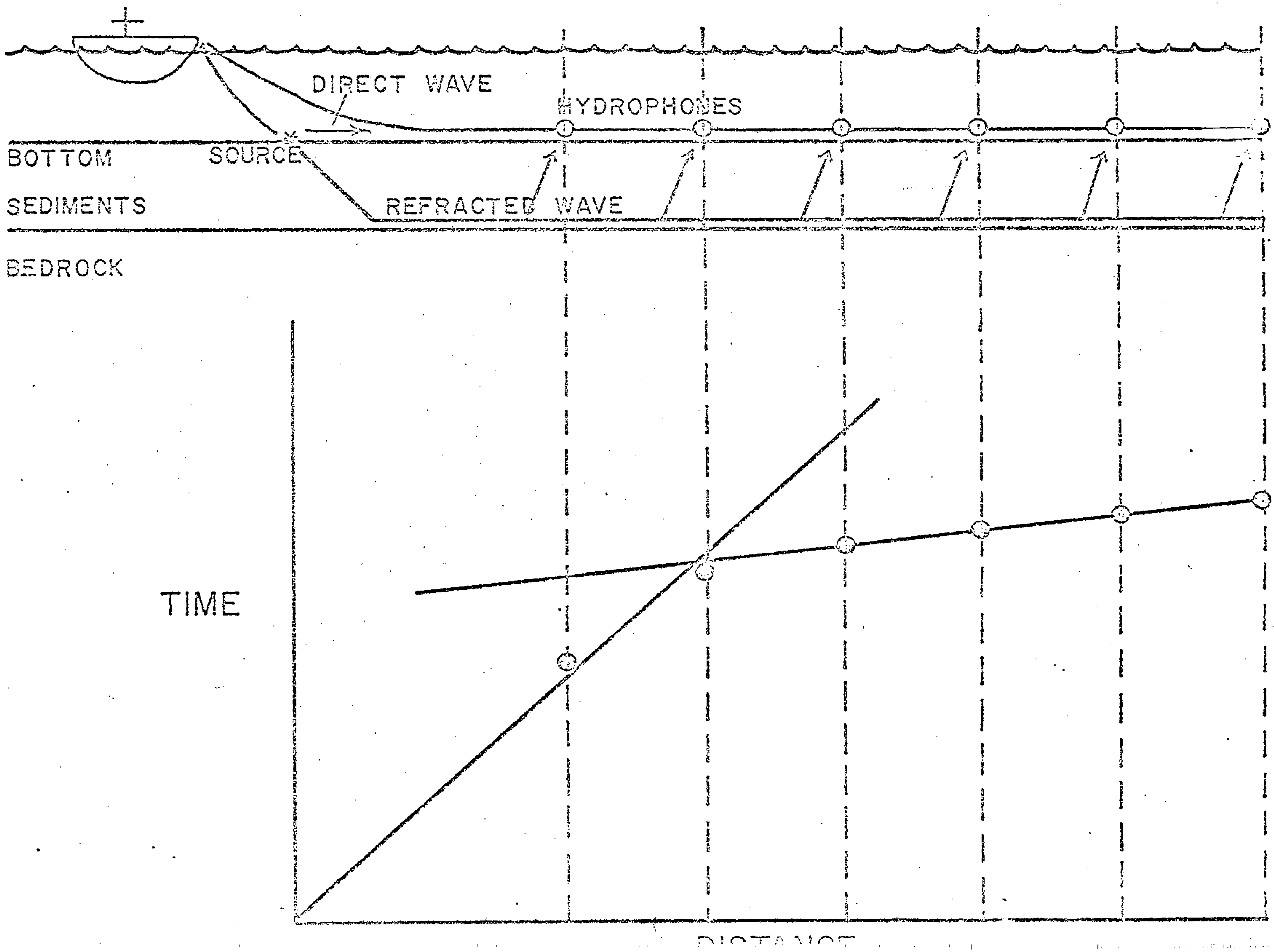
ASSESSING SUB-BOTTOM MATERIAL AND STRUCTURE

THICKNESS OR DEPTH TO AN INTERFACE LATERAL EXTENT AND VARIABILITY

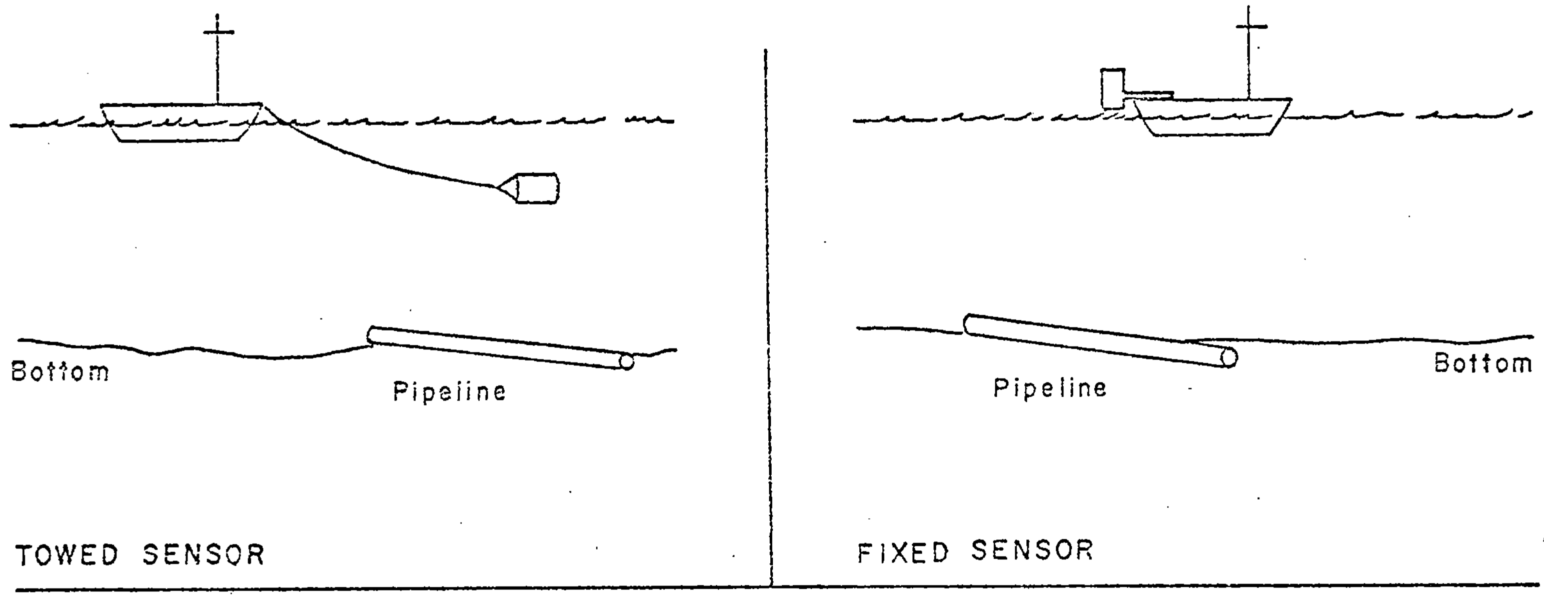
THE METHOD IS SLOWER WITH LESS DETAIL. WORKS IN SHALLOW WATER AND INTERFACE AREAS SUCH AS BEACHES AND SWAMPS

YIELDS ACOUSTIC VELOCITY OF MATERIAL WHICH IDENTIFIES MATERIAL AND PROVIDES CORRECTIONS FOR REFLECTION WORK

FIG. 24 MARINE REFRACTION TECHNIQUES



MARINE MAGNETIC SURVEY



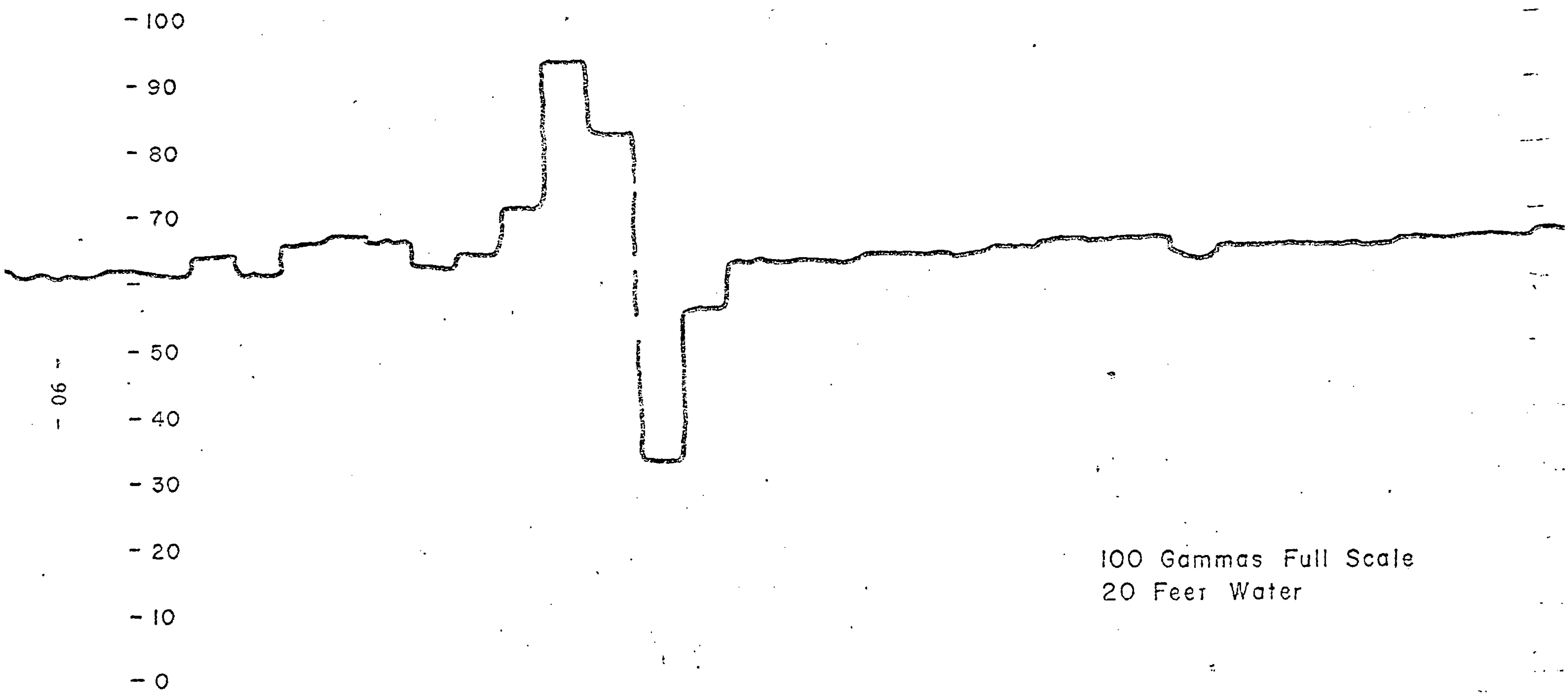
USED FOR

SEARCH PROJECTS - Location of (ferrous) pipeline / cables / wellheads

- Archeological surveys

GEOLOGIC - DETECTION OF MAGNETIC ORES AND STRUCTURES HAVING MAGNETIC ANOMALIES

25A

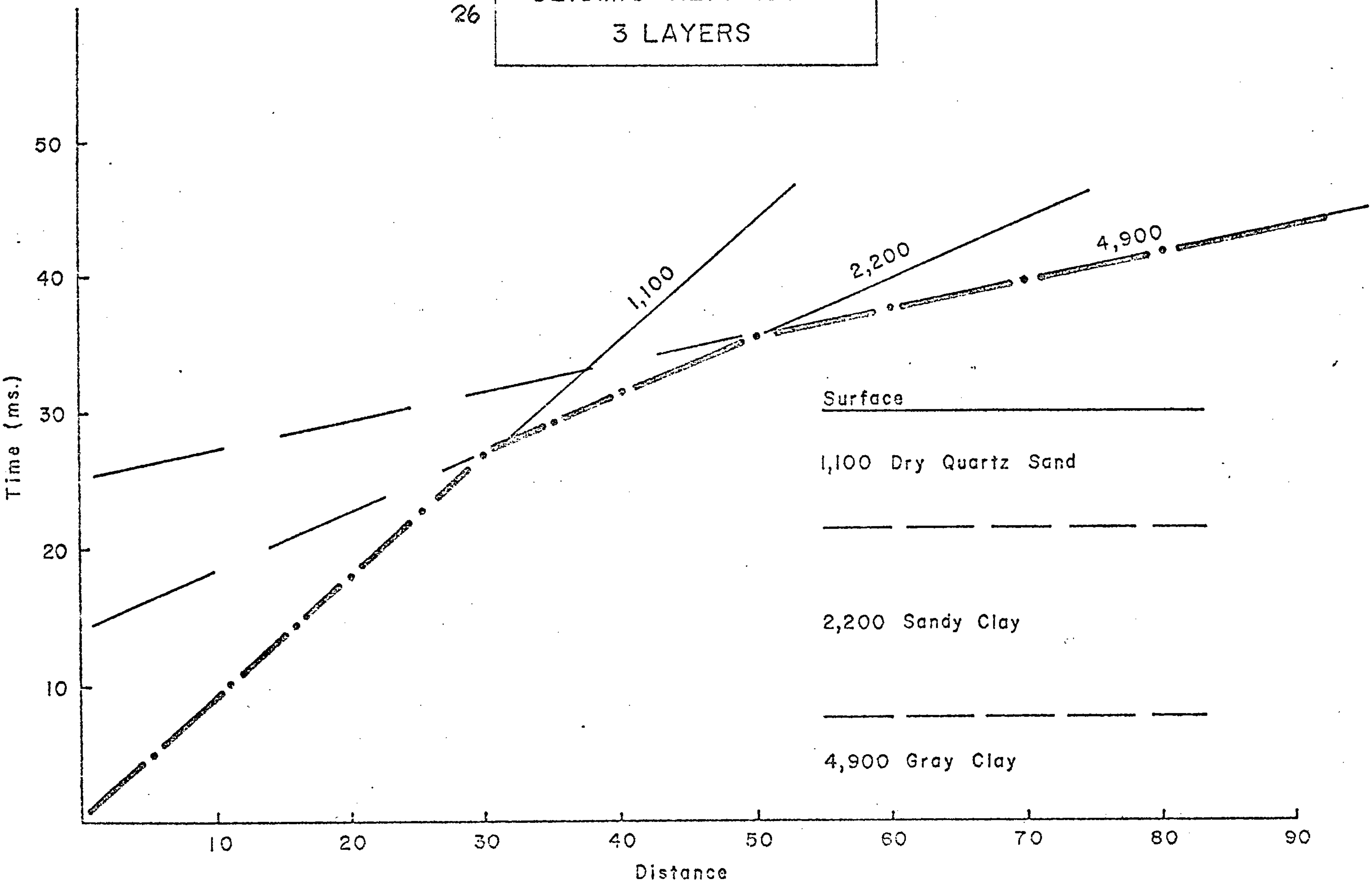


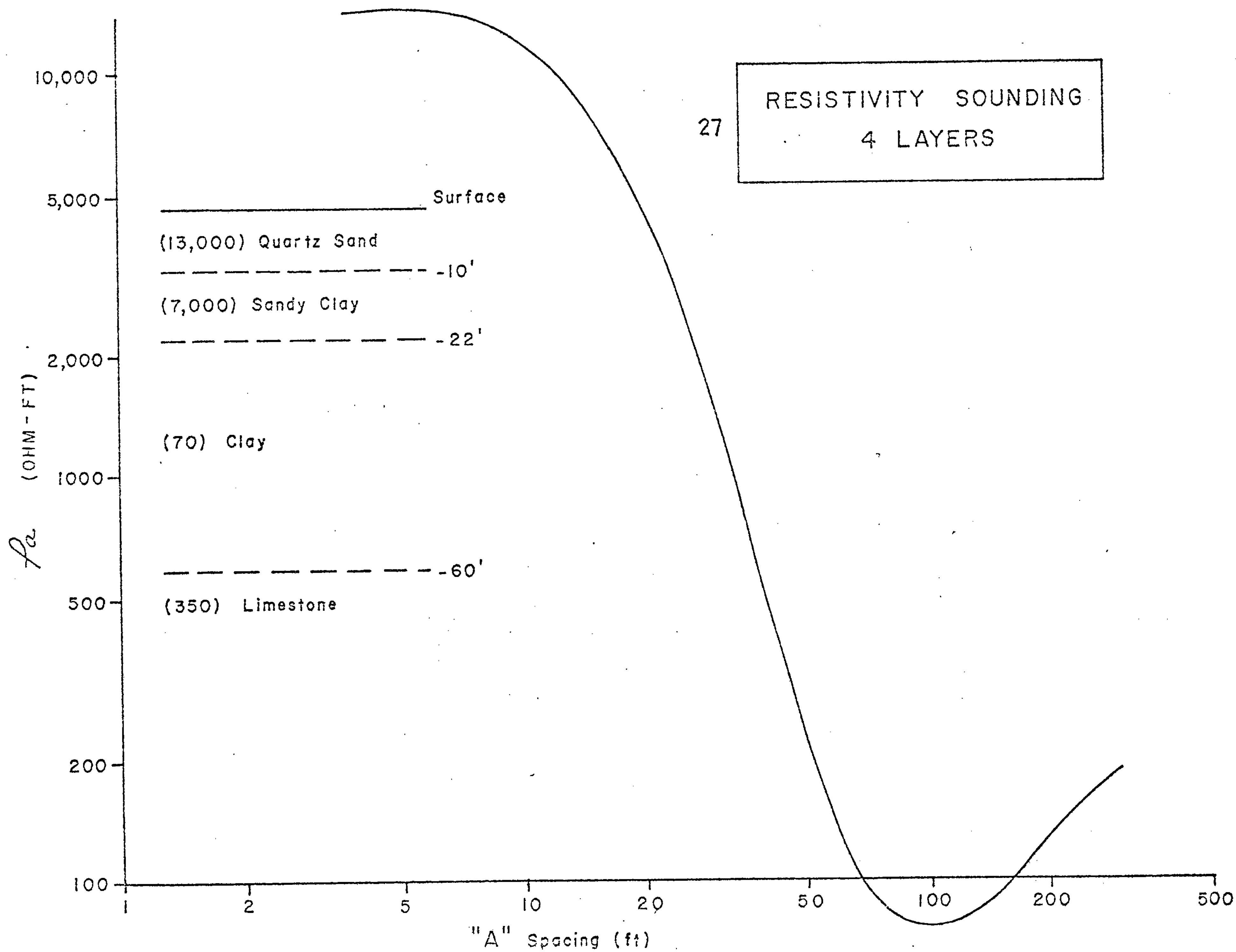
PROTON MAGNETOMETER RUN
OVER OLD ANCHOR

25 B

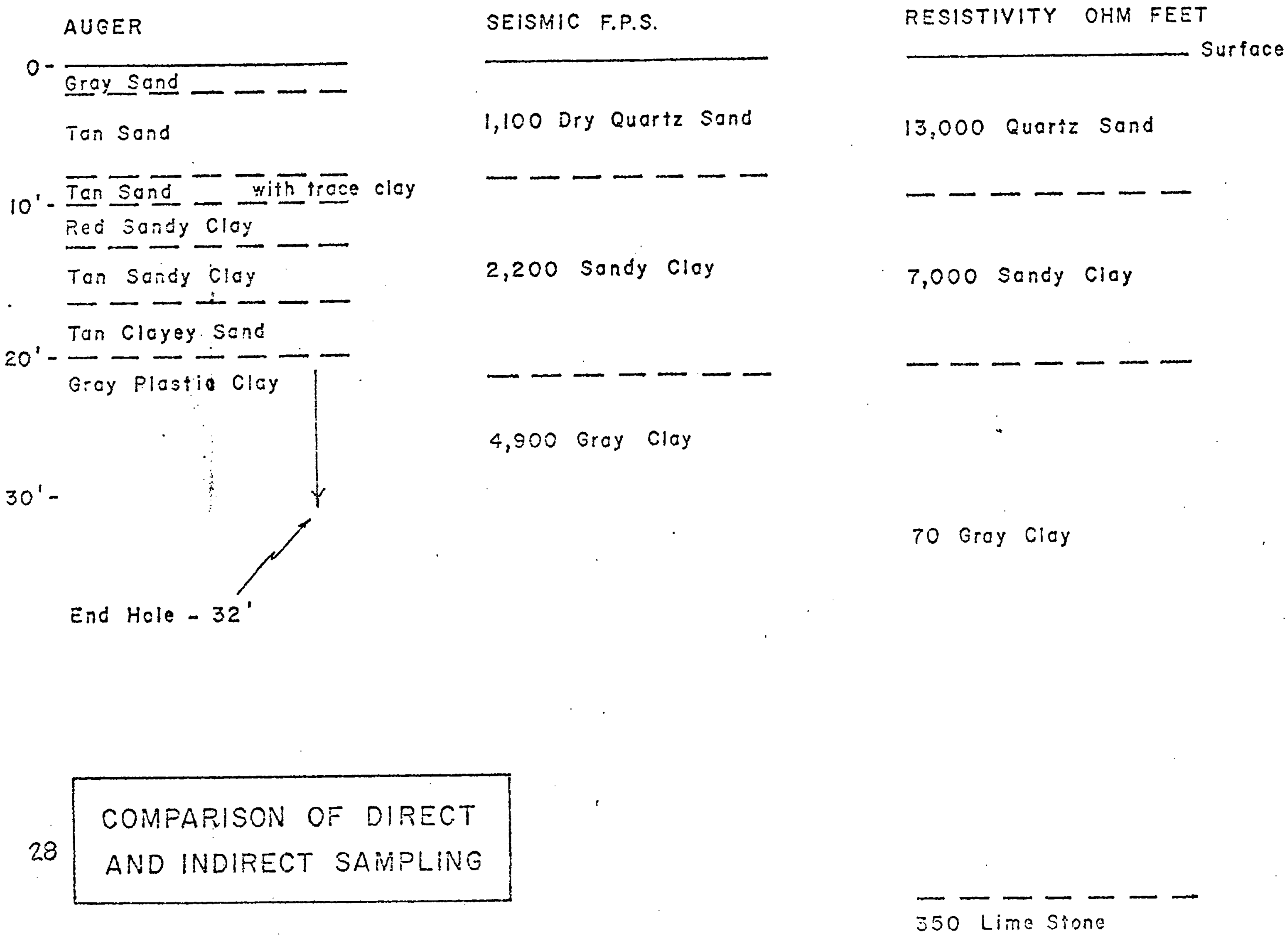
26

SEISMIC REFRACTION
3 LAYERS

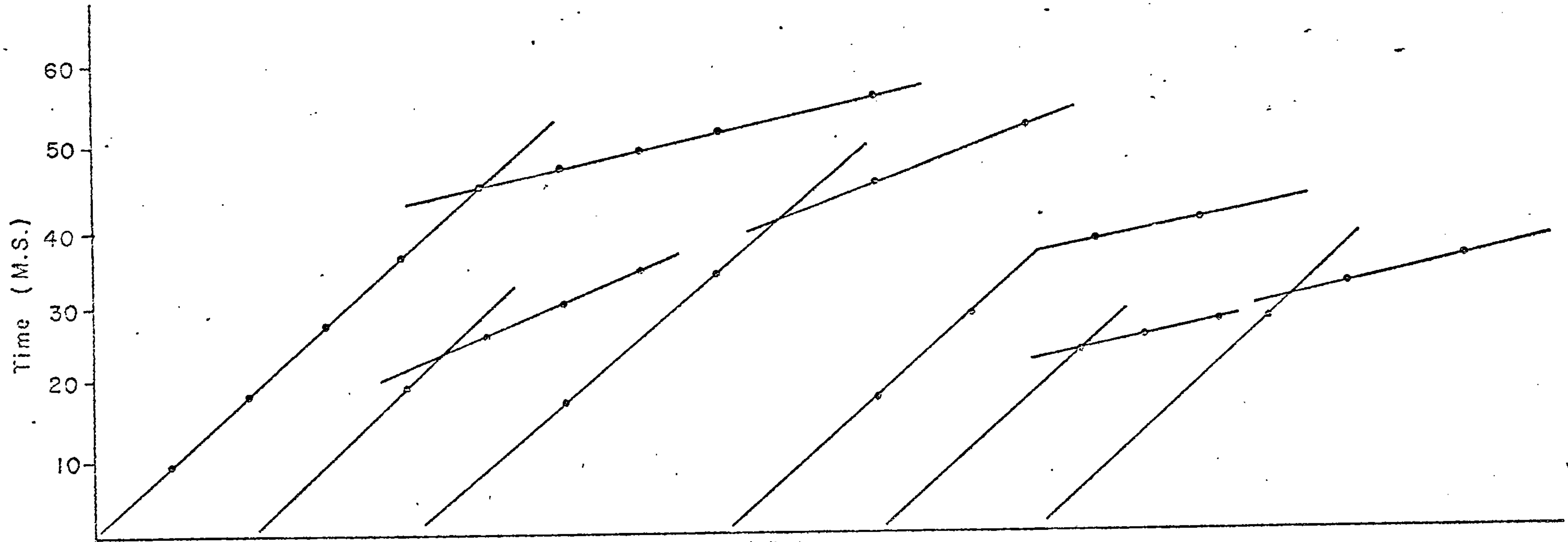




RESISTIVITY SOUNDING
4 LAYERS
27
Surface
(13,000) Quartz Sand
-10'
(7,000) Sandy Clay
-22'
(70) Clay
-60'
(350) Limestone
10,000
5,000
2,000
1000
500
200
100
ρ_a (OHM - FT)
1
2
5
10
20
50
100
200
500
"A" Spacing (ft)



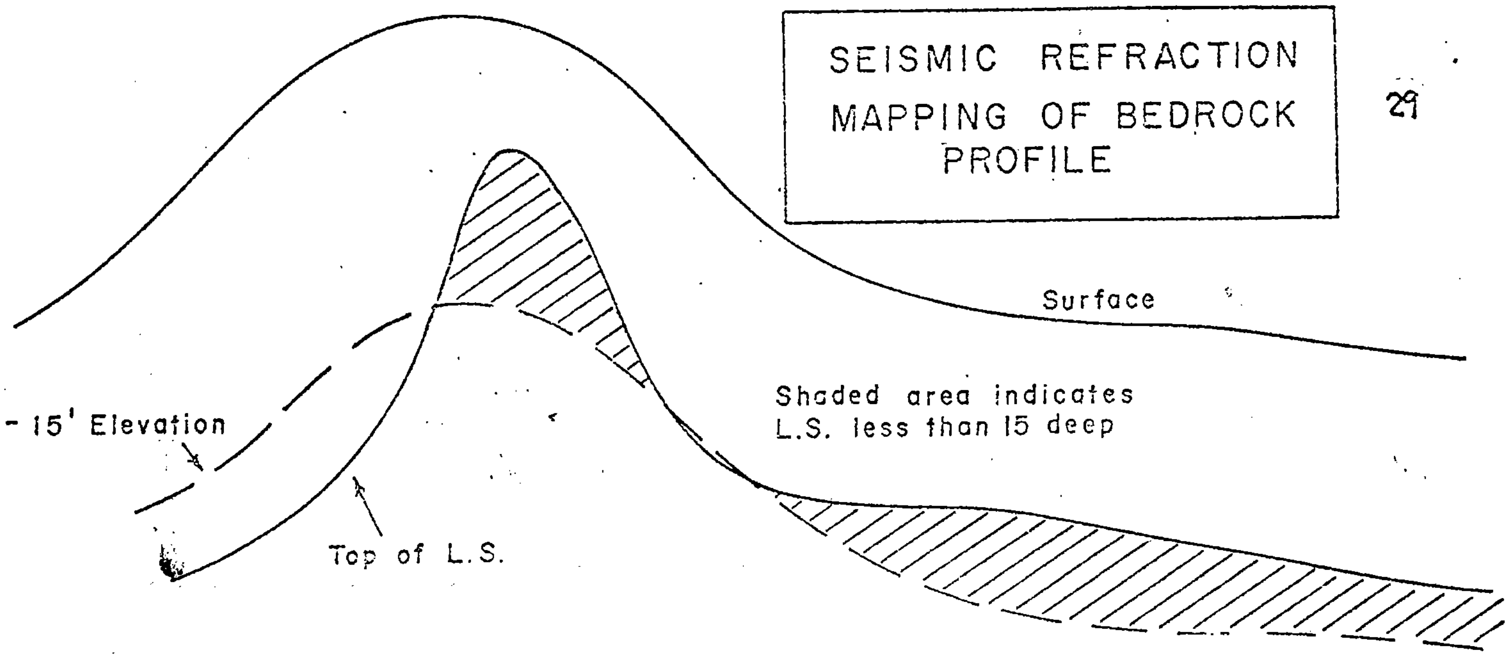
28 COMPARISON OF DIRECT AND INDIRECT SAMPLING



1" = 20' Seismic lines not spaced to actual field distance

SEISMIC REFRACTION MAPPING OF BEDROCK PROFILE

29



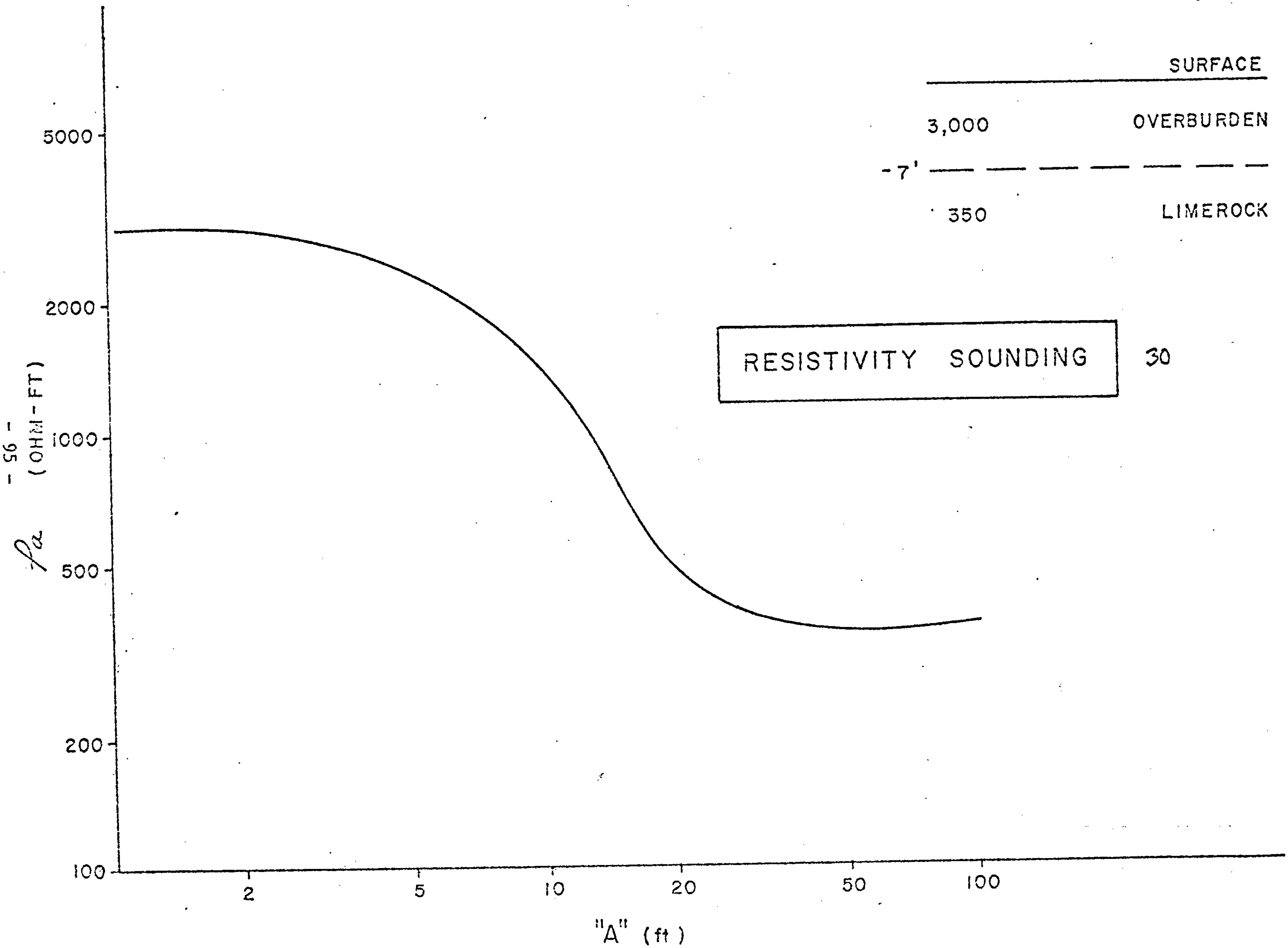
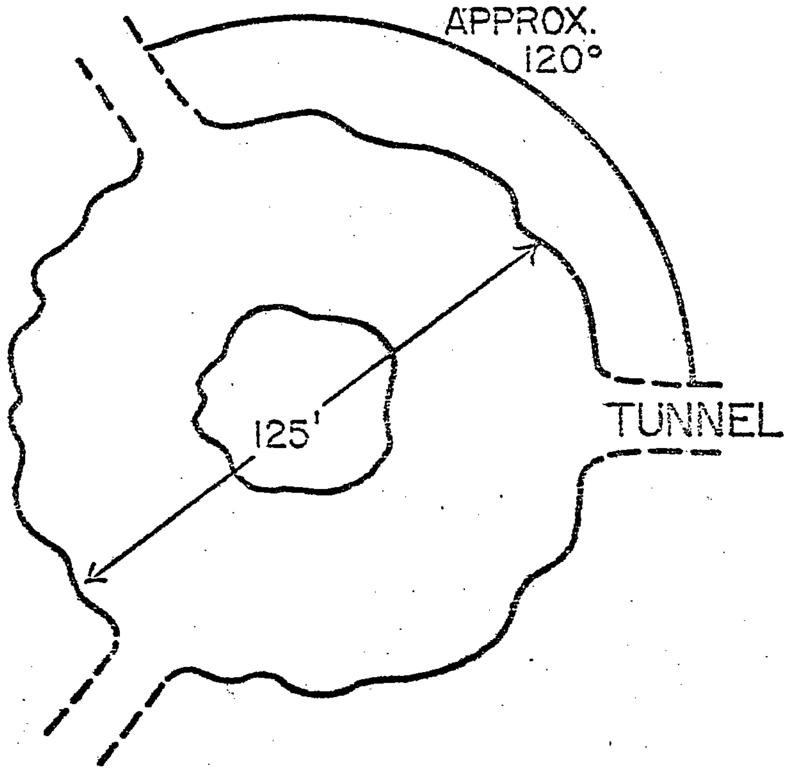
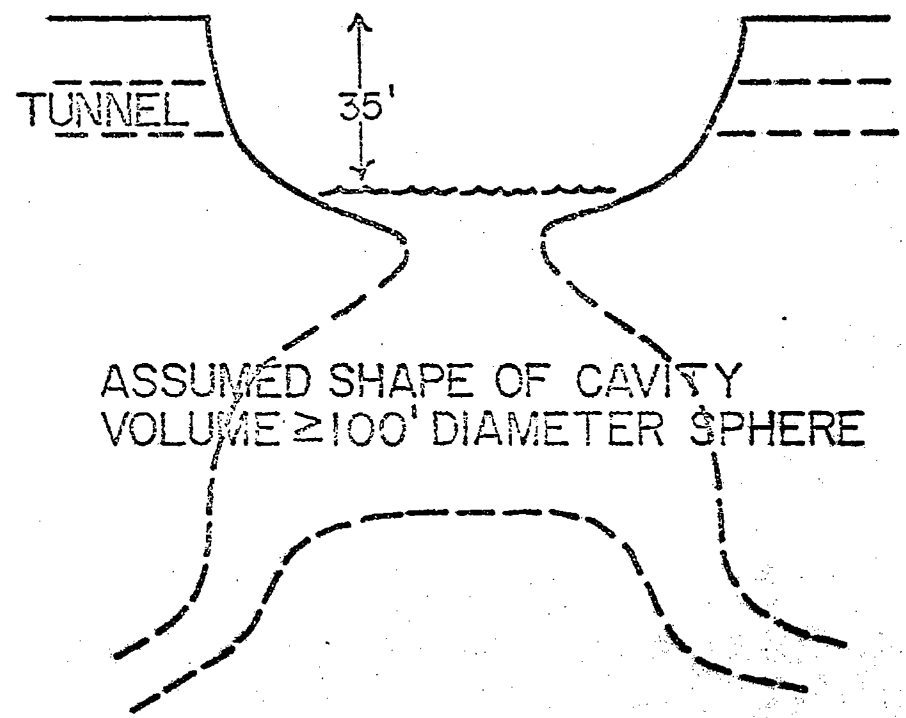

SURFACE
3,000
OVERBURDEN
-7'
350
LIMEROCK
RESISTIVITY SOUNDING
30
5000
2000
1000
500
200
100
2
5
10
20
50
100
"A" (ft)
ρa (OHM-FT)

fig. 38 BALM SINK ILLUSTRATES A COMBINATION PIPE/ROOF COLLAPSE SINK WITH WATER TABLE NEAR THE LIMESTONE ROOF



PLAN VIEW



CROSS SECTION

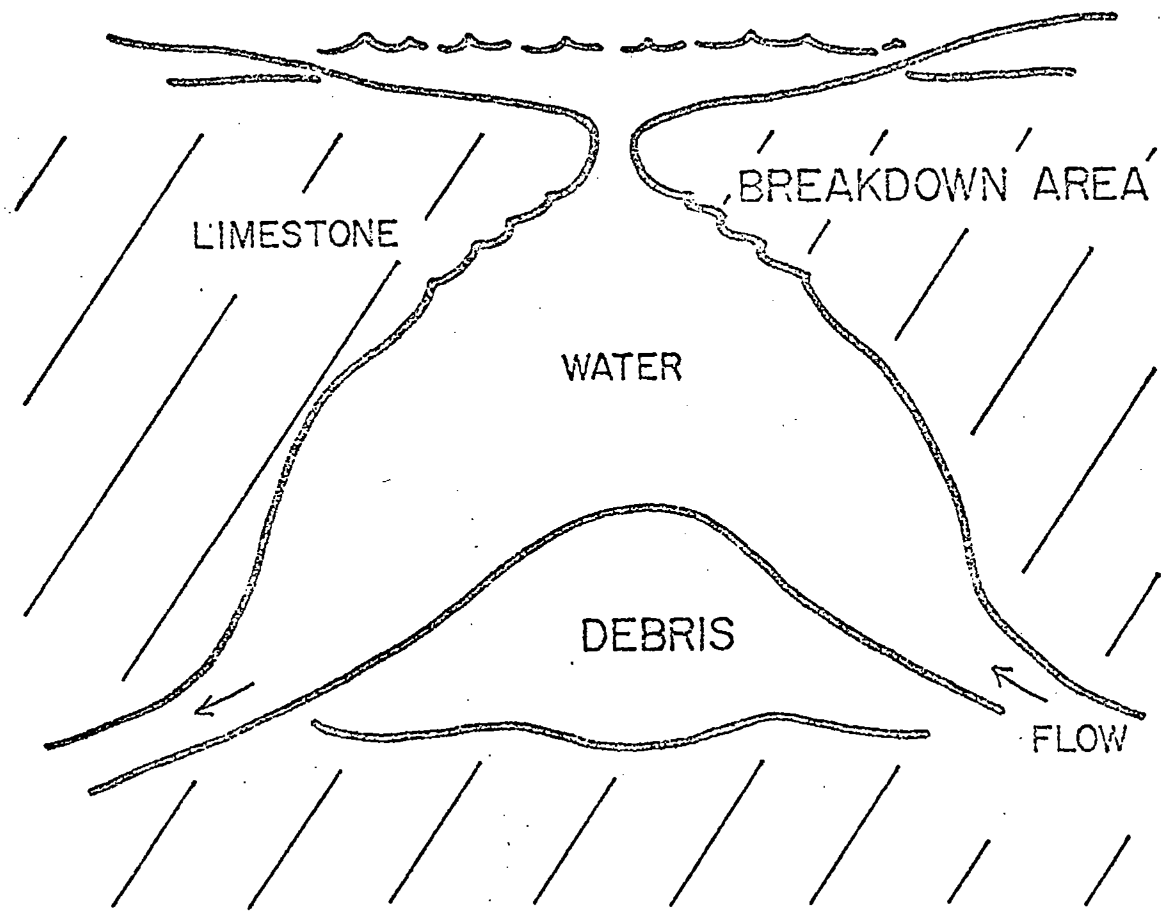
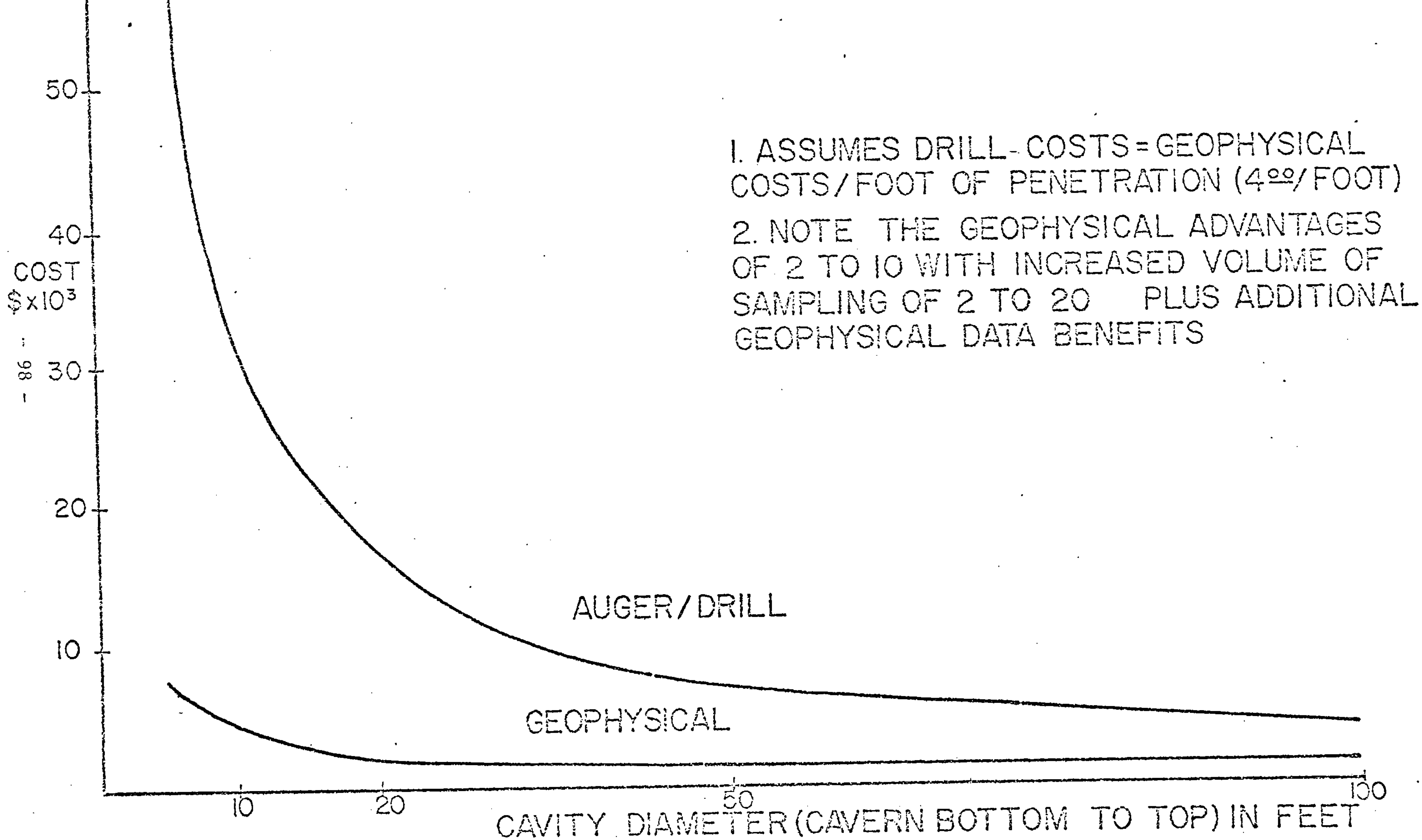


fig. 39 CROSS SECTION - TYPICAL OPEN SINK

fig. 47 COST COMPARISON OF LOCATING CAVERNS
BORING VS GEOPHYSICAL METHODS

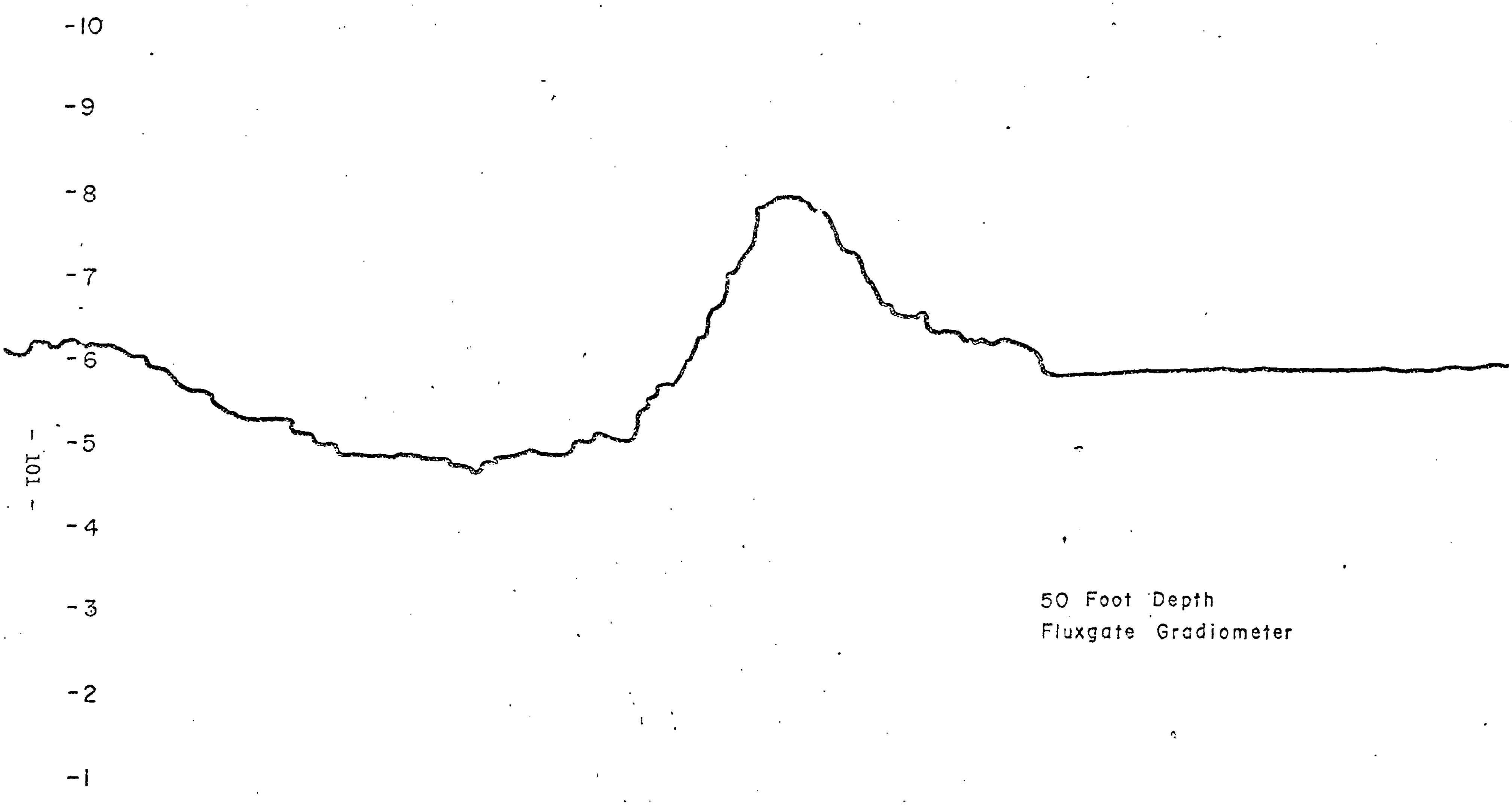


DEPTH OF INTEREST	% VOLUME AUGER	TOTAL AUGER DEPTH (feet)	AUGER COST $2.00/ft.	% VOLUME SEISMIC	TOTAL SEISMIC LINE (feet)	SEISMIC COST $2.00/ft.
10	0.9	50	$100	4.6	200	$100
20	0.9	100	$200	7.4	320	$160
40	0.9	200	$400	7.4	320	$160
80	0.9	400	$800	14.7	640	$320

43 SIMPLE 1 ACRE SUBSURFACE INVESTIGATION

	# STATIONS PER 6 SECTIONS	# OF FIELD DAYS 10 HRS./DAY	TOTAL COST GEOPHY. ONLY	GRID SPACING	COST/DATA POINT	COST/DRILL HOLE $2.00/ft.	TOTAL COST OF DRILLING ONLY
300 ′ CENTERS	1500	90	$54,000	300′ x 400′	$36	$170	$224,400
600′ CENTERS	750	50	$30,000	600′ x 400′	$40	$170	$112,200
900′ CENTERS	500	35	$21,000	900′ x 400′	$42	$170	$56,100

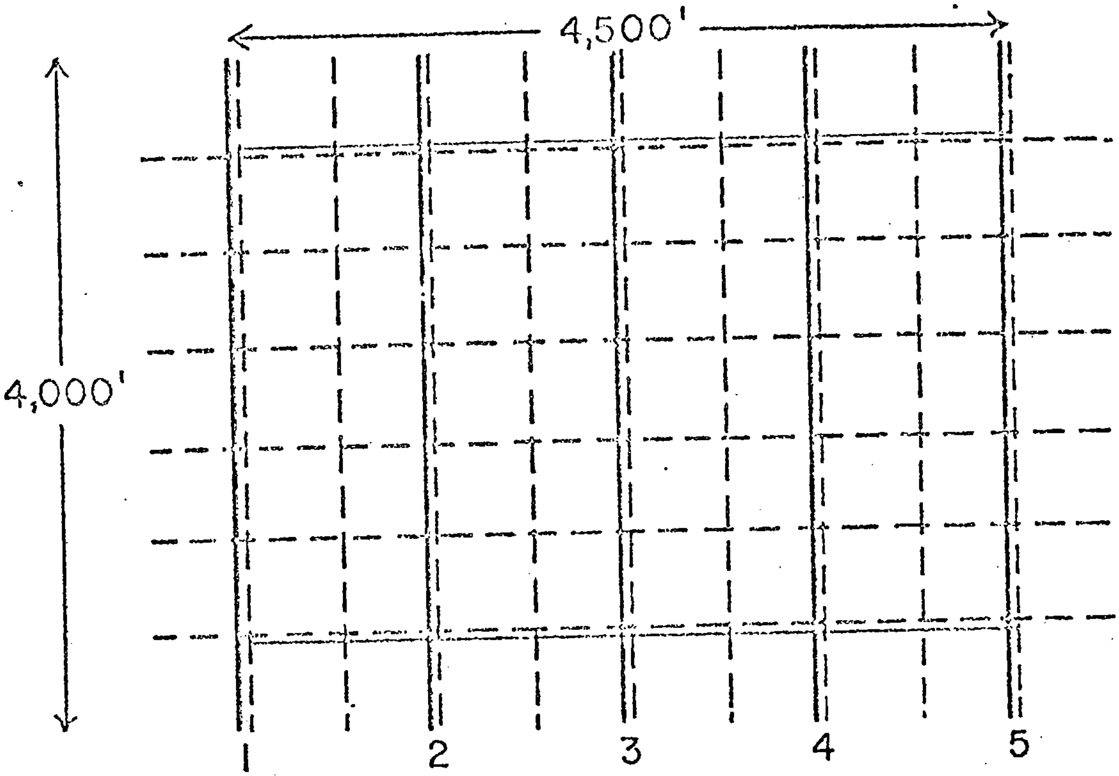
49 TECHNICAL & COST DETAILS FOR GEOPHYSICAL (RESISTIVITY) SURVEY OF SIX SECTIONS OF LAND



52

MAGNETOMETER LOCATION
OF DRAIN PIPE

FIG. 71 EXAMPLE - MARINE SITE EVALUATION

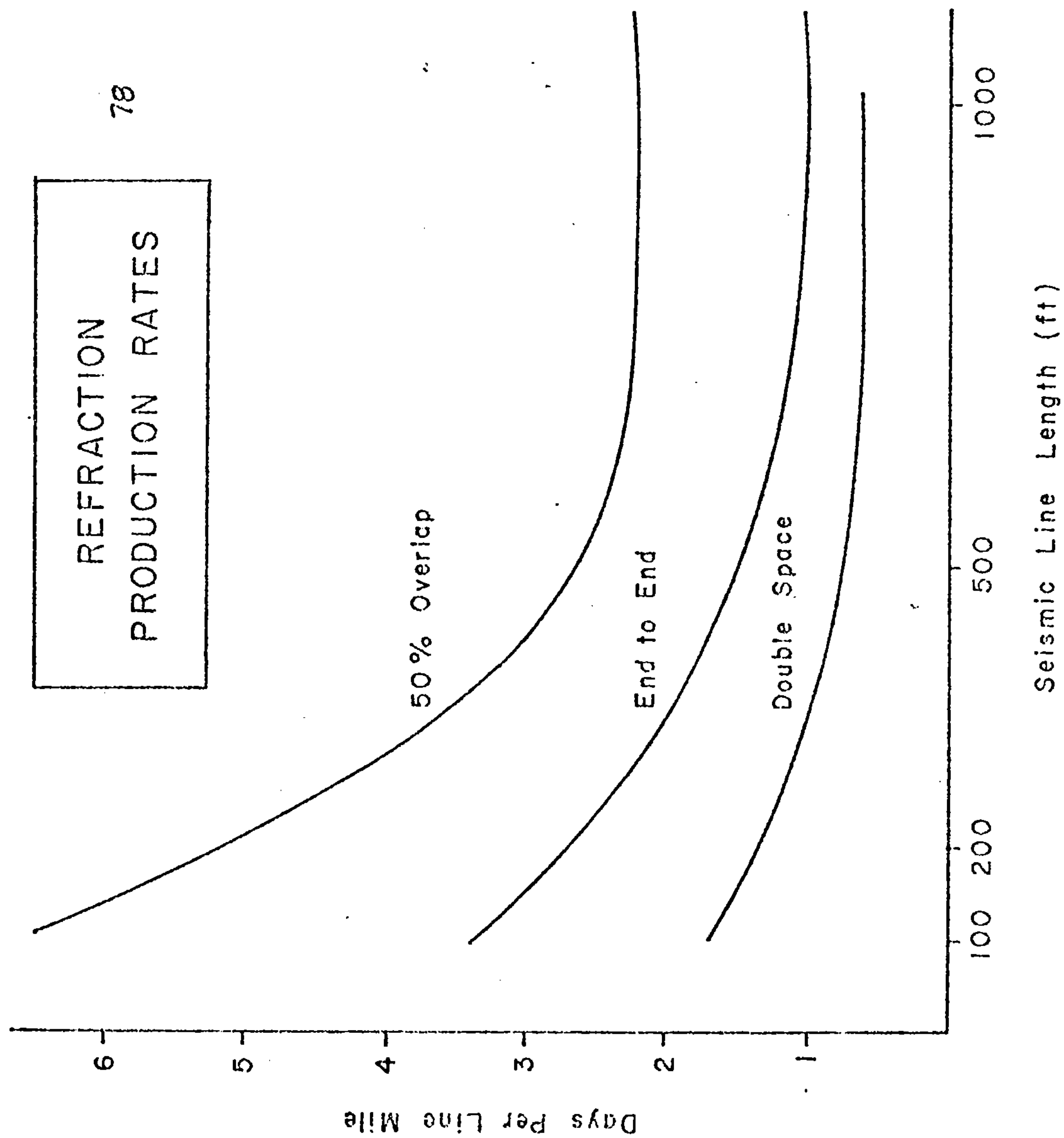


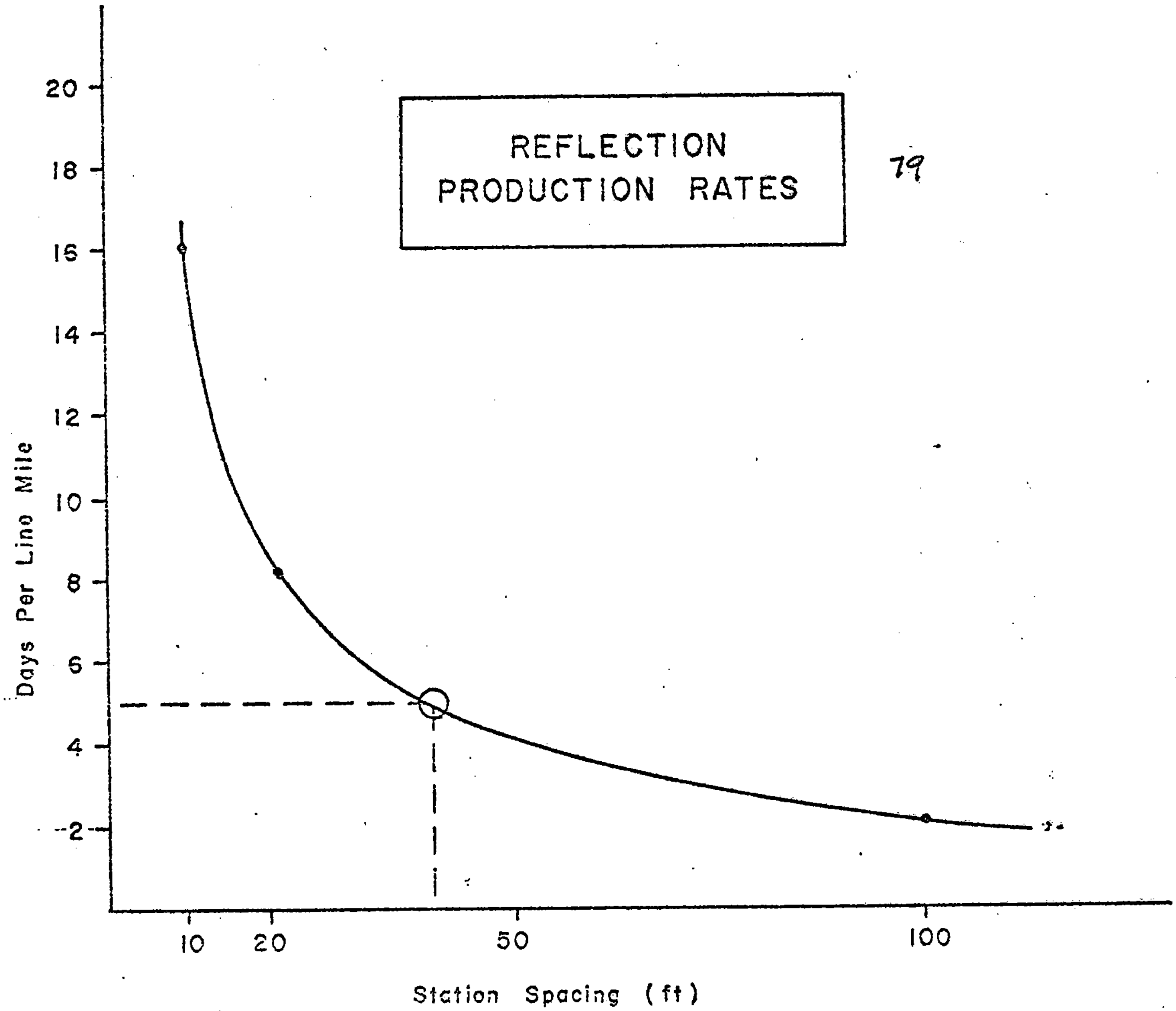
DIRECT SAMPLING - WASHBORINGS (LINES 1 THRU 5)

20,000 LINEAR FEET OF SAMPLING LINE
155 WASHBORINGS TOTAL
31 WASHBORINGS PER LINE (129 FEET APART)
DENSITY 2.7 ACRES/WASHBORING
LARGEST UNSAMPLED AREA 104 ACRES
TIME FEW WEEKS

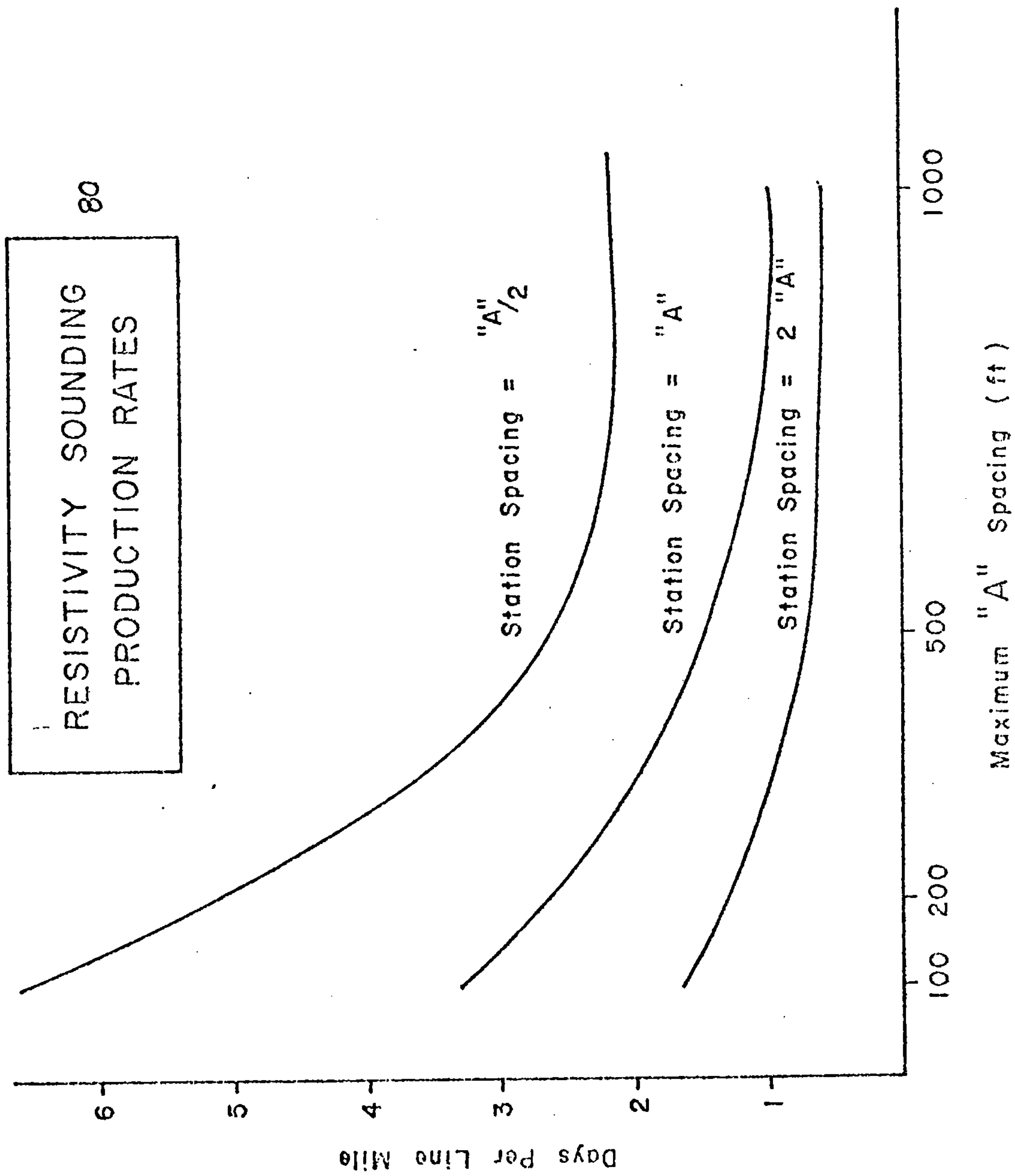
INDIRECT SAMPLING - SUB-BOTTOM REFLECTION (DASHED LINES)

63,000 LINEAR FEET OF SAMPLING LINE
<u>CONTINUOUS DATA</u>
ACTUAL SAMPLING 5-10% OF AREA
LARGEST UNSAMPLED AREA 7.8 ACRES
TIME FEW DAYS

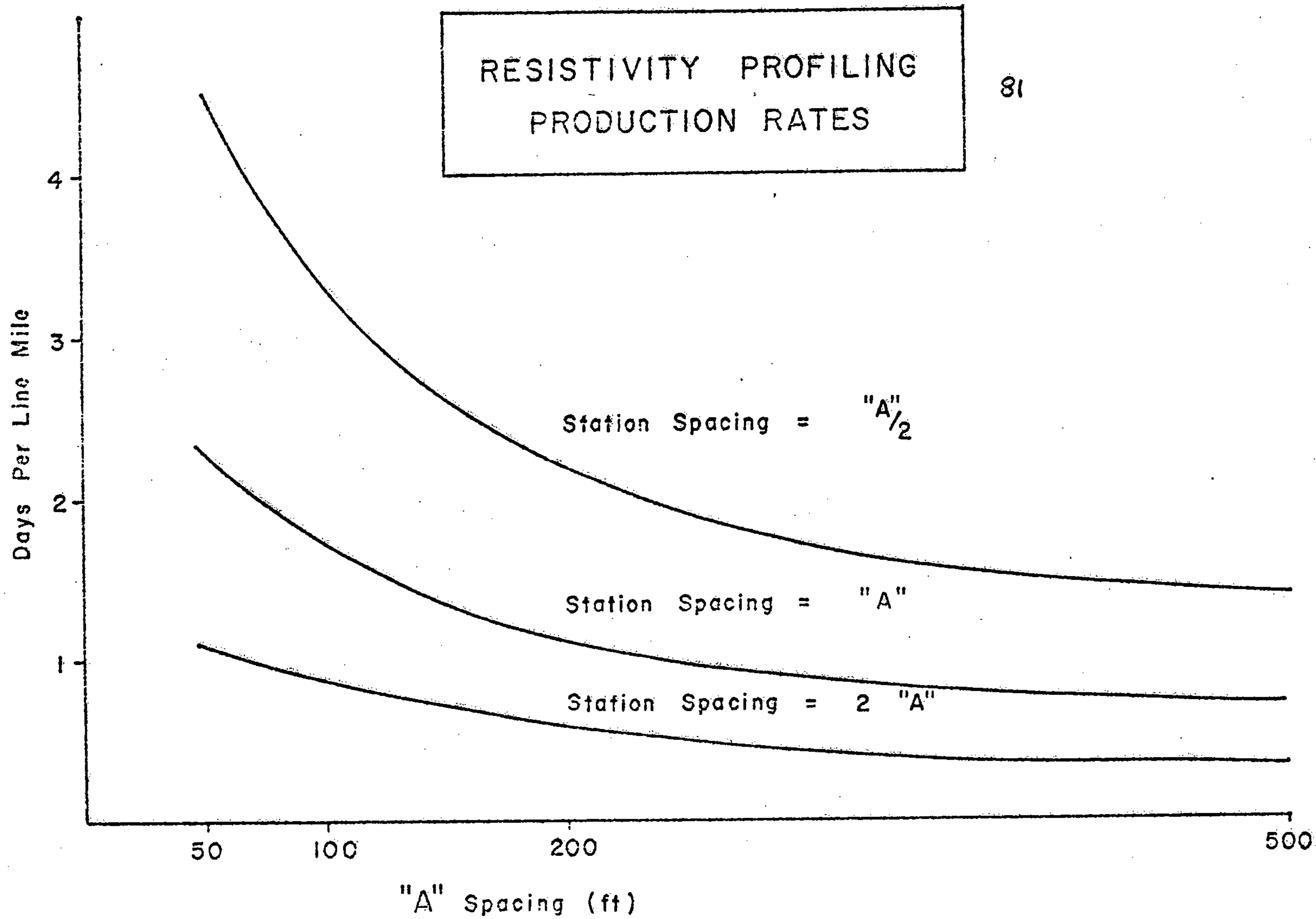

REFRACTION PRODUCTION RATES
78
50% Overlap
End to End
Double Space
Days Per Line Mile
1
2
3
4
5
6
100
200
500
1000
Seismic Line Length (ft)

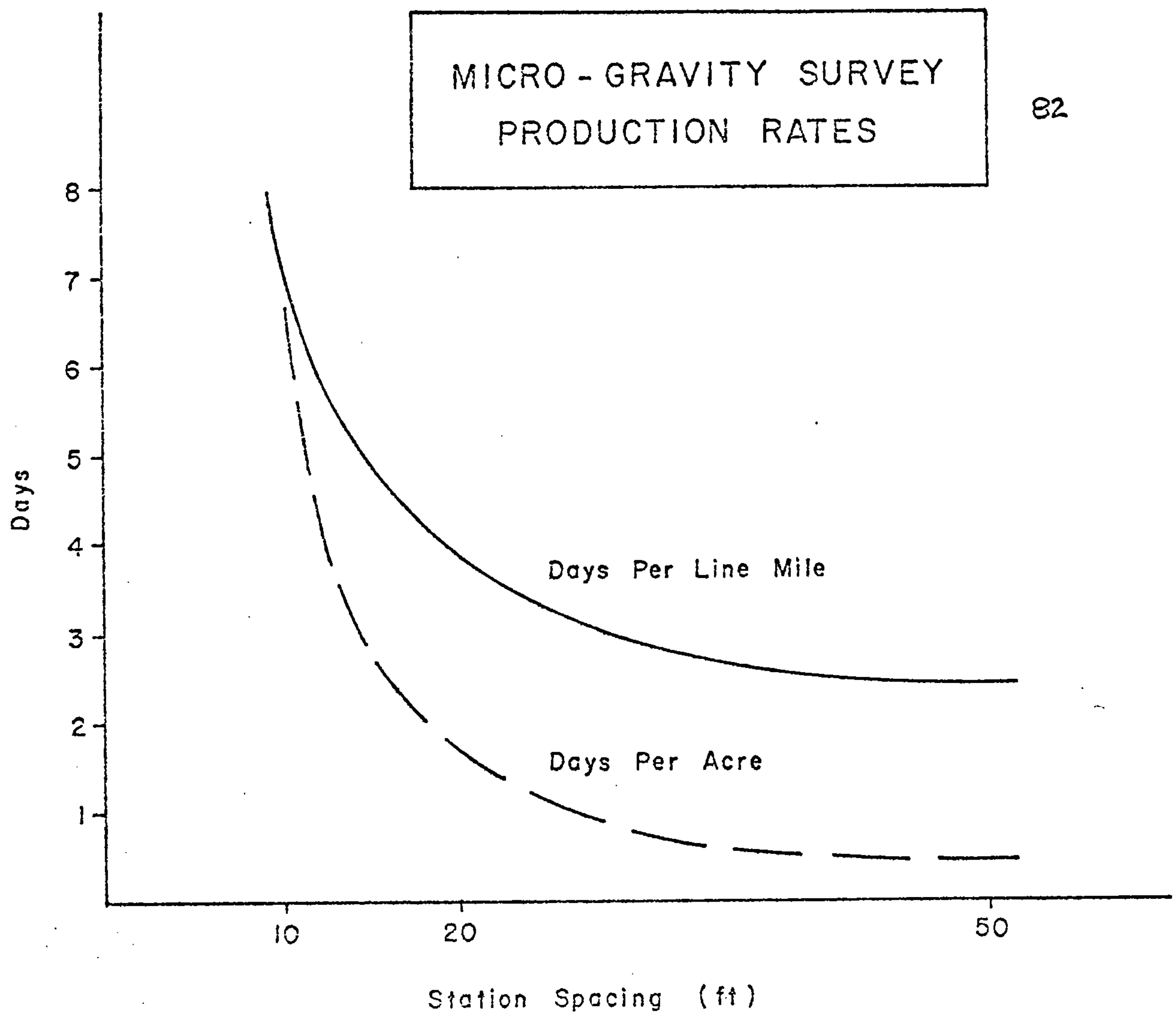

REFLECTION
PRODUCTION RATES
Days Per Line Mile
20
18
16
14
12
10
8
6
4
2
10
20
50
100
Station Spacing (ft)

79



RESISTIVITY SOUNDING PRODUCTION RATES
80
Station Spacing = "A"/2
Station Spacing = "A"
Station Spacing = 2 "A"
Days Per Line Mile
1
2
3
4
5
6
100
200
500
1000
Maximum "A" Spacing (ft)


RESISTIVITY PROFILING
PRODUCTION RATES
81
Station Spacing = "A"/2
Station Spacing = "A"
Station Spacing = 2 "A"
Days Per Line Mile
1
2
3
4
50
100
200
500
"A" Spacing (ft)


MICRO - GRAVITY SURVEY
PRODUCTION RATES
Days
8
7
6
5
4
3
2
1
Days Per Line Mile
Days Per Acre
10
20
50
Station Spacing (ft)

82

MARINE SURVEYS 83

INCLUDES

BATHYMETRY - SUBBOTTOM - SIDESCAN - MAGNETOMETER

SPEEDS 2 - 6 knots preferred (maximum 10-15)

PRODUCTION RATES 20 - 50 nautical miles / day

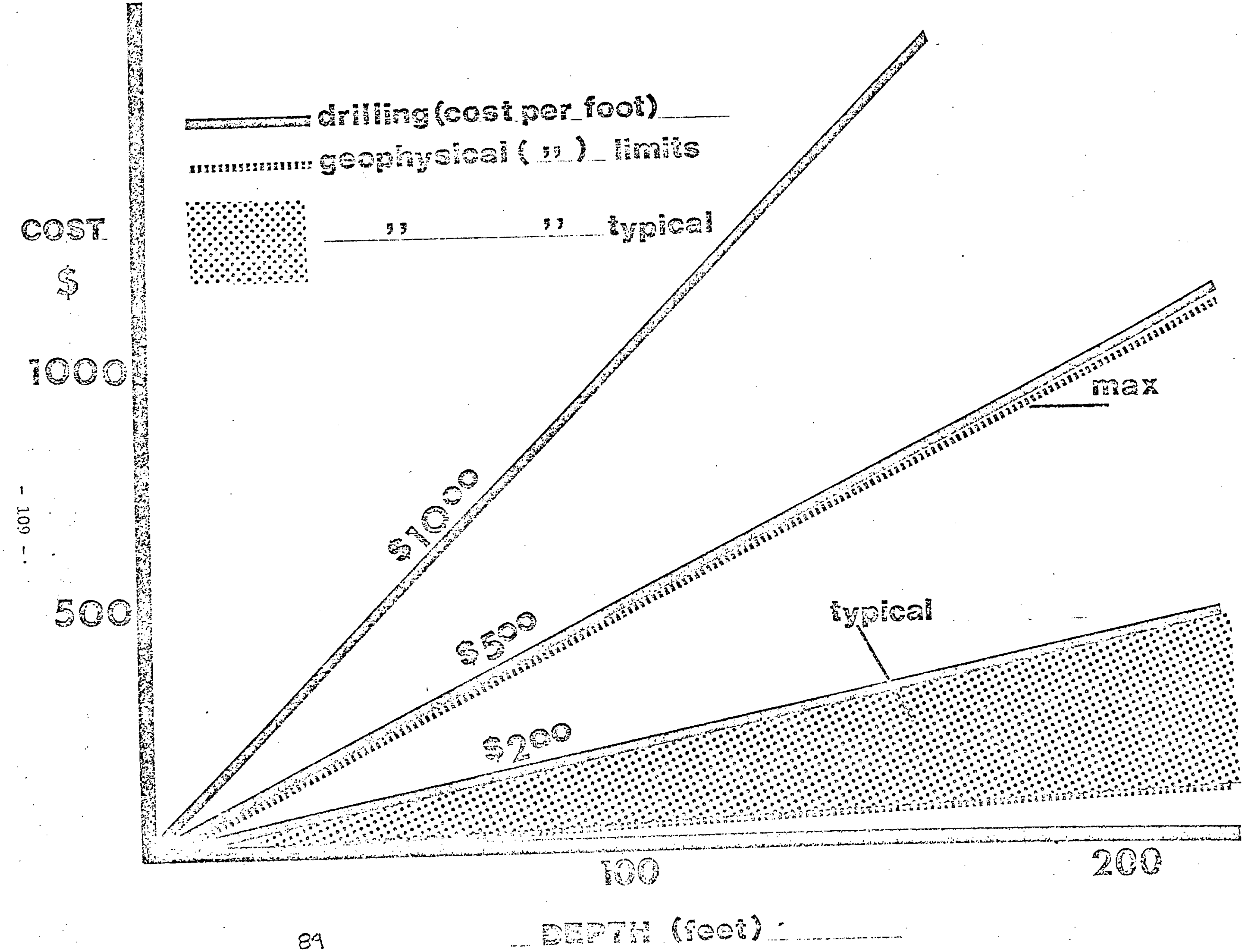
EXCEPTIONS Short survey lines and unidirectional runs
May yield - 5 - 10 nautical miles / day
Long continuous and bidirectional runs
May yield - upwards of 100 nautical miles/day

LANE SPACING

50'
100'
200'
500'

LINE NAUTICAL MILES/SQ. NAUTICAL MILES

121
61
31
13


drilling (cost per foot)
geophysical (") limits
" " typical
COST
$
1000
500
$10.00
$5.00
$2.00
max
typical
100
200
DEPTH (feet)

CURRICULUM VITAE

RICHARD C. BENSON(Dick)

EDUCATION

B. S. Exploration Geophysics, Minor Electrical Engineering, University of Minnesota.

Graduate Studies, Ocean Engineering including:

Coastal Engineering
Physical Oceanography
Materials & Marine Corrosion

Institute of Marine Science, University of Miami.

WORK EXPERIENCE

Past four (4) years, President and Chief Geophysicist of TECHNOS INC., a Miami-based firm, offering serrices in applied geophysics and earth sciences.

Four (4) years Assistant Professor and Department Chairman of the Marine Technology Department, Miami-Dade Community College. Founded one of the first programs of its kind in the nation.

A total of thirtheen (13) years with Honeywell. Most recently as a Systems and Research Engineer in Marine and Atmospheric Geophysics.

PRINCIPAL TECHNICAL INTERESTS

Application of geophysical techniques to engineering and other small scale problems such as search and archeology, including development of new instruments and techniques.

Have presented numerous papers and articles on Engineering Geophysical Methods, as well as other diverse subjects including technical education, corrosion, etc.

Have provided lectures on Geophysical Methods to Professional Groups and College Courses. Lectures also include Geophysical Techniques as applied to Marine Archeology - University of Miami.

MEMBER

Society of Exploration Geophysicists
Marine Technology Society
National Association of Corrosion Engineers
American Boat & Yacht Council

DRAINAGE PROBLEMS IN

CARBONATE Terrane OF

EAST TENNESSEE

By

Harry L. Moore

TENNESSEE DEPARTMENT OF TRANSPORTATION

BUREAU OF HIGHWAYS

DIVISION OF SOILS & GEOLOGICAL

ENGINEERING

1976

DRAINAGE PROBLEMS IN CARBONATE TERRANE OF EAST TENNESSEE

By

Harry L. Moore

Tennessee Department of Transportation
Bureau of Highways
Division of Soils & Geological Engineering

ABSTRACT

The involvement of geotechnical expertise in the design, construction, and maintenance programs of the Tennessee Department of Transportation, Bureau of Highways, has resulted in an expansion of knowledge dealing with drainage problems in carbonate terrain.

In order to provide a complete understanding of the phenomenon of "piping" (i.e. the development of solution cavities at the point of intersection with the surface), geological conditions must be evaluated. Specifics such as lithology type, jointing, faulting, strike, dip, type of weathering, geomorphic background, etc. must be considered.

The theoretical development of cave systems (Speleogenesis) can give insights on cause and effect relationships applicable to "piping" failures. A hypothetical model of the formation of "piping" structures is suggested as a possible guide to understanding the mechanical processes. Certain characteristics of piping failures can be observed from the numerous highway construction areas of East Tennessee.

Remedial concepts related to piping failures, although not mechanically intricate, must be individually contoured to meet the site conditions. A remedial model is conceptualized as a hypothetical standard with the realization that these are general recommendations and cannot be applied to every "piping" failure

Three case histories of "piping" related failures along East Tennessee highways are discussed with regard to development and remedial concepts. One case history involves construction and maintenance related failures while the other two case histories involve pre-construction design concepts.

It is evident the "piping" related failures are and will continue to be a geologic and geomorphic problem to highway construction and maintenance. Adequate measures have been adopted to remedy numerous "piping" failures in the East Tennessee area, but new and more expedient techniques and concepts are continually being searched. Treatment of possible, karst hazards in the design phase of highway engineering is essential economically, environmentally, and safety-wise for highway travelers.

Drainage Problems in Carbonate
Terrane of East Tennessee

INTRODUCTION

The involvement of geotechnical expertise in the design, construction, and maintenance programs of the Tennessee Department of Transportation, Bureau of Highways, has resulted in an expansion of knowledge dealing with drainage problems in carbonate terrain. Of particular interest in this discussion is the phenomenon of "piping", i.e. the development of depressions, sinkholes, or callapse features. The discussion will pertain to problem areas within the valley and ridge province of East Tennessee. The study has been broken down into two parts: (1) Theoretical cause and effect relationships (Speleogenesis) and (2) Actual on the job case histories.

The author wishes to express his appreciation to fellow workes Messrs. David Royster, Jim Aycock, and Joe Blackburn for their guidance and experience used in this paper; to Mr. Ken Brinkly for his excellent drafting of diagrams; and Mr. George Hornell for his photographic expertise used in the visual part of this discussion.

GENERALIZED GEOLOGIC SETTING

To provide a complete framework for our review of the "piping" phenomenon associated with highway construction and maintenance. a brief and general review of the geologic setting is necessary. The area of itensified highway construction relative to this discussion is found along broad hilly country valleys of the valley and ridge province of East Tennessee.

This province is geologically composed of the Cambrian, Ordovician, and Silurian systems of rocks. The lithologies vary from siltstones and sandstones to shales and various types of carbonate strata, ranging from dolomicrites to pelmatazoian sparites. The carbonate units consist approximately of 5700 feet (55%) of the stratigraphic section.

These strata are folded and faulted to form a complex structural network of lithologic dis-continuity. Tightly folded anticlines and synclines have been twisted and faulted to add to the complexity of the structural subsurface.

As a result of the weathering process, long ridges and valleys have developed paralled to the strike of the sedimentary strata. The more resistant units forming ridge crests and the less resistant carbonate strata forming broad hilly country valleys.

Due to the intense folding and faulting of the rock strata, numerous and complex sets of joints and/or cracks have developed providing access for groundwater activity by which solution enlargement takes place. Through the natural processes of weathering and groundwater flow, the carbonate strata have developed openings, conduits, or voids commonly interconnected to form complex systems of solution cavities. It is these solution cavities in the form of "piping" structures and in the geologic and geomorphic context just discussed, that our discussion will pertain.

PART 1

SPELEOGENESIS

A. HISTORIC WORKS

Due to the Scope and purpose of this discussion, only a brief statement of previous theoretical concepts pertaining to cave origins will be dealt with.

Most known theoretical ideas concerning cave origins are divided into three (3) conflicting groups: vadose, water table, and deep phreatic. For a complete in depth review of these theoretical ideas, the reader is referred to Ford, 1965.

Most workers now believe that a combination of the three (3) theories, mentioned above, may best explain the origin of caves. Present researchers discount the concept of percolation through interstitial openings because of the density and structural make-up of most carbonates (Moore, et. al., 1969). The percolation of groundwater through carbonate strata is thought by Davis, 1966, and Moore et. al.,1969,to begin through a network of tiny openings along joints and bedding planes. Davis (1966) points out that "the contrast in permeability between joints and limestone may be highly significant, even with joints closed to less than ten (10) microns".

It is important to understand that subsurface flow patterns are so variable that no one theory can be either correct or incorrect in an overall sense. As White and Longyear (1962) states, " The theories are irrelevant...each theory may be developed and supported in the field as a special consequence of a specific flow pattern." In this same light, it is proposed by White and Longyear (1962) that the threshold value (ie,opening at which turblent flow may first develop) for cave development is 5.0 mm.

In agreement with Barr (1961) the author has decided to employ the terms ground-water table, vadose and phreatic zones with the realization that the water table in carbonate strata is perhaps not strictly comparable to that of porous rocks, and that these zones may vary indiscriminately in response to the surface conditions.

It must be pointed out that while most studies in speleogenesis deal with the development of cave systems, a direct relationship exists between "piping" failures and cave development. It is the development of a cavern system, at the stage of intersection with the surface, in which man becomes directly involved with the process of maintaining the equilibrium between the surface and subsurface ecosystems.

To bring the groundwater concepts into perspective with engineering geology the reader is referred to works pertaining to landuse problems relative to groundwater discharge and equilibrum disruption in the environment. Such works by Parizek, 1970, and Lattman and Parizek, 1964, may be referred to.

B. THEORETICAL DEVELOPMENT OF PIPING STRUCTURES

The development of caverns, solution cavities, etc. by solution, dissolution, corrasion, and abbrasion is an intricate key to the development of "piping" structures common along the highways of East Tennessee. In a complex relationship between the processes of surface erosion, water table migration, and geochemical processes, there is the phenomenon of solution cavity enlargement. This enlargement is commonly in response to gravity with the major emphasis of solution in a vertical element with a lateral dimension. In areas where this enlargement of solution conduits is found, solution encroachment from the bedrock to the surface commonly results in numerous karst topographic features such as dolines (sinkholes), ponors (sinkholes into which surface streams disappear), , surface depressions and numerous limestone outcrops.

In the construction of highways with minimum grades and low degrees of vertical and horizontal curvature, there are frequently deep large cuts into residual clays and bedrock with corresponding high, thick fill intervals required (Fig. 1). In some cases, these excavations bring closer together the intersection of the subterranean voids with the surface. In response to these excessive cut and fill intervals, the natural ground-water and soil-water regimes regain a status of equilibrum. With the activity of solution encroachment toward the surface and a regaining of ground-soil water equilibrium, conduits within the bedrock commonly intrude the residual soil cover forming voids within the residuum. During periods of excessive precipitation and increased channelization (ie. roadway ditches), soil-water seepage pressures increase and migrate toward areas of less pressures; commonly voids in the subsurface (Fig. 2).

Settlement of the clay particles toward the area of less seepage pressure in response to the flow of water, will occur. When this settlement occurs, partly due to the increased weight of the soil mass and partly due to the flow of water in response to gravity, to a degree of "collapse" from the surface downward, a piping failure has resulted (Fig. 3).

After several seasons of construction problems related to "piping" failures one can begin to recognize characteristic features associated with the phenomenon of "piping". In the East Tennessee area, at least twelve such characteristics have been observed. These are listed in Table 1. It should be made clear that no one characteristic is diagnostic but any combination of characteristics should be expected.

C. THEORETICAL REMEDIAL CONCEPTS

As in most other disiplines, the use of geotechnical expertise in the repair of geologically related roadway failures involves the cooperation and synthesis of knowledge between the geologist and highway designer and construction engineer. Not to do so would only result in further failures.

Remedial concepts related to piping failures, although not mechanically intricate, must be individually contoured to meet the specific conditions. Because such is the case, blanket recommendations involving remedial measures cannot be made.

To alleviate semantic problems the following remedial concepts are suggested and will be discussed with the understanding that these are general concepts and cannot be applied to all "piping" failures. This procedure was first adopted by Mr. Jim Aycock, Geologist III, of the "Division" in his work dealing with sinkholes developing in the subgrade of I-81, Sullivan County, Tennessee, in 1971.

The following discussion outlines the present "general" recommendations (Fig. 4):

1. Removal of the "failed residual soil is usually the first procedure involved in correction of "piping" failures. The soil removal process proceeds downward until bedrock, pinnacles, and/or a solution cavity is reached. Pinnacles of carbonate strata are usually removed to prevent seepage pressures from being focused on the pinnacle itself and to provide an even, plain surface with which to "choke" the failure.

2. Backfilling of the excavated area proceeds with clean limestone "chunk" rock. Rock specifications for backfill vary depending upon the site conditions (size of failure, aperature size of solution cavity, drainage of surrounding topography, etc.). A "choke pad: is constructed on top of the "chunk" stone. This choke pad varies from 18 inches to 3 feet in thickness and is composed of small rock (No. 2 stone, #57 stone, and/or crusher run material). The choke stone pad is designed to prevent stoping of the residual clay soils both by downward seepage pressure forces and upward movement by groundwater.

3. Placement of 2 to 3 feet of residual clay soil over the backfilled area is usually recommended: A - to provide somewhat of a seal from the surface; B - in some instances to provide foundation for paved ditches or roadway subgrades; and C - to dress-up backslope areas for seeding.

In most instances additional design is required to provide for increased groundwater drainage by placement of french type drainage conduits connecting affected areas to free draining areas. Other design concepts such as installation of vertical perforated pipes, direct channelization into subterranean streams, and gratings and horizontal drains and pumps may be used depending upon the site conditions. In most all cases, the piping failures seem to result from ponding of water along the ditchline for a prolonged time (where time is variable). As a remedial concept paved ditching is a necessity and must be adopted in areas prone to solution activity although present design standards (per cent of grade) prohibit paved ditches.

PART II

CASE HISTORIES

In discussions such as these, one can become bogged down in theoretical innvendo and lose sight of the objective--actual job situations. Direct application of theoretical design concepts (with necessary tailoring) results in additional knowledge for future situations and a basis with which to make wise conceptual and applicable decisions.

The following case histories (discussed very briefly) are but a few of the "piping" problems dealt with by the Division of Soils and Geological Engineering staff. These case histories are all located in the East Tennessee area between Knoxville and Johnson City. For discussion purposes, the case histories will be broken down into (I) construction or maintenance related problems and (II) preconstruction design concepts.

I. Construction or maintenance related problems

(A) I-40, Sevier County.

Date: Late winter of 1975

Location: Ditchline and subgrade roadway W.B.L.
just west of S.R. #66 Interchange.

Formation: Knox Group

Geologist: Jim Aycock

Discussion:

After bringing the roadway cut to subgrade elevation and ditchline construction was completed, winter precipitation resulted in the channelization of run-off within the new ditchline. Seepage pressures built up around rock pinnacles and resulted in piping failures along the ditchline and into the roadway subgrade. Excavation of "failed" material yielded a solution cavity and dolostone pinnacles. Backfilling with clean chunk limestone larger than the solution cavity opening was recommended to within 5 feet of the subgrade. A 2' thick choke stone pad was placed on top of the chunk rock. The remainder was filled with residual clay soil and compacted. A concrete drainage ditch was recommended along the entire cut ditchline to prevent any future seepage pressure build-up and failure.

II. Pre-Construction design concepts

(A) 11-W relocation, near Surgionsville

Date: Design: Fall of 1973

Construction: Spring and Summer of 1975

Location: Relocation of 11-W from Sta. 714+00
to Sta. 720+00, Hawkins Co.

Formation: Upper Conasauga Group

Geologist: Harry Moore

Discussion:

In preparing a complete soils and geological report for design and construction purposes, numerous sinkholes and depressions were found (Fig. 5). Special design concepts, employing the use of limestone "chunk" rock, were adopted to treat the "karst" areas. The following design concepts were adopted and employed during construction (Fig. 6).

1. Clean all free draining sinkholes of debris.

2. Backfill with specified rock and carry rock backfill as a pad to fill edge.

3. Place a "choker" pad of No. 57 stone on top of the rock pad to prevent stoping of fill fines.

4. Construction of French type drainage ditches from sinkholes to paved ditch at toe of fill (to act as a "bleeder").

A total of 10 sinkholes and depressions were treated in this manner along the construction project. Rains since the construction of these rock pads in Spring of 1975 has resulted in free flow drainage of water "pumped" up out of the sinks (by ground-water pressures) and into the prepared French drains and into the paved drainage ditches.

(B) Cocke County--S.R. #35 relocation
Date: Design: Late summer and fall 1973
Location: S.R. #35 near Sta. 266+00 of new relocation
Geologist: David Royster
Discussion:

A roadway fill was designed over a large sinkhole which required special design concepts by our division (Fig. 7). An intensified boring program was carried out to delineate the soil-rock interface. The sinkhole is located in the Knox Group (undivided) and is part of a series of solution cavities and depressions in that area that covers several acres. The large sinkhole ponds water after moderate to heavy rains and eventually drains in several days. The sink structure seemed to be the result of solution along a joint parallel to strike. It was discovered during drilling operations that the deepest accumulation of sediment and debris and closest point to the actual subsurface solution cavity appears to be along both left and right of the interval Sta. 265+70 to Sta. 266+00 adjacent to centerline (Fig. 8).

Due to the fill height, size of drainage area, size of sinkhole, and inadequate draining facilities it was decided to install a system of vertical perforated drainage wells (Fig. 9, 10). It was decided that the holes for the vertical wells be drilled "dry"; that is, with a large auger, or with a percussion drill to reduce the possibility of silting or altering the drainage channels that have already been developed in the underlying rock and along the residuum-rock interface.

It was felt that this procedure would provide drainage or run-off, to the soil-rock interface. The combination of the free draining rock pad and vertical perforated drainage wells should alleviate any major roadway due to the karst features in the area.

SUMMARY

In summary, it is evident that "piping" related failures are and will continue to be a geologic and geomorphic problem to highway construction and maintenance. Adequate measures have been adopted to remedy numerous "piping" failures in the East Tennessee area, but new and more expedient techniques and concepts are continually being searched. Treatment of possible karst hazards in the design phase of highway engineering is essential economically, environmentally, and safety-wise for highway travelers.

TABLE 1

CHARACTERISTICS OF "PIPING" FAILURES

After several seasons of construction problems related to "piping" failures one can begin to recognize characteristic features of the phenomenon of "piping". In East Tennessee at least twelve (12) such characteristics have been observed. These are:

(1) Piping failures usually occur after alteration in area drainage patterns due to

- (A) Excavation
- (B) Rerouting, Concentration, and funnelling of run-off.
- (C) Adversely altered erosion-siltation patterns.

(2) Piping failures usually have pinnacles of carbonate strata in area of "failure".

(3) Very moist to very plastic residual clay soils commonly found in failed area.

(4) A large number of piping failures were observed to have actual "open-space" connections from the surface to subterranian solution cavities.

(5) Numerous "piping" failures are commonly located along excavated ditchlines that are not paved.

(6) Most failures occur as a circular-type depression with tensional cracks commonly found around the failed area.

(7) Piping failures seem to usally occur after moderate to intense amounts of precipitation.

(8) Piping failures occur in areas of excessive residual excavation where proposed sub-grade is nearing the rock pinnacle--residual contact:

(9) Piping failures are related to lowering of the immediate local "water table".

(10) The degree of strike and dip of rock strata does not seem to influence piping failures.

(11) The majority of piping failures have been in the residual soils of the Knox Group and Holston Formation.

(12) The prediction of where piping structures may occur

(exact locations) more specifically than a given station interval (within a geologic formation) is still somewhat arbitrary.

It should be made clear that no one characteristic is diagnostic but any combination of characteristics should be expected.

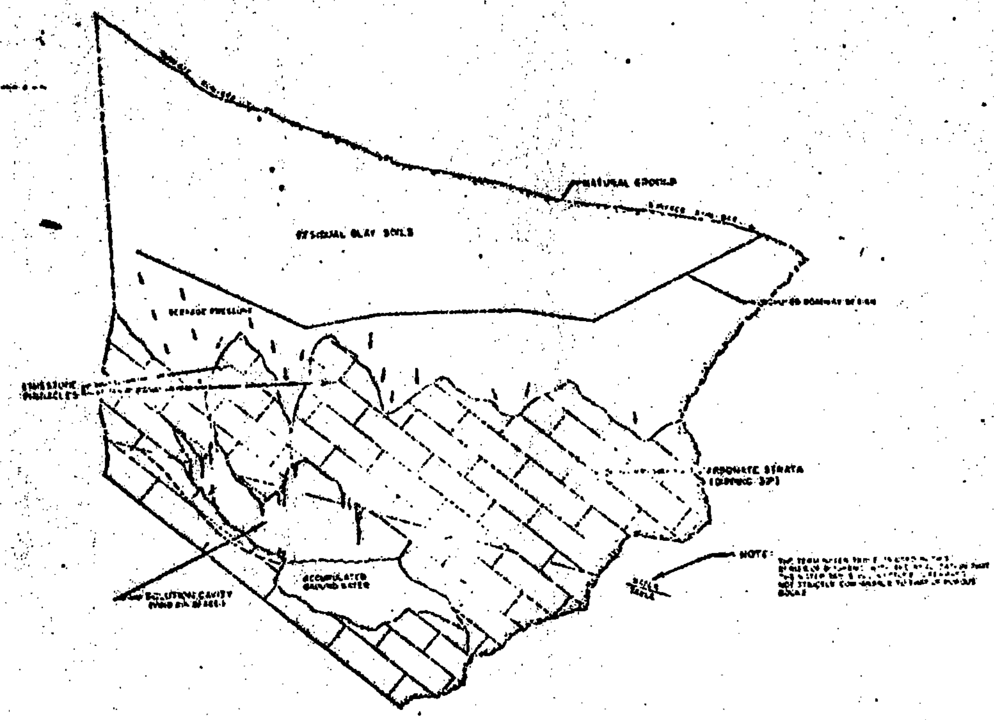


FIG. I: HYPOTHETICAL ROADWAY TEMPLATE IN CUT SITUATION

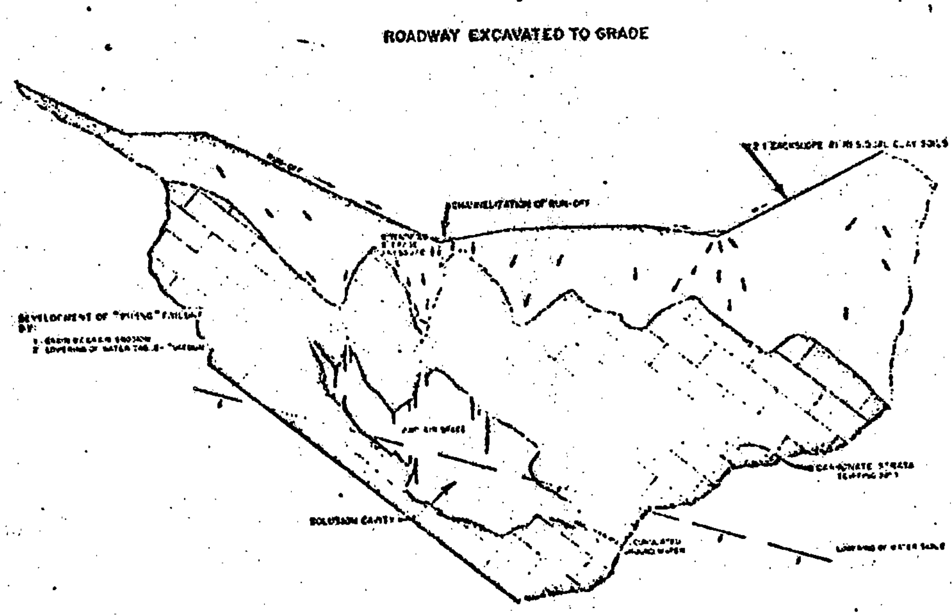


FIG. II: ROADWAY EXCAVATED TO GRADE

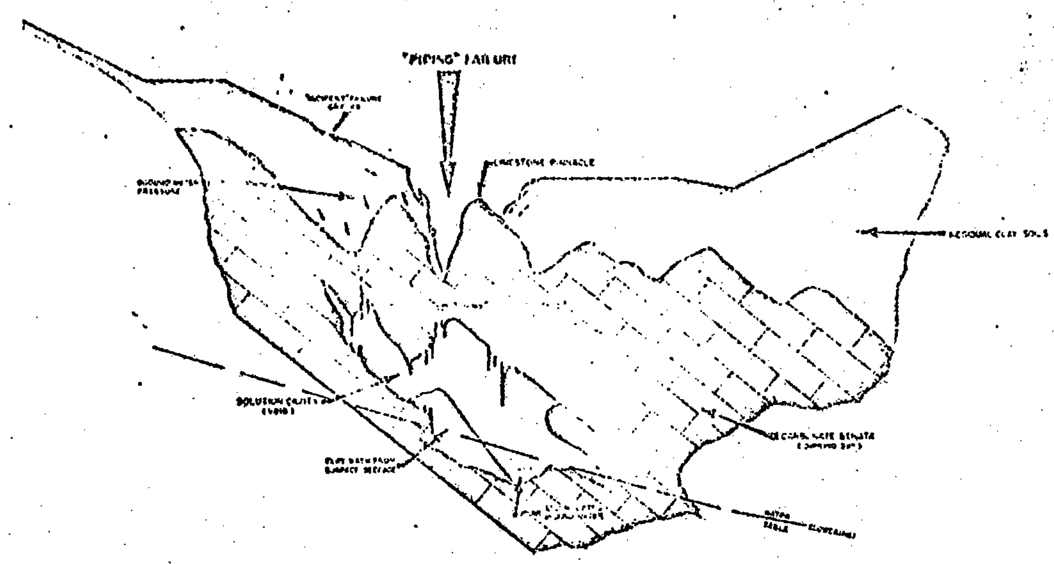


FIG. 3: HYPOTHETICAL PIPING FAILURE IN ROADWAY SUBGRADE

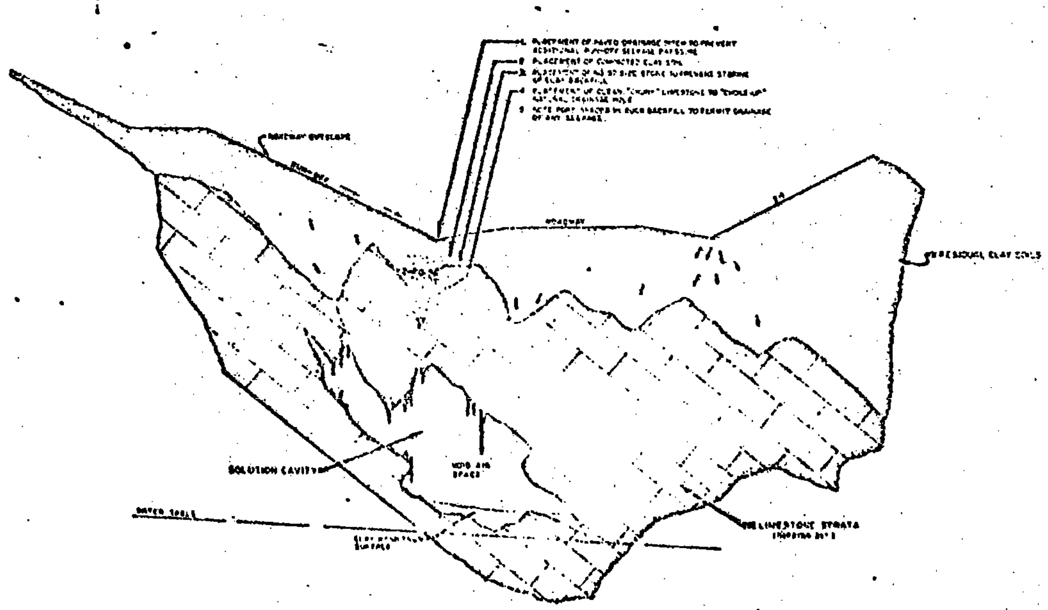


FIG. 4: GENERAL REMEDIAL CONCEPT FOR PIPING FAILURES

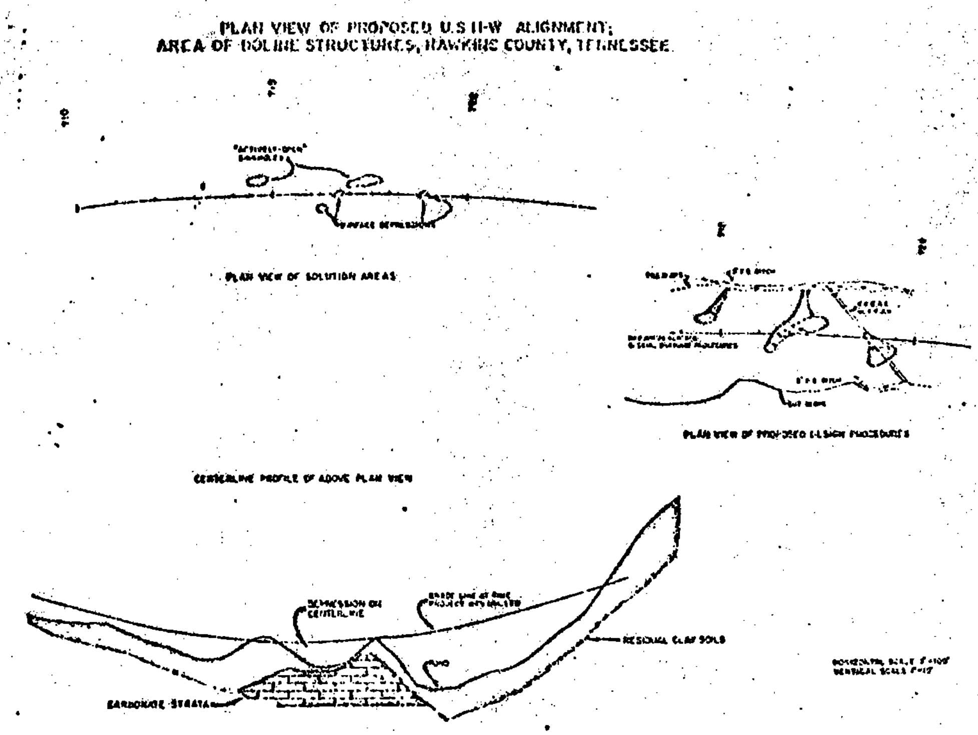


FIG. 5: PLANVIEW AND PROFILE OF U.S. 11-W, DOLINE AREA, HAWKINS COUNTY, TENNESSEE

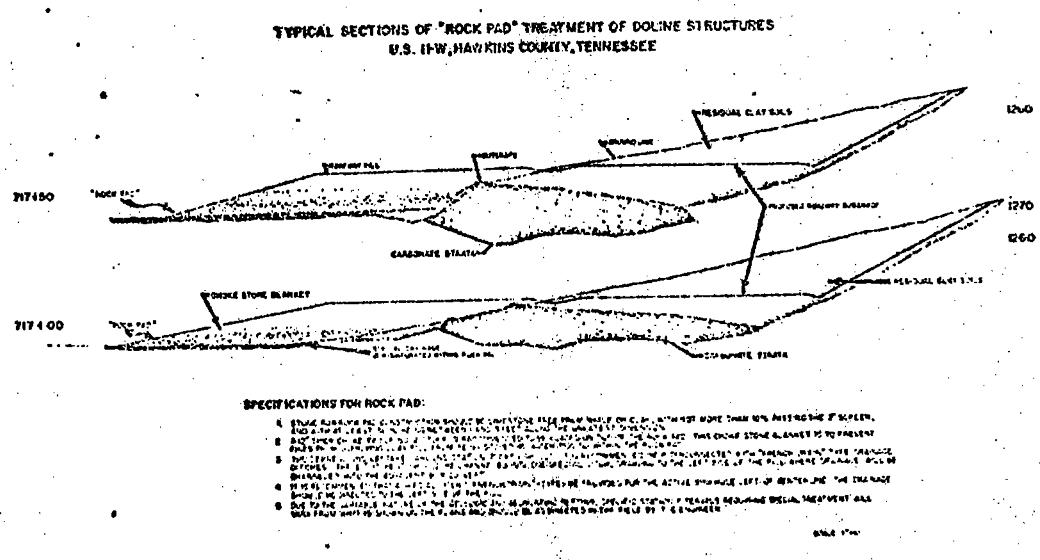


FIG. 6: TYPICAL SECTIONS OF "ROCK PAD" TREATMENT OF DOLINE STRUCTURES; U.S. 11-W, HAWKINS COUNTY, TENNESSEE

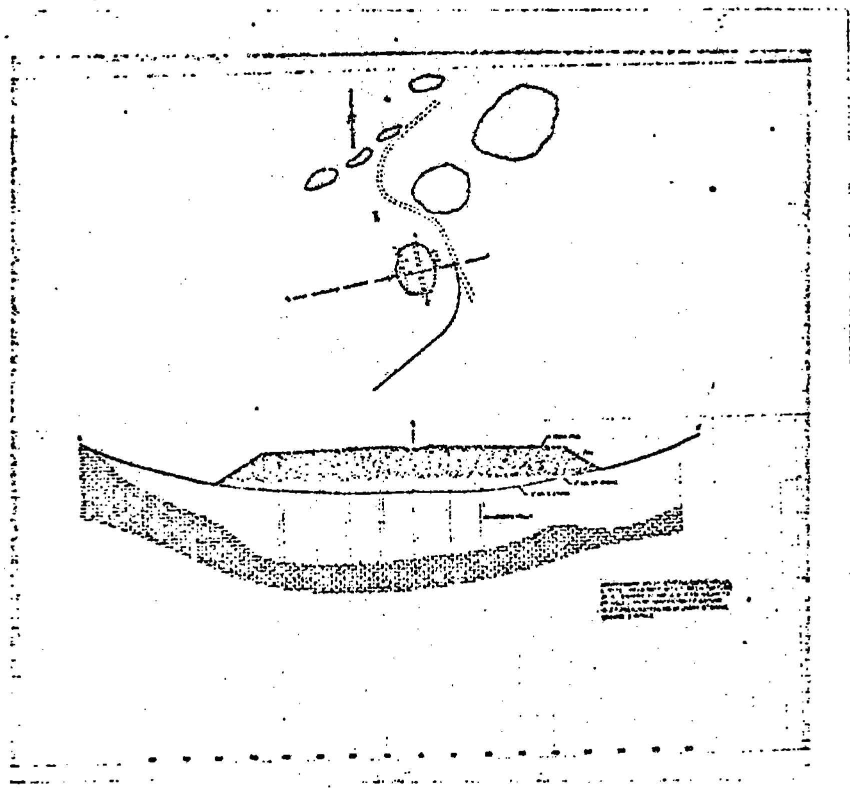


FIG. 7: PLANVIEW AND PROFILE OF SINKHOLE AREA, S.R. #35, COCKE COUNTY, TENNESSEE.

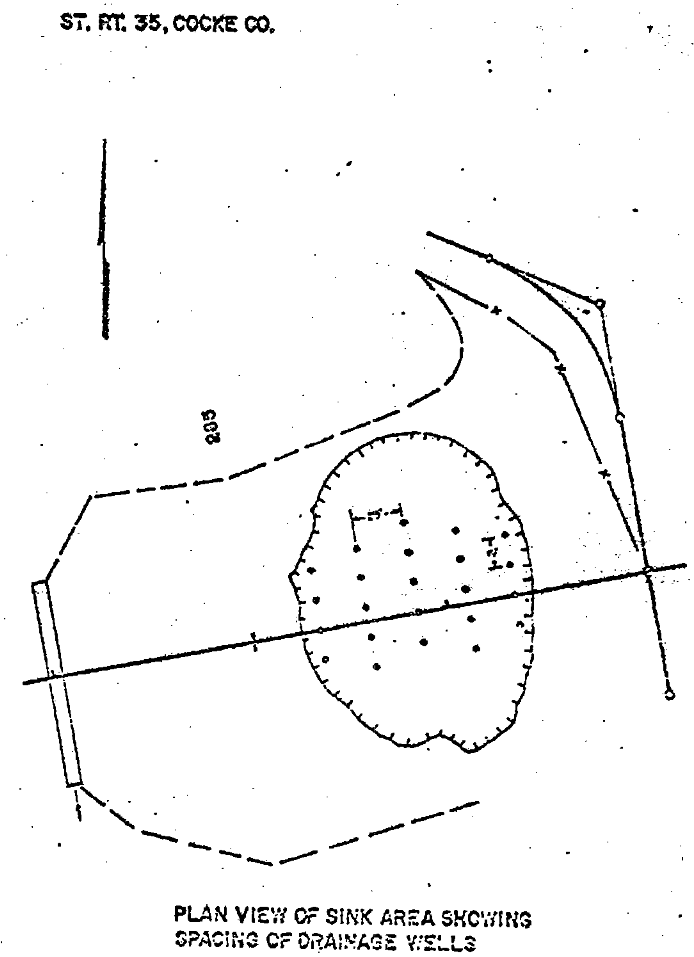


FIG. 8: PLANVIEW OF AFFECTED SINKHOLE, S.R. #35, COCKE COUNTY, TENNESSEE.

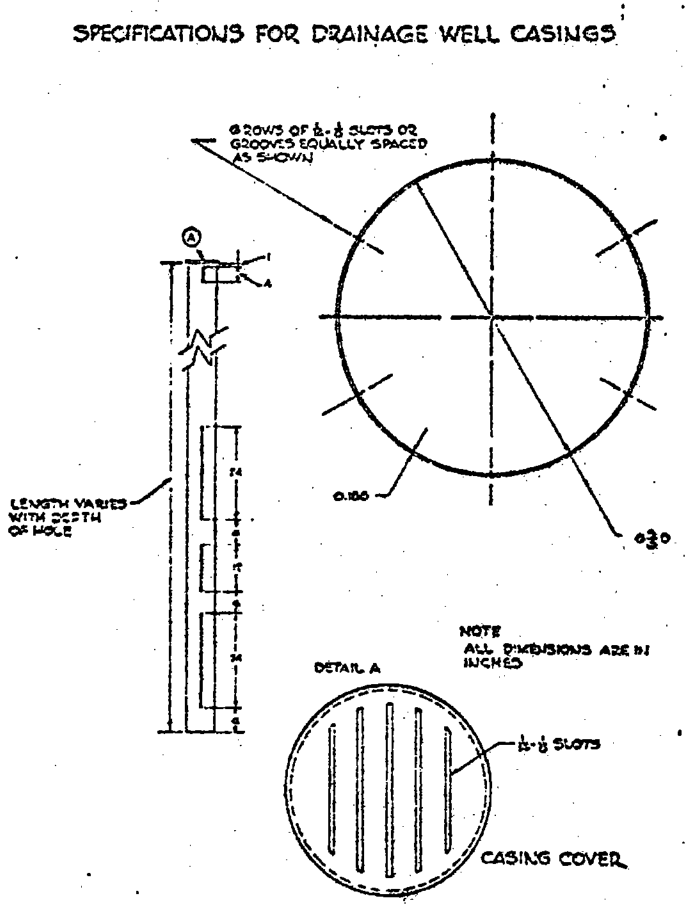


FIG. 9: SPECIFICATIONS FOR SINKHOLE DRAINAGE CASINGS.

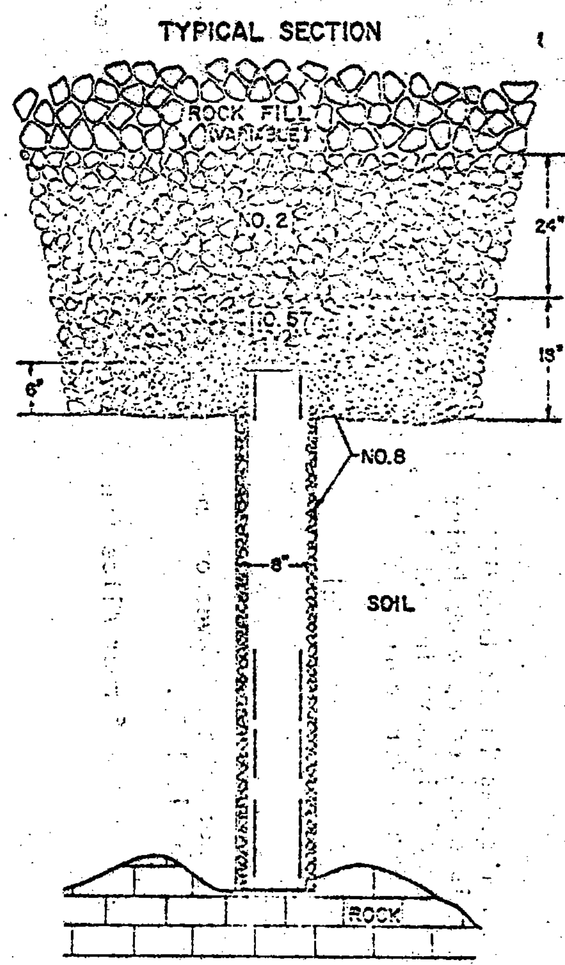


FIG. 10: TYPICAL SECTIONS OF BACKFILL AND ROCK FILL PROCEDURE IN SINKHOLE AREA; S.R. #35, COCKE COUNTY, TENNESSEE.

BIOGRAPHICAL SKETCH

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LAPIES-TYPE FEATURES IN THE KENTUCKY KARST REGION

By

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University of Kentucky
Lexington, Kentucky

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ABSTRACT

Lapies-type features are thought to represent a common but little known and rarely exposed geomorphic development in the karst region of Kentucky and neighboring areas. An understanding of the occurrence and distribution of irregularly dissolved upper surfaces of limestone strata should be of interest not only to the student of underground drainage phenomena but also to those involved in construction activities and the removal of earth materials in limestone terranes. Failure to recognize the existence and distribution of lapies-type surfaces could result in unexpected and unplanned excavation costs or inadequate foundation conditions for anticipated structures.

INTRODUCTION

The karst landscapes of Kentucky are well known and have attracted the attention of geologists for may years. Much has been published about the classical karst region of the Commonwealth especially the development of subterranean solution features. However, little attention has been given to the irregular, furrowed, grooved, corroded, and otherwise dissolved upper surfaces of exposed limestone referred to as lapies.

Lapies features have been described in American and European literature for many years. Cvijic (1924), in his classical paper on lapies, reported that it had been classified as a karst form as early as 1893, but to the present writer's knowledge the term designating these solution features and the application of the recognition of their existence to land-use problems have not previously been recorded in the geologic literature of Kentucky.

Opportunities to view exposed upper surfaces of the etched and weathered limestone strata are usually quite limited in areas of low relief and humid climate. A cover of red, residual clay blankets most of the limestone terrane of Kentucky, varying in thickness from a few inches (5 to 10 cm) to tens of feet (10 m or more). Because of this residual mantle, good examples of limestone surfaces are relatively rare, and recourse generally must be sought in man-made excavations in order to view the detailed characteristics. Therefore, one must be alert to problems relating to construction and excavation in areas of variable thickness of soil cover and irregular and uneven subjacent bedrock surfaces.

Cvijic (1924), in describing lapies in the Dalmatian karst region, maintained that lapies is found chiefly on outcrops of bare, inclined rock and that it was rare on horizontal rocks, being replaced there by sinkholes. In Kentucky, lapies-like features have been developed by differential solution of relatively pure, massive-appearing, flat-lying carbonate rocks beneath a soil cover. Jointing of the stone is an important aspect of the development of lapies. Meteoric waters, charged with inorganic and organic acids, percolating through the soil cover, are concentrated along pre-established fractures and grooves, enlarging them many times and forming the individual features which characterize the lapies surfaces. Lapies forming beneath an overburden of soil generally exhibit rounded features. When exposed, the upper surfaces may be sculptured by moving water into pinnacles, grooves, and other less subdued forms. The geologic process forming and modifying the upper surfaces of limestone strata is a continuing one. It is possible in areas of moderate to heavy annual rainfall that lapies features may be so altered in time that they will become inconspicuous, or even largely destroyed.

EXAMPLES

Although generally hidden, lapies-like features are believed to be common and of widespread occurrence in Kentucky. Striking development has been observed in the Ste. Genevieve, Girkin, and Monteagle Limestones of Late Mississippian age and the Jeffersonville Limestone of Devonian age. Similar expressions of this karst feature have been observed in Silurian and Middle Ordovician carbonate rocks.

An excellent example of lapies developed in the Ste. Genevieve Limestone is present in southern Todd County (Fig. 1). The locality is on the north side of Kentucky Highway 102, approximately 1.5 miles (2.4 km) southeast of Elkton, the county seat of Todd County. Soil generally mantles the karst plain in western Kentucky, but at this site the soil was removed, apparently for fill for the adjacent highway. After the surface soil had been removed artificially, rainfall and off-flowing waters removed some of the remaining residual clay and rock fragments which filled joints, cracks, and other openings and revealed a ragged, irregular rock surface with local relief of 2 to 3 feet (0.6 to 0.9 m). The residual material lying between and around the bedrock exposures is a red clay, tough and sticky when wet, with occasional silicified fossils and chert fragments. The thickness of the residual clay remaining between the irregular rock masses is probably as great as that already naturally removed.

Another excellent example of lapies was observed in the Jeffersonville Limestone (Devonian) capping a now-abandoned quarry in the St. Matthews area of Jefferson County (Fig. 2). Soil-removal operations prior to quarrying revealed an irregular, uneven rock surface with local relief of 6 feet (1.8 m) or more along the enlarged and deepened joints. A number of years ago a developer contracted to build a number of homes, each with a basement, in an area of Jeffersonville Limestone outcrop in Jefferson County. His probings of the site indicated to him that his excavations would be in soil. After construction started, it became evident that his probings had been along clay-filled solution fractures of a lapies-like surface and that much solid limestone had to be removed to construct the basements. It is this writer's understanding that the transaction was a financial disaster.

Exposures of solution-enlarged, clay-filled joints in carbonate rocks may provide clues to the presence of lapies-type surfaces (Figs. 3 and 4). Although very little of the upper surface may be exposed in these instances, it doesn't take much imagination to visualize the configuration of the upper rock surfaces if the soil cover were removed.

In the central Blue Grass region, lapies-type development was revealed by the mining of phosphate from residuum overlying the Ordovician Woodburn Limestone (equivalent to the Tanglewood Member of the Lexington Limestone) near Wallace, Woodford County, more than a half century ago. In mining parlance, the ridges and troughs were referred to as "horses" and "cutters," but they form a type of irregular bedrock surface similar to that referred to as lapies.

Lapies features are not restricted to the previously listed Kentucky formations; they may be found in any massive, well-jointed limestone beds. For example, in the building-stone district of south-central Indiana, the Salem Limestone (Mississippian) presents striking lapies development (McGrain, 1948). Where this formation exhibits a thick-bedded facies in Kentucky, such as is found locally in the Elizabethtown area, a similar irregular and uneven upper surface can be expected.

Another example of the type of problem that might be caused by lapies features was encountered in the quarry that supplied the stone for concrete aggregate for Kentucky Dam on the Tennessee River in western Kentucky. The quarry, now abandoned, was in the Mississippian Warsaw Limestone, situated on the north bank of the Tennessee River at the edge of the Mississippi Embayment region. The following account is given in a published report (Tennessee Valley Authority, 1949, p. 527-528).

"The quarry, after it had started to produce rock, was operated with only very minor difficulties. Its development was seriously handicapped by the unexpected amount of overburden. This overburden was a mixture of clay and badly channeled rock, and its removal required expensive mixed excavation. The presence of rock pinnacles prevented the free use of scrapers in removing the clay; also the presence of considerable amounts of clay in deep vertical channels within large masses of sound rock made it necessary to waste large amounts of good rock in order to dispose of the clay. The stiffness of the clay in these channels made sluicing almost impossible. The total average thickness of the rock and clay overburden over the quarry was 50 feet much greater than that indicated from the exploratory drilling."

SUMMARY

Field observations by the writer in the limestone terrane of Kentucky indicate that lapies-type development is greater than has been generally recognized and reported. The karst feature is, or should be, of more than academic interest. Those concerned with construction or the removal of earth materials in limestone terranes should be aware of the possible irregularities and variations which might be encountered on bedrock surfaces. Vertical, clay-filled solution openings may pose problems in quarrying operations because clay is deleterious to concrete aggregate, the stone adjacent to solution features may be soft or stained as the result of water action, and cracks and solution openings prevent the excavation of sound and uniform blocks of building stone. Contractors, developers, engineers,and others engaged in housing, pipeline, sewer line, underground cable, and similar projects in areas of outcrop of the geologic formations cited previously should be particularly alert. They should be knowledgeable that this phenomenon can and does exist in carbonate rock terranes; they should recognize that lapies-like features can be present in certain stratigraphic formations and in specific type lithologies and be alert to clues as to their possible presence. This is an example of applied geomorphology.

Except where man-made exposures are available, few, if any, clues may be present to warn one of these conditions. New 1:24,000-scale geologic maps, which are available for most of the karst areas of Kentucky, should be useful "tools" because they delineate the boundaries of many of the stratigraphic units that may contain lapies-like features; some of them show the strike of prominent joints. A limited or improperly planned probing program could fail to recognize the existence of these irregular surfaces and cause one to infer a greater or lesser amount of soil cover than is actually present. This, in turn, could result in unexpected or unplanned excavation costs, or inadequate foundation conditions for anticipated structures.

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CAPTIONS FOR ILLUSTRATIONS

Figure 1. Lapies in Mississippian Ste. Genevieve Limestone adjacent to Kentucky Highway 102, southeast of Elkton, Todd County, Kentucky.

Figure 2. Lapies in Devonian Jeffersonville Limestone capping an abandoned quarry at the junction of Brownsboro Road and Hubbards Lane, St. Matthews, Jefferson County, Kentucky. Maximum relief on the bedrock surface is approximately 6.5 feet (2 m).

Figure 3. Solution-enlarged, clay-filled joints in Mississippian Girkin Limestone, U. S. Highway 231, northwest of Bowling Green, Warren County, Kentucky. Depth of the fractures at this point is 10 feet (3 m) or more. Such exposures provide clues to the presence of lapies-type surfaces.

Figure 4. Solution-enlarged, clay-filled joints in Silurian Laurel Dolomite, Blue Grass Parkway, west of Bardstown, Nelson County, Kentucky. Solution features such as this can greatly diminish the probable reserve of a limestone deposit and can cause quarrying problems.

CURRICULUM VITAE

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UNDERGROUND CONSTRUCTION IN THE DEVELOPMENT OF TRAILS IN A LARGE CAVERN SYSTEM IN THE SOUTHERN OZARK MOUNTAINS KARST TERRAIN

By

Don E. Williams

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Forest Service

UNDERGROUND CONSTRUCTION IN THE DEVELOPMENT OF TRAILS IN A LARGE CAVERN SYSTEM IN THE SOUTHERN OZARK MOUNTAINS KARST TERRAIN

Don E. Williams

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Forest Service

The U. S. Forest Service is in the process of developing a large cavern system for the tourist use on National Forest land in northern Arkansas. The site, known as Blanchard Springs Caverns, is within a recreational complex and is partially completed.

The beauty of this cavern system is unsurpassed and can compare favorably with other major cavern developments of the world.

The presentation will depict the unusual problems encountered in construction of walking trails and access tunnels. The difficulties involved with materials handling, special safety considerations and unusual construction limitations will be shown and discussed.

"This was basically a slide presentation - there is no Report".

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SHALE DETERIORATION RELATED TO HIGHWAY EMBANKMENT PERFORMANCE

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Problem

1. Construction of the interstate highway system has required high embankments using economically available shale* and rock from adjacent cuts. Settlements of one to three feet occurring in the middle of shale embankments over several years have required frequent overlaying to maintain grade (Figure 1). Raising of bridge abutments founded on approach embankments is also required. Continuing settlements are followed by slides in some embankments while in others the settlement stops and no further distress occurs. Slides in small embankments are often repaired by state forces using wooden pile and guard rail retaining structures (Figure 2) until berms can be added (Figure 3). Slides in large embankments require expensive reconstruction costing $2 million in one case[1].

2. The underlying cause appears to be deterioration of certain shales with time after construction. Some shales are rock-like when excavated but when placed as rock fill can deteriorate into weak soil. Other shales, often interbedded with limestone or sandstone, break down when excavated but the large durable rocks prevent adequate compaction. The difficulties encountered in using shale in embankments are complicated by variations in the geology and physical properties of weak sedimentary rocks, the climate and groundwater conditions, and the weather and construction methods.

3. A four year research study sponsored by the Federal Highway Administration (FHWA) was started in July 1974. Work on Phase I to define the problems and current practices and Phase II to recommend evaluation techniques and remedial measures was completed in July 1975 to meet critical needs. Reports on these two phases have been published and distributed by the FHWA[2, 3]. Phase III is underway and will conclude in June 1978 with a Phase III report and technical manual.

Limitations

4. The scope of the research study excludes:

a. Settlement and stability problems originating in the foundation beneath an embankment.

b. Stability problems in cut slopes.

* The term shale includes weak sedimentary rocks such as claystone, siltstone, mudstone, etc.

c. Deterioration of compacted shale caused by frost action.

Phase I results

5. The results of Phase I indicate:

a. All of the states concerned have recognized the problems in using shale in embankments. A number of states have delineated problem shale formations and have developed special design features and specification provisions for the use of shale in embankments.

b. Basic causes and contributing factors summarized in Fig. 4 emphasize the critical need for reliable tests and criteria to predict shale behavior after construction. The severest problems occur in five of the east central states, Indiana, Kentucky, Tennessee, West Virginia, and Virginia.

c. Current construction practices. Nine (9) of the sixteen (16) states included in the study require all shale to be placed in thin lifts and compacted as soil (Figure 5) for lack of reliable tests and criteria to define durable shales. Seven (7) states use shale as rock fill under certain conditions; four of these states have special criteria or provisions for selecting shales that can be used as rock fill (California, Indiana, Kentucky, West Virginia).

d. Shale deterioration index tests. Index tests to assess shale deterioration (Figure 6) were selected considering previous experience, simplicity, test time, cost, and sensitivity range. Soaking of an oven dry specimen in a jar ofwater is the simplest test; systematic testing can be done quickly such as on core pieces to provide continuous data with depth in a shale formation. It may be the only test needed on weaker shales.

(1) The slake-durability test is considered a basic test. The index from this test is a measure of the percent remaining and the higher the index the more durable the shale.

(2) The jar-soaking and slake-durability tests are similar to two of four tests for a proposed classification system developed at Purdue University under JHRP sponsored by the Indiana State Highway Commission[4].

(3) The standard procedure for slake-durability tests (Figure 7) adopted by the International Society of Rock Mechanics was used. A commercial slake-durability apparatus (Fig. 8) was used for the tests and can accommodate four tests at one time by adding another unit on each side. The slake-

durability index and the type of break down need to be considered together since a high index can occur with break down of shale into gravel size fragments (Figure 9). The natural water content is also a useful index and is obtained as part of the slake-durability test when shale samples are sealed in plastic bags in the field.

(4) The pH of the water after a slake-durability test is an important indicator of potential chemical weathering. X-ray diffraction tests are also considered important in identifying mineral content which may effect deterioration, especially with regard to chemical weathering.

e. Formation variability. The limited variability study indicated that certain shale formations have unique characteristics and variations that can be defined by visual examination, index tests, and mineralogical studies. An example for the problem shales in the Dillsboro and Kope formations (Figure 10) shows increasing trend in slake-durability index (all below 70 percent) with depth (also a decrease in water content and an increase in calcite in the Kope). The Nilichucky problem shale (Figure 11) appears to be relatively uniform (I_D = 60 to 90) over the upper section.

f. Comparison slake-durability with performance. The index properties of the natural unweathered shales associated with a specific problem embankment were compared (Figure 12) and indicate a low slake-durability for the embankments in Kentucky and Indiana. The I_D values of 84 and 91 which are relatively high (KY 12 and KY 20) were for samples which broke apart in thinner pieces. The jar slake index values of 1 or 2 correspond to I_D values of 0 to 15. The pH values are greater than 7.

For the I-64 embankment in Virginia where settlement of bridge abutments is occurring the I_D values for the problem Millboro shale are 97 to 98 but the pH is 5 to 6. Mineralogical examination of samples from the embankment and of fresh shale from a cut indicated that chemical weathering of pyrite was producing sulfuric acid and causing deterioration of clay minerals in the shale to form secondary minerals.

Phase II results

6. Infiltrating water and subsurface seepage into shale embankments over several years is the primary cause of shale deterioration and produces weak impervious zones. Settlement, deformation, and small slope failures are often handled at the district level by pavement overlaying, and pile retaining structures. The geotechnical staff at the state level is often not involved until a large slide occurs.

a. Evaluation procedures. Successful evaluation requires early detection, field inspection, and a carefully planned observation and subsurface investigation program. Periodic longitudinal and cross-section profiles or traverse surveys along the slopes can help define the extent and rate of settlement and deformation. Disturbed sampling helps to define the material character and profile and the borings can be used to install piezometers and slope indicator casing. Undisturbed core barrel sampling should be tried. Penetration resistance and water content data can define weak soil zones. A complete evaluation of all information is needed to define the type, source, and extent of distress.

b. Shear Strength. Undrained triaxial tests on 12- and 15-in. diameter compacted samples of shale (Figure 13) show a high friction angle at a water content less than 10 percent. However, when saturation occurs (Figure 14) the shear strength parameters (c,∅) drop to low values.

c. Remedial measures. Remedial Methods can be grouped into six types (Fig. 15). Selecting the best method depends on a number of considerations (Fig. 16). Drainage is the most important and includes repair of ditches and grading to prevent ponding, horizontal drains, and vertical drains in interbedded limestone and shale connected to horizontal drains at side hill embankments. Drainage is also an important aspect of other methods.

Phase III

7. During the current Phase III work, 5-in. diameter undisturbed sampling (using a special double tube sampler with a carboloy bit[3] and compressed air for removing cuttings) and pressure meter tests (using the Menard apparatus) were completed at six problem embankments. The field work was necessary to better define the role of shale deterioration, and determine the deformation and strength characteristics. The selected sites in Tennessee, West Virginia, Ohio, and Indiana are described below.

8. I-75, Tennessee. North of Knoxville, Tennessee, an embankment for I-75 (Figure 17) crosses from the Hance formation (shale) into the Lee formation (sandstone). Both formations are Pennsylvanian in age. This long, 100-ft-high embankment was constructed with shale as rock fill in 3 ft. lifts. The Hance shale underlying the Lee sandsonte is shown in Figure 18.

a. Undisturbed samples (Figure 19) contained shale, some sandstone, and considerable soil which did not appear to be a result of shale deterioration. Four 55-gallon drums of unweathered Hance shale were obtained from cuts where the embankment material was obtained. This shale is very hard and corresponds to unweathered chunks found in the embankment.

b. It appears that considerable soil and finer grained shale particles were mixed with rock during construction and the loosely compacted material settled and increased in water content from infiltration of surface water.

c. This section of I-75 was investigated in detail by Law Engineering Testing Company for the Tennessee Department of Transportation in 1972.

9. U.S. 460, West Virginia. A series of five (5) embankments 12 miles east of Princeton, West Virginia, have settled 2 to 3 ft. since construction in 1970. This downhill section cuts through upturned beds of Hinton Formation (Figure 20) which contains shale, siltstone, and occasional sandstone and limestone strata of Mississippian age. Sandstone and limestone were placed in 3-ft. lifts at the base of the fills. Hard shale was compacted in 2-ft. lifts and soft shale, soil and random materials were compacted in 8-in. lifts.

a. Undisturbed sampling and pressuremeter tests were performed in a 70-ft. high embankment near the bottom of the downgrade (Figures 21 and 22) where it was certain that materials for the embankment came from the adjacent cuts. Undisturbed samples (Fig. 23) indicated that the embankment was generally well compacted but contained some loose layers. Hard pieces of shale and siltstone were found between gravelly soil strata as indicated by the wasted samples etched by rain (Figure 24).

b. The West Virginia State Highway Department is concluding a special investigation of this section of U.S. 460. A unique feature is the use of well-logging services to study the moisture-density changes with time. Settlement of the embankments appears to have stopped.

10. I-74, Ohio. Two (2) bridge approach embankments were sampled on I-74 northwest of Cincinnati. This section cuts through interbedded shale and limestone of the Fairview and Kope formations (Ordovician age).

a. The first site is at State Route 128 (Figures 25 and 26) where shale with about 30 percent limestone was compacted in 8-in. lifts by four coverages of a heavy tamping roller and two coverages of a 50-ton, 4-wheel pneumatic tired roller. Construction was completed in 1961. Measured settlements of the 55-ft. high embankment amounted to only 3 in. Lateral movement (perhaps along the west abutment hillside foundation) has been enough to cause distress to the abutments. The undisturbed samples (Figure 27) of the east side embankment indicate that the fill was well compacted in thin lifts. The fill was not wet and included unweathered chunks of Kope shale.

b. The second site is at Wesselman Road and Taylor Creek (1.3 miles east of State Route 128) where 50-ft. high bridge approach embankments were constructed in 1962 using lifts as thick as 3 ft. Settlement of the soil foundation (80 ft. to bedrock) may have been the main problem here since 15 ft. of soft foundation material was removed at the start of construction. Approximately 22 in. of settlement at the abutments occurred from 1962 to 1968. A test pit made in 1968, after grouting the embankment (very low grout take), indicated no large voids in the fill. The undisturbed samples contained shale chunks and looser material (which was not wet) compared to the samples at the State Route 128 location.

c. Current construction on I-275 near Wesselman Road during April 1976 indicated the difficulty of compacting shale with slabby limestone boulders (Figure 28).

11. I-74, Indiana. A five mile section of I-74 at the Indiana-Ohio line has experienced several embankment slides and large settlements since construction in 1964-1965. Thinly interbedded shale and limestone (45 percent shale, 55 percent limestone) of the Dillsboro formation (late Ordovician in age) was used as rock fill in lifts up to 3 ft. An extensive investigation of the current stability is nearing completion by E. D'Appolonia consulting Engineers for the Indiana State Highway Commission.

a. The longest embankment (Figure 29) along this 5 mile section (60 ft. high at the median on a 3:1 foundation slope) has settled several feet and has had a small surface slide on the north side (Figure 30). Seepage water is percolating out of the north slope at several locations. Undisturbed samples (Figure 31) showed a large amount of limestone and loose shale chunks with zones of soft soil.

b. Obtaining unweathered shale from the adjacent cut was difficult because of the thin beds and in-place hardness. This can be seen from the back hoe teeth marks (Figure 32). However, once excavated the 2- to 6-in. shale chunks could be broken easily by hand. Thus the high percentage of thin limestone (1 to 2 in. thick) and soft nature of the shale made it very difficult to obtain adequate compaction of the mixture.

12. Pressuremeter tests. The Menard pressuremeter[5] (Figure 33) was used with the 30-in. long NX-size probe and a urethane sheath. The pressure regulator is connected to a bottle of nitrogen or carbon dioxide and the bottle pressure is read on the right dial; the inner membrane fluid pressure is read on the large center dial, and the outer sheath air pressure is read on the left dial. The black inner rubber membrane used to measure volume change has an effective length of 18.3 in. The fluid pressure on the inner membrane is kept 1.0 to 2.0 kg/cm^2 higher than the outer sheath air pressure to make the inner membrane push against the outer sheath. After lowering

to the test depth, the probe was allowed to equilibrate under the fluid head on the inner membrane. An applied pressure of 1.0 kg/cm^2 was then applied for three minutes. The applied pressure was increased by 1.0 kg/cm^2 about each 1-1/2 min. and volume reservoir were recoreded at 15-, 30-, and 60-sec. intervals under each applied pressure. Volume readings were also recorded before the probe was lowered into the boring and after equilibrium under the hydrostatic fluid pressure at the test depth.

a. A 2-15/16-in.-diameter tricone roller bit and compressed air was used to advance the pressuremeter boring to one-half the embankment depth. Tests were made at 2.5 to 5.0-ft. intervals from the bottom upward. The boring was then drilled the remaining depth, usually the next day, and the tests completed. This procedure freed the drill rig for undisturbed sampling in a second boring. A smooth and fairly undisturbed borehole wall was obtained and the nominal 3-in. diameter allowed sufficient space around the probe (1/8 in. all around) to prevent the probe from getting stuck. However, at the Tennessee site, the probe could not be pushed below 65 ft. in two trial borings.

b. The urethane sheath was stiffer than desired, but was necessary to prevent puncturing by small gravel particles. Thus a large inertia correction was required for the stiffness of the urethane sheath. This correction was found to decrease with increased use of the same sheath.

c. Results of the tests at the Tennessee site have been reduced. A plot of the test at 59 ft. is shown in (Figure 34). The term P_o on the corrected test curve denotes the pressure when the probe has compressed the borehole wall to its orginal state and is assumed to be a rough measure of the at-rest lateral earth pressure. The term P_f denotes the pressure at the start of plastic failure (or start of shear failure) and P_1 the limit pressure indicating complete shear failure. The moduli values are calculated using mearured pressures and volume changes for the steepest portion of the corrected test curve which is analogous to a stress-strain curve. The estimated undrained shear strength is calculated from pressure and volume changes near the P_f pressure and is about one half of P_f.

d. The fluid line to the probe from the surface caused a hydrostatic head of 1.8 kg/cm^2 at a depth of 59 ft. With an applied pressure of 1.0 kg/cm^2 at the ground surface, the total pressure of 2.8 kg/cm^2 at the probe increased the probe diameter to 3.0 in. At a depth of

100 ft. the hydrostatic head alone is enough to increase the diameter to 3 in. Thus the pressuremeter is limited to depths of about 100 ft. in soft soils to prevent excessive expansion of the borehole wall by hydrostatic pressure before the test is started.

e. The six valid tests (out of 10 tried) at the Tennessee site are summarized in Figure 35. The pressure and modulus values at the left side of the plot fall into two groups and are somewhat lower than expected.

Conclusion

13. Based on the recent field tests, it appears that shale deterioration in problem embankments is not as great a factor as originally believed. Mixing of soil and fine-grained particles of shale with large chunks of hard shale, siltstone, sandstone or limestone which prevent adequate compaction in thick lifts is a major cause.

a. The results of the laboratory investigation being started will better define the role of shale deterioration and provide information needed to better utilize shale materials in embankments.

b. The laboratory investigation will include petrographic examination of undisturbed samples, triaxial compression and shear tests on selected undisturbed samples, slake-durability and compaction tests on the parent shales used in the embankments, and development of a rapid aging test on compacted parent shale to aid in predicting long-term deformation and strength characterstics. This information and that gathered from the states and other agencies will provide technical guidance in the form of a manual for use by the operating engineer.

9

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5. Roy, M., Jeneau, R., La Rochelle, P., and Tavenas, F. A., "In-Situ Measurement of the Properties of Sensitive Clays by Pressuremeter Tests," Vol I, Proceedings of the Conference on In-Situ Measurement of Soil Properties, June 1975, American Society of Civil Engineers, New York, New York.

Figure 4. **CAUSES OF SHALE EMBANKMENT SETTLEMENT AND FAILURES**

BASIC CAUSES

- INFILTRATION OF SURFACE AND GROUND WATER
- SHALE DETERIORATION (PHYSICAL AND CHEMICAL) EXPANSIVE CHARACTERISTICS

CONTRIBUTING FACTORS

- EXCESSIVE LIFT THICKNESS
- INADEQUATE COMPACTION
- MIXING SHALE WITH HARDER ROCK (LIMESTONE OR SANDSTONE)
- INADEQUATE CONSIDERATION OF ALL GEOLOGICAL CONDITIONS
- SIDE SLOPES TOO STEEP
- INADEQUATE DRAINAGE
- LACK OF SIDE HILL BENCHING

BASIC NEED

ESTABLISHED TESTS AND CRITERIA TO RELIABLY PREDICT SHALE BEHAVIOR AFTER PLACEMENT

Figure 5. SHALE EMBANKMENT CONSTRUCTION SPECIFICATIONS FOR 16 STATES

A. ALL SHALE PLACED IN THIN LIFTS (8 TO 12 IN) AND COMPACTED AS SOIL:

OREGON	MISSOURI
MONTANA	OHIO
UTAH	NORTH CAROLINA
SOUTH DAKOTA	PENNSYLVANIA
OKLAHOMA	

B. SHALE PLACED AS ROCKFILL USING CRITERIA OR SPECIAL PROVISIONS

CALIFORNIA	– GRADATION CRITERIA FOR LIFT THICKNESS OF ROCKY MATERIALS. 90 PERCENT RELATIVE COMPACTION
INDIANA	– ROCK-LIKE SHALE DEFINED BY DURABILITY TESTS 24 IN. MAX LIFT THICKNESS
KENTUCKY	– CLASS TESTS AND INSPECTION OF ROCK SAMPLES. 18 IN. MAX LIFT THICKNESS
WEST VIRGINIA	– FIELD COMPACTION RESPONSE 24 IN. MAX LIFT THICKNESS

Figure 6. **TESTS SELECTED FOR ASSESSING SHALE DETERIORATION**

TEST	DETERIORATION
JAR SLAKING	PHYSICOCHEMICAL
SLAKE-DURABILITY	PHYSICAL AND PHYSICOCHEMICAL
pH ON SLAKE-DURABILITY RESIDUE	PHYSICAL, PHYSICOCHEMICAL, AND CHEMICAL
X-RAY DIFFRACTION	MINERAL IDENTIFICATION; PHYSICAL, PHYSICOCHEMICAL, AND CHEMICAL

Figure 7. SLAKE-DURABILITY PROCEDURE

1. 10 ROUNDED CUBES, EA 40 TO 60 GRAMS
2. OVEN DRY
3. PLACE IN CAGE
4. SUBMERGE CAGE PARTWAY IN WATER
5. ROTATE AT 20 RPM FOR 10 MIN
6. REPEAT STEPS 2-5 (2ND CYCLE)
7. OVEN DRY MATERIAL LEFT IN CAGE

SLAKE-DURABILITY INDEX

$$I_D(\%) = \left(\frac{\text{DRY WT AFTER 2 CYCLES}}{\text{DRY WT BEFORE TEST}}\right) \times 100$$

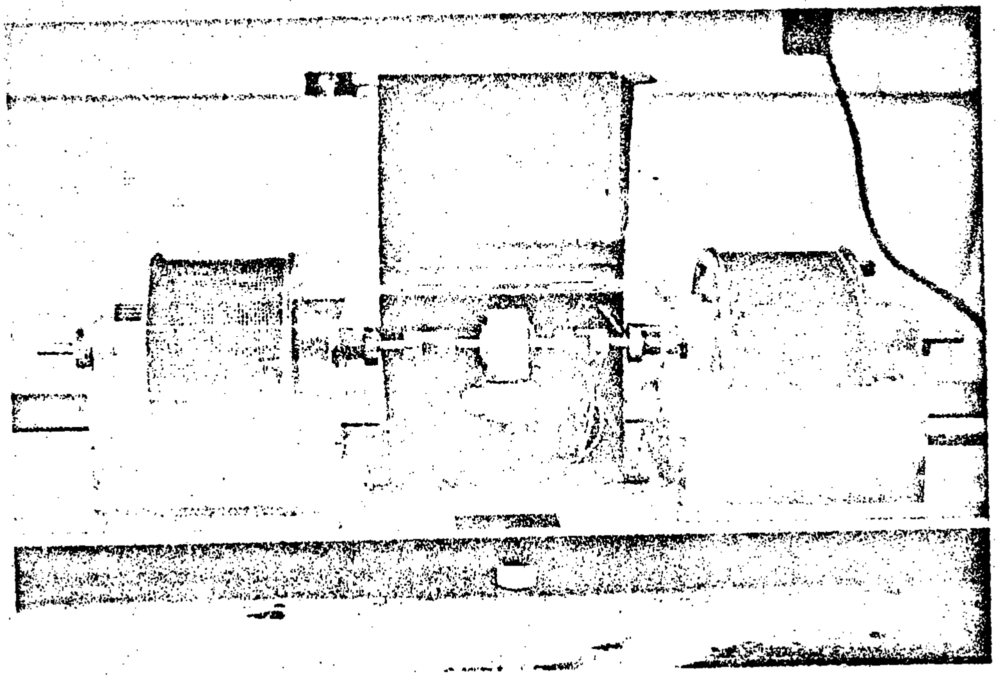


Figure 8. Commercial slake-durability apparatus.

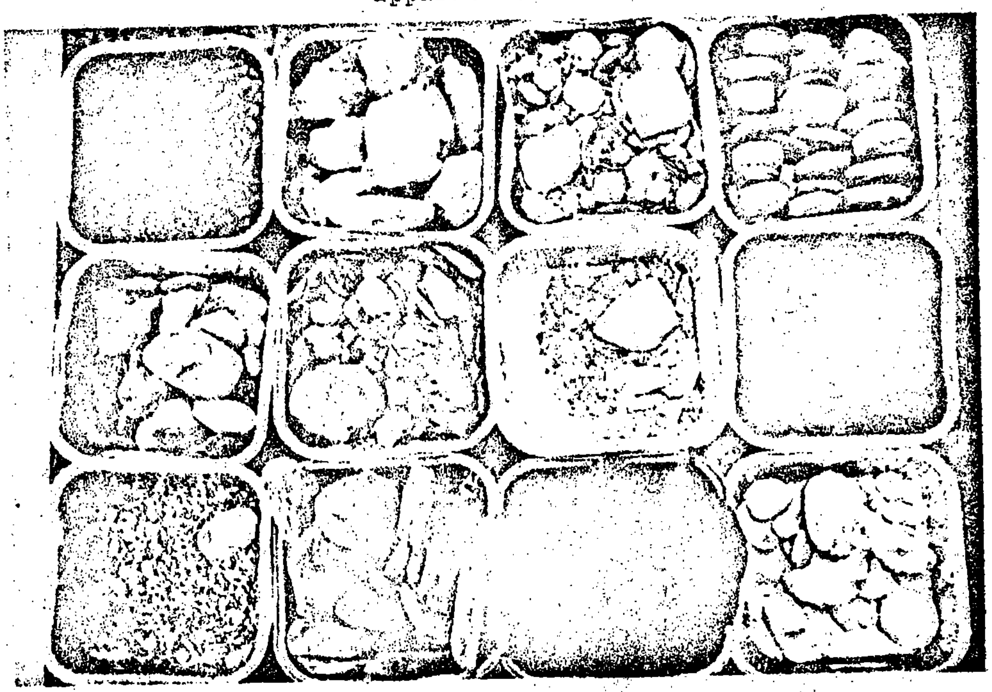
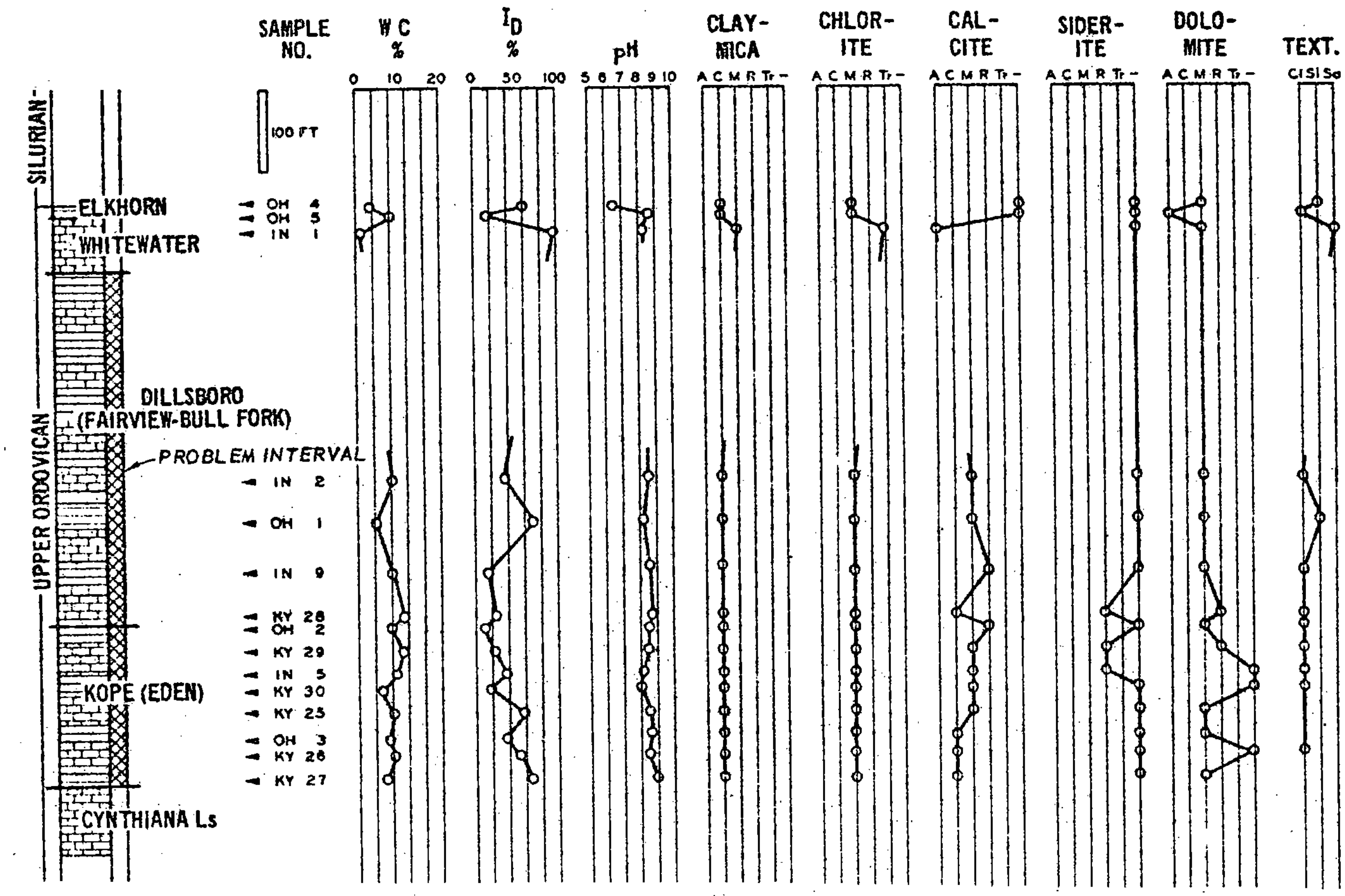


Figure 9. Character of shales after slake-durability test.

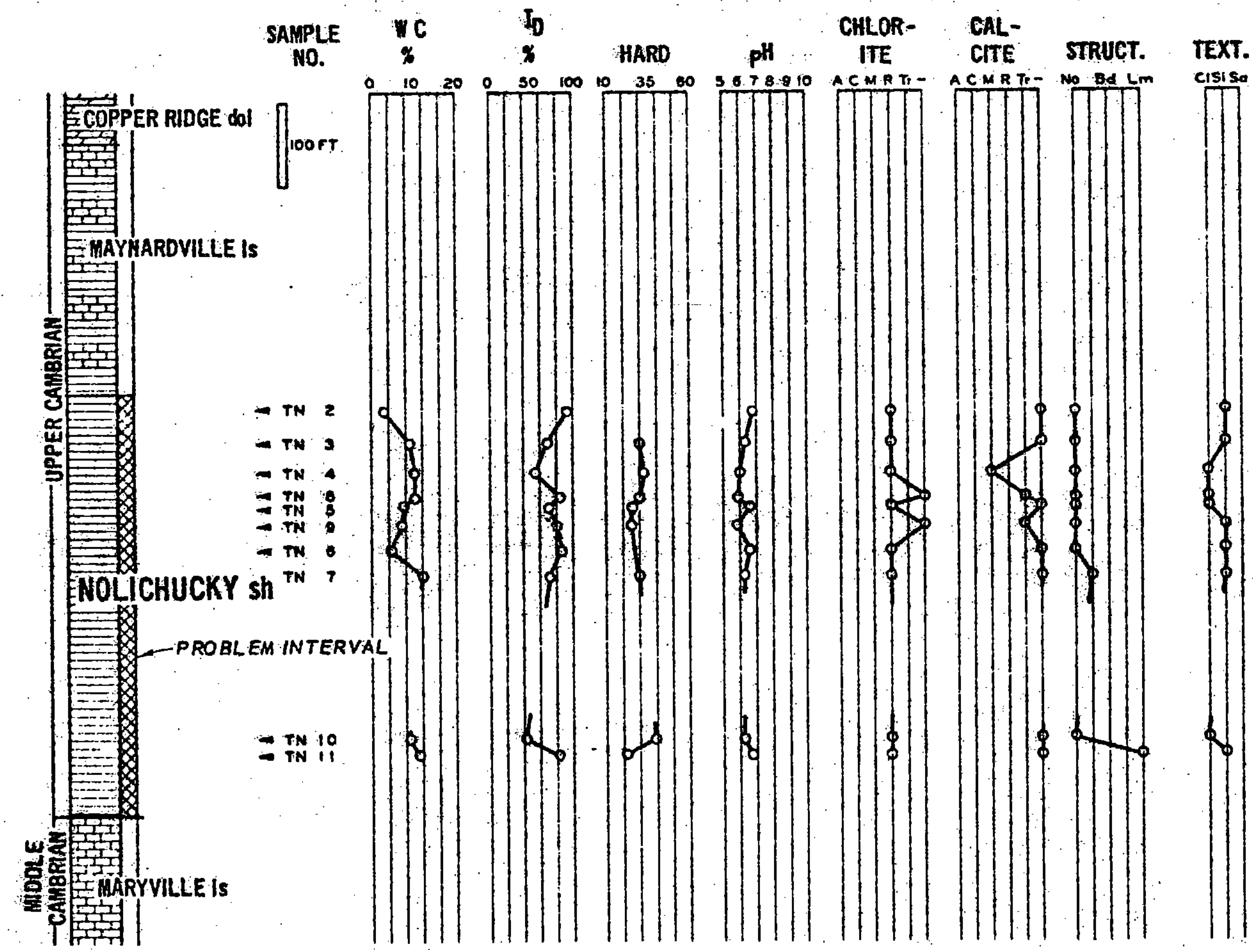
Figure 10. **CHARACTERISTICS OF DILLSBORO AND KOPE FORMATIONS**



Legend:

A = abundant
C = common
M = minor
R = rare
Tr = trace
Cl = clayey
Si = silty
Sa = Sandy

Figure 11. **CHARACTERISTICS OF THE NOLICHUCKY FORMATION**



Legend:

A = abundant R = rare No = no structure HARD = hardness using Model D Scleroscope.
C = common Tr = trace Bd = bedded
M = minor Lm = laminated

Figure 12. **COMPARISON OF SHALE TEST RESULTS FOR SELECTED PROBLEM EMBANKMENTS**

STATE	LOCATION	GEOLOGIC UNIT	SAMPLE NO.	NATURAL WATER CONTENT, %	SLAKE-DURABILITY INDEX, I_D %	JAR SLAKE INDEX	RESIDUE WATER pH
IN.	I-74, SE INDIANA	EDEN GROUP, KOPE FM	IN 5	9.3	38	4	8.2
KY	WESTERN PARKWAY, GRAYSON CO	TRADEWATER FORMATION	KY 12	--	84	--	7.5
			KY 13	9.0	53	4	7.2
	I-64, BATH CO	CRAB ORCHARD FORMATION	KY 14	3.3	65	5	8.2
			KY 15	8.3	4	2	7.3
			KY 16	14.8	0	1	7.4
	I-64, CARTER CO	BREATHITT GP	KY 20	4.8	91	4	7.2
			KY 23	8.6	6	1	8.1
	BLUE GRASS PARKWAY, NELSON CO	OSGOOD FM	KY 1	7.1	15	2	--
			KY 2	20.3	9	1	9.2
			KY 3	18.3	11	2	9.4
VA	I-64, CLIFTON FORGE	BRALLIER*	VA 1	1.1	98	6	4.6
		MILLBORO	VA 2	0.6	99	5	6.1
		MILLBORO	VA 3	1.4	97	6	5.2

* NONPROBLEM SHALE

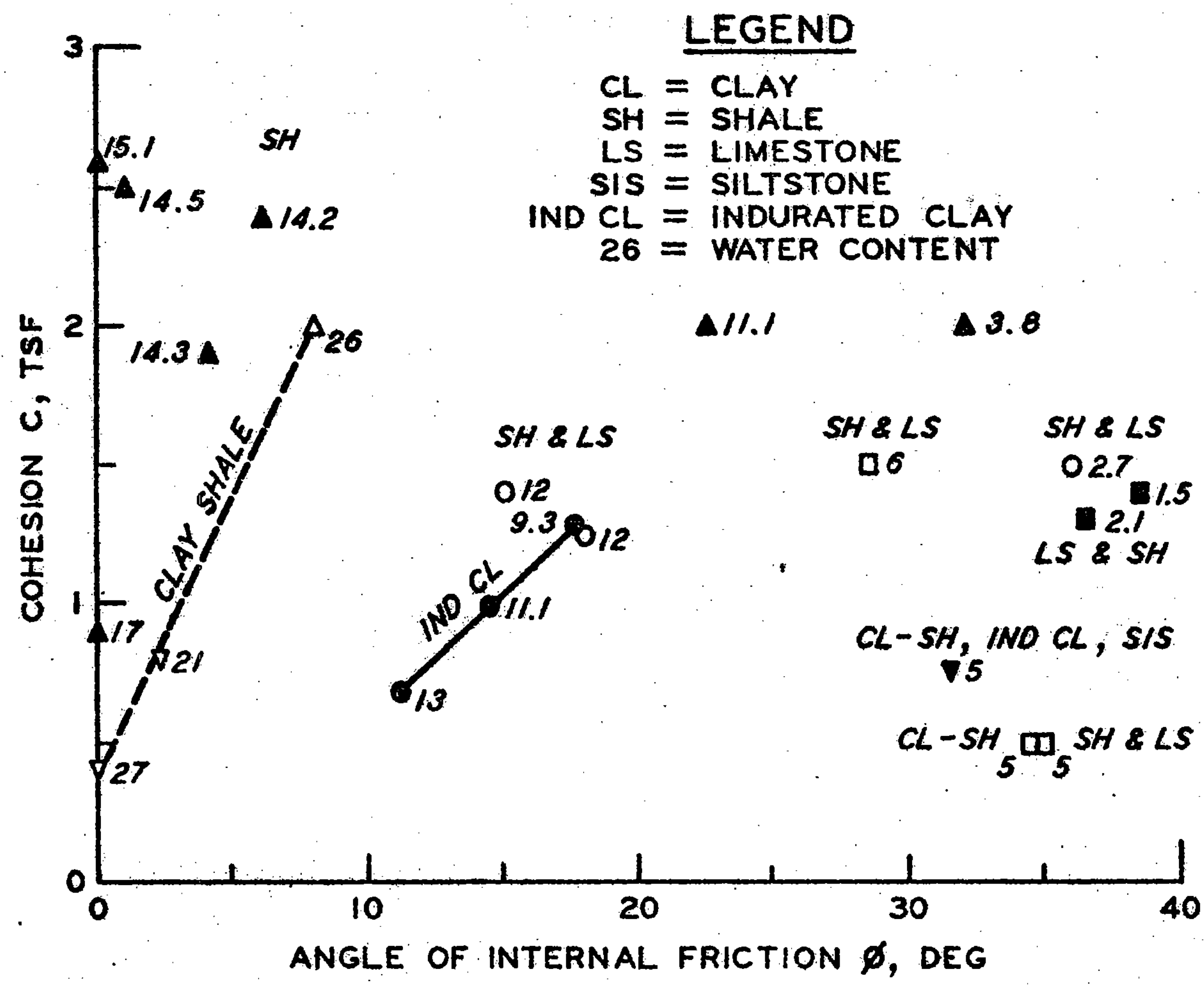


Figure 13. COMPARISON OF Q STRENGTHS, 7CE PROJECTS

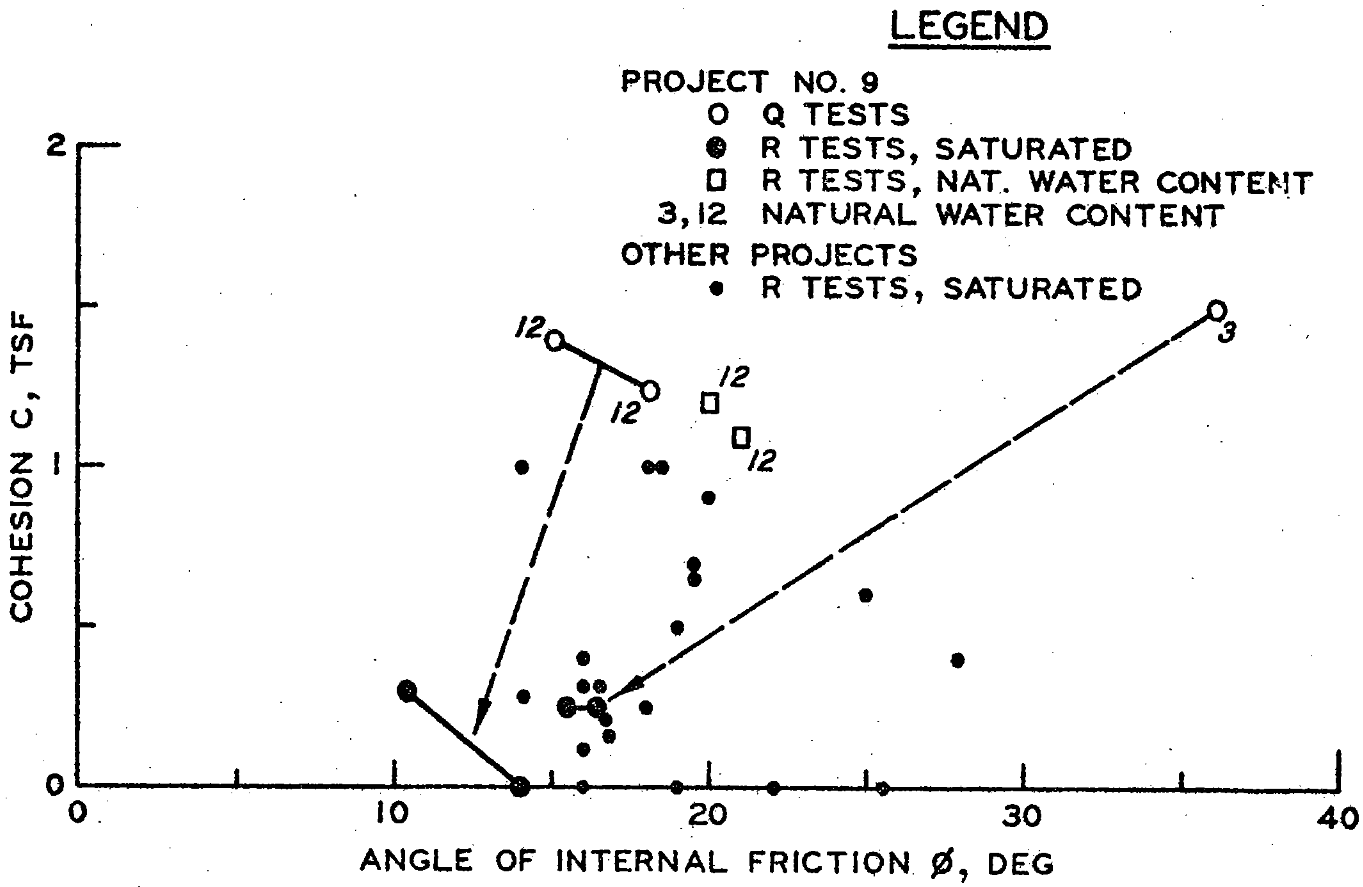


Figure 14. COMPARISON OF Q AND R STRENGTH PARAMETER

Figure 15. **REMEDIAL METHODS INVESTIGATED**

A. PAVEMENT OVERLAY.

B. DRAINAGE SYSTEMS.

C. SLOPE FLATTENING, BERMS, AND BUTTRESSES.

D. RETAINING WALLS.

E. LIME, CEMENT, AND CHEMICAL STABILIZATION.

F. RECONSTRUCTION (REMOVAL AND REPLACEMENT OF MATERIALS).

Figure 16. RECOMMENDED TREATMENT

A. GENERAL CONSIDERATIONS

1. STRENGTH OF FOUNDATION AND FILL
2. RIGHT-OF-WAY LIMITATIONS
3. AVAILABILITY OF MATERIALS
4. CONSTRUCTION FEASIBILITY, COSTS, AND TIME LIMITATIONS
5. LOCAL EXPERIENCE
6. ADVANTAGES AND DISADVANTAGES OF EACH METHOD

B. DRAINAGE

1. MINIMIZE INFILTRATION OF SURFACE RUNOFF
2. EARLY APPLICATION OF HORIZONTAL AND/OR VERTICAL DRAINS

C. OTHER METHODS

1. REMOVAL AND REPLACEMENT OF MATERIALS
2. SLOPE FLATTENING, BERMS (OR BUTTRESSES)
3. RETAINING WALLS
4. CEMENT GROUTING

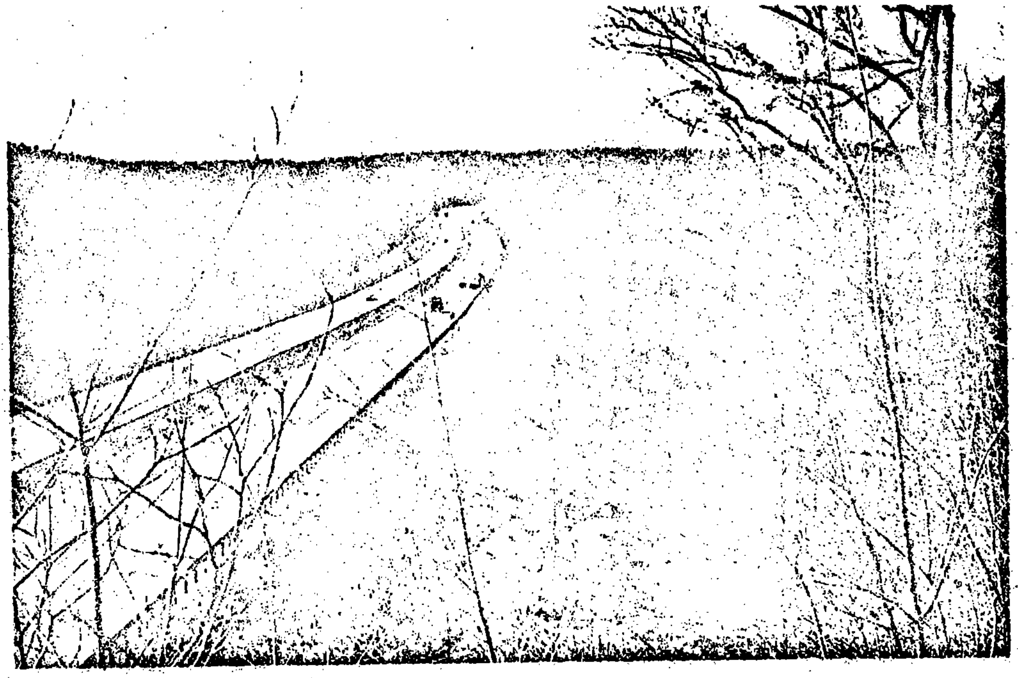


Figure 17. I-75 embankment north of Knoxville, TN.

Figure 18. Hance formation (shale) underlying Lee formation (sandstone).

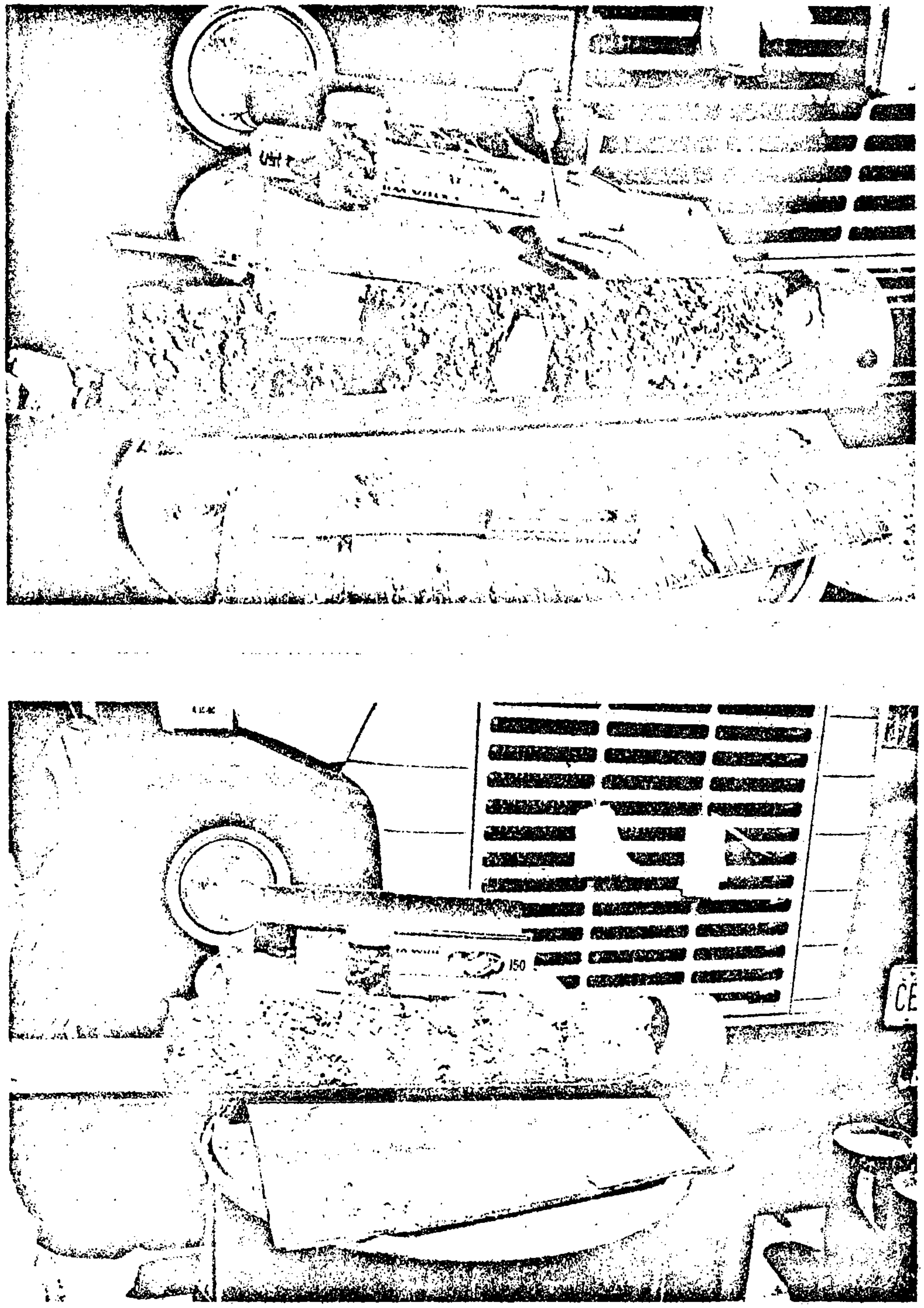


Figure 19. Undisturbed samples,
I-75, TN.

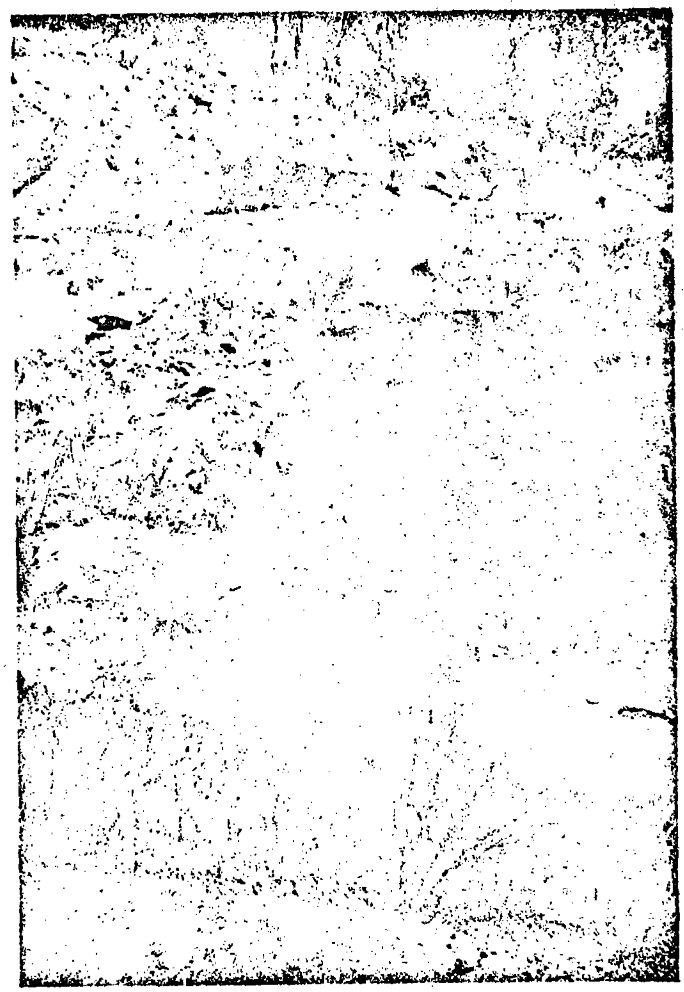


Figure 20. Upturned beds of Hinton formation, US 460 east of Princeton, WV.

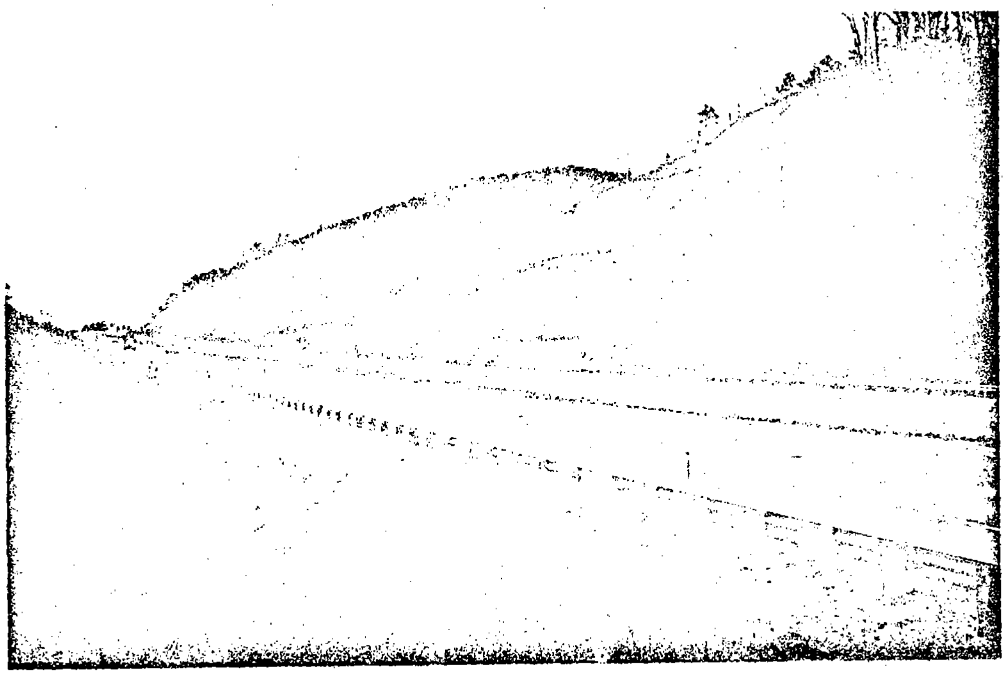


Figure 21. US 460, looking west.

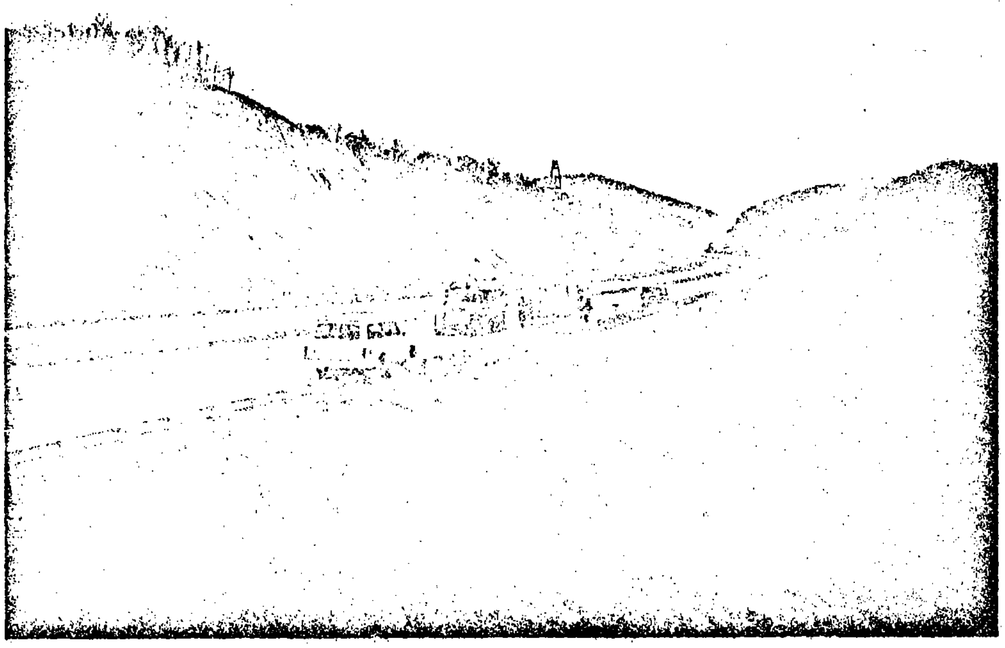


Figure 22. US 460, looking east.

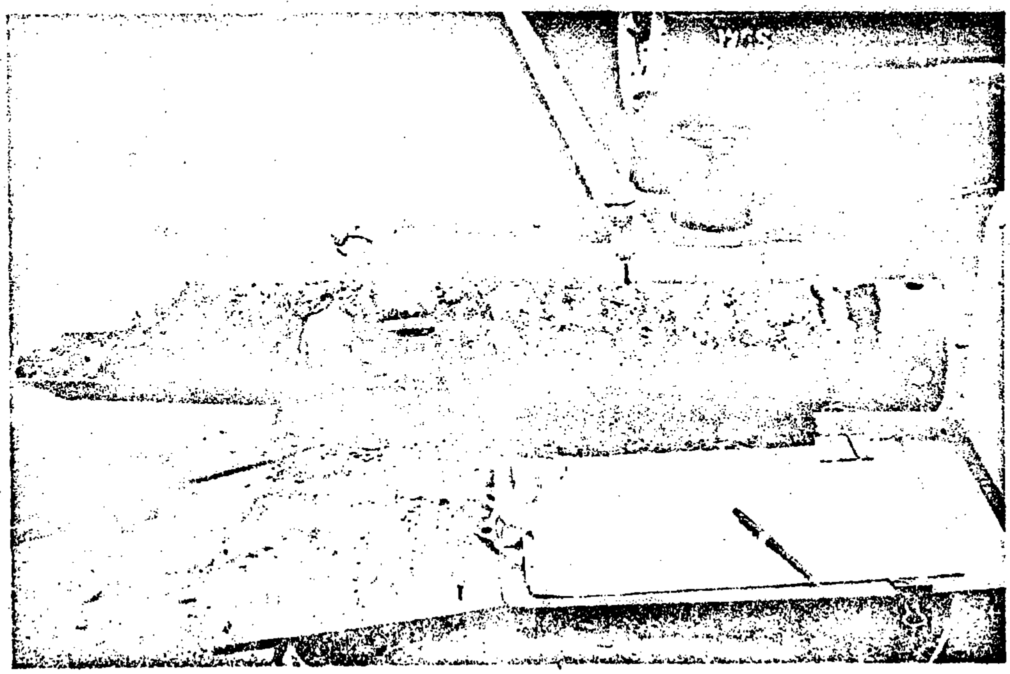


Figure 23. Undisturbed sample, US 460 embankment.

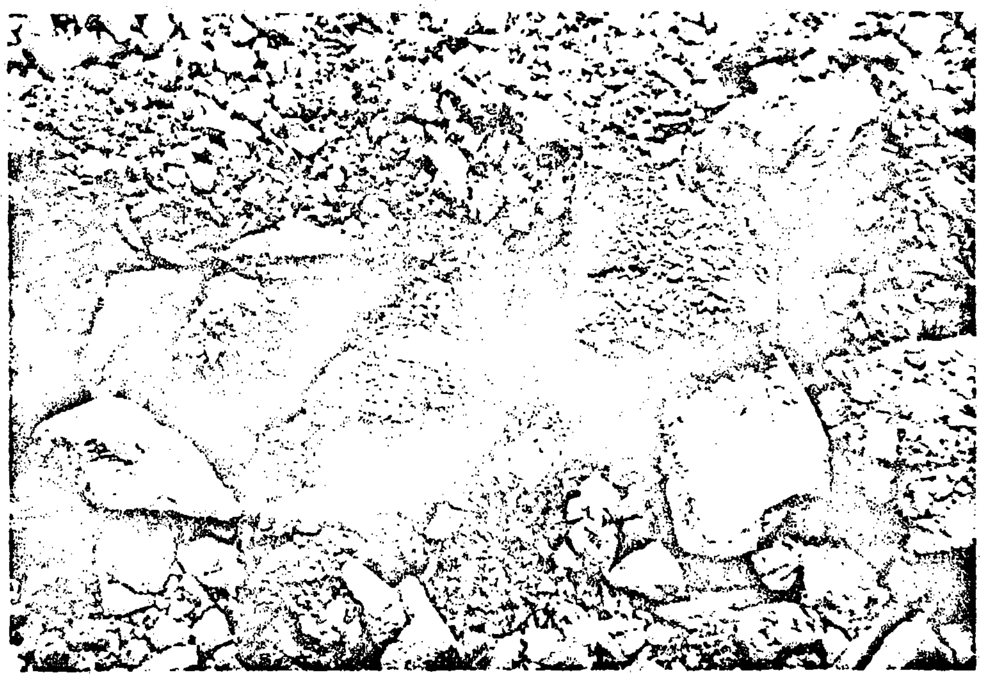


Figure 24. Discarded samples etched by rain, US 460.

Figure 25. Bridge approach embankment on I-74 at SR 128, OH.

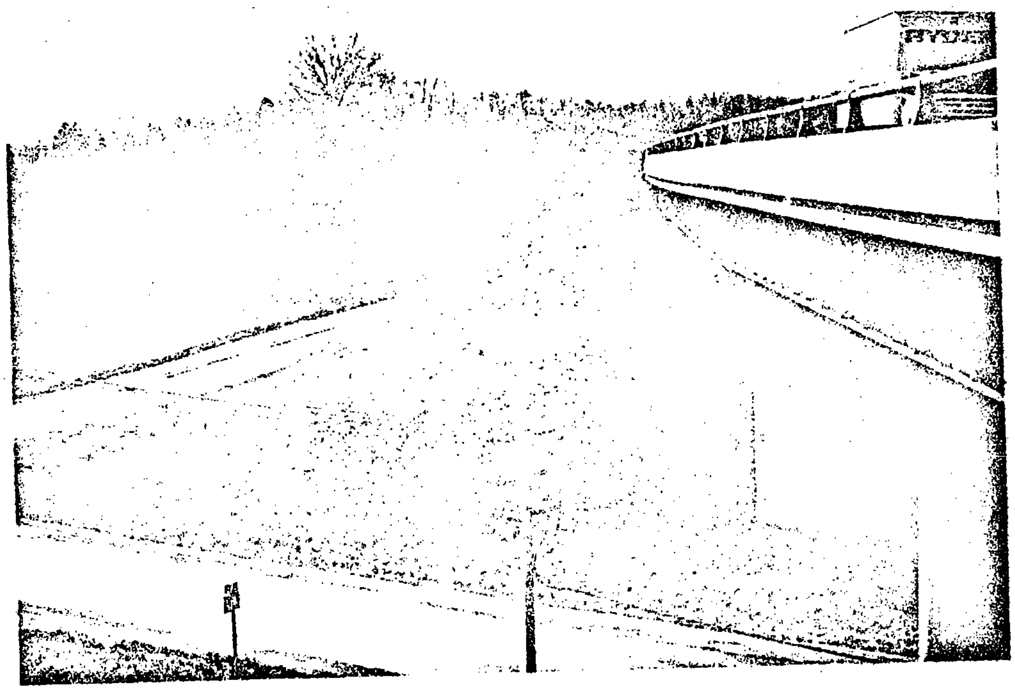


Figure 26. West approach embankment, I-74 at SR 128, OH.

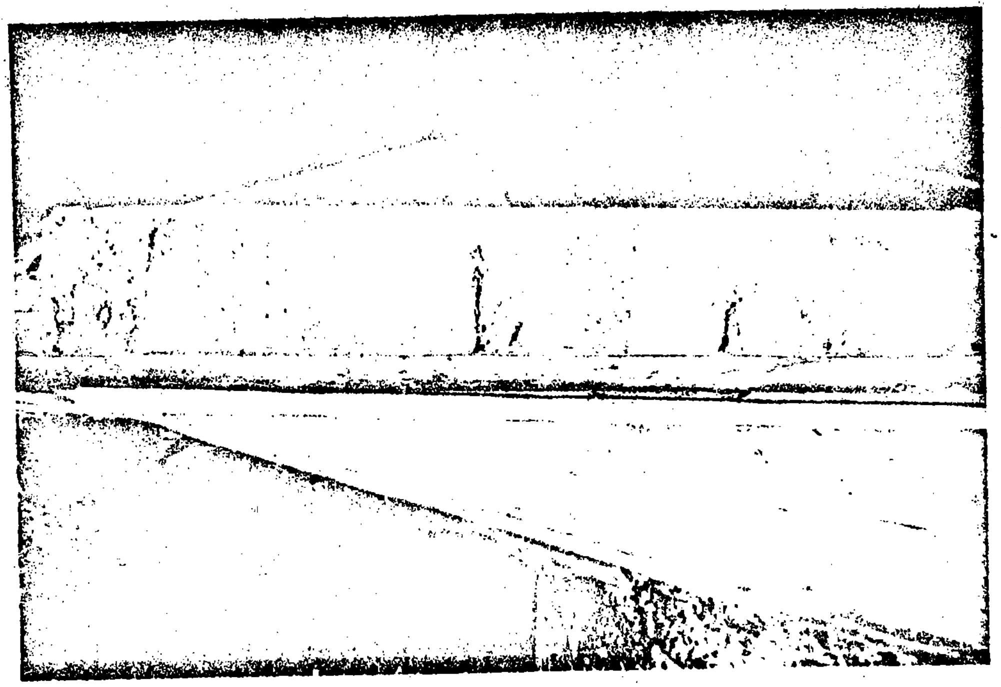


Figure 27. Undisturbed sample, I-74 at SR 128, OH.

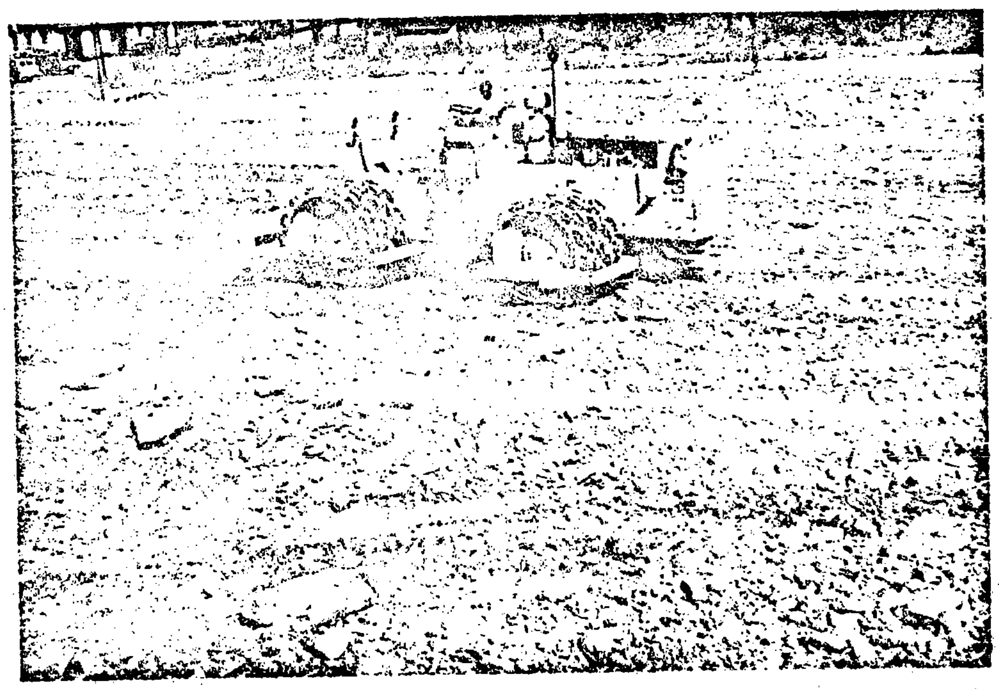


Figure 28. April, 1976 compaction of interbedded shale and limestone on I-275, northwest of Cincinnati, OH.

Figure 29. I-74 embankment near St Leon, IN (looking west).

Figure 30. Surface slide on north slope of embankment, I-74, IN.

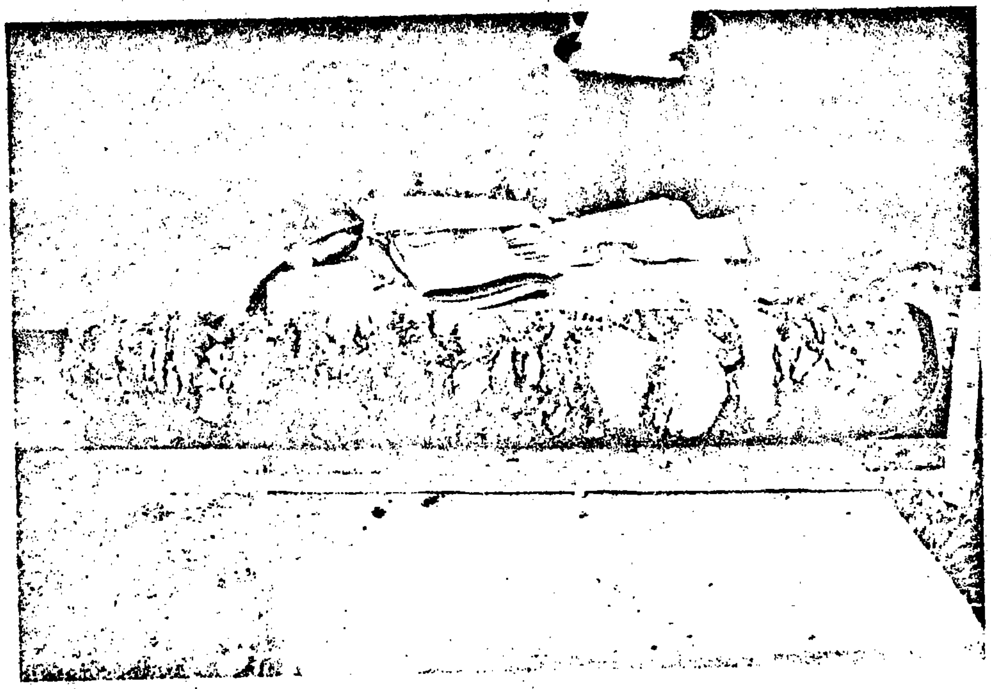


Figure 31. Undisturbed sample, I-74, IN.

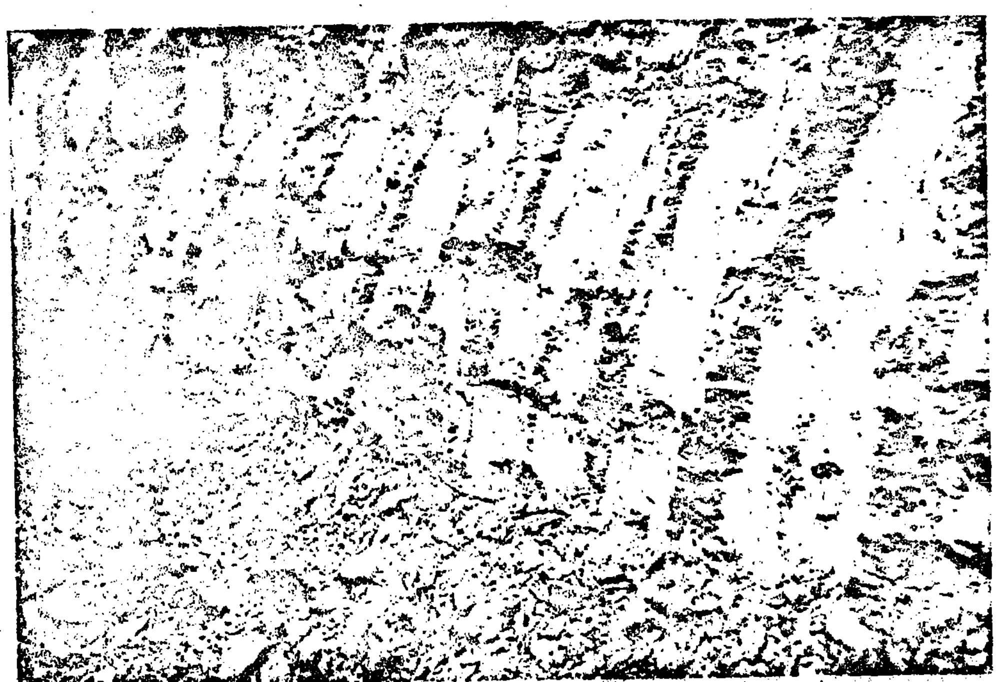


Figure 32. Backhoe teeth marks in excavation for shale, I-74, IN.

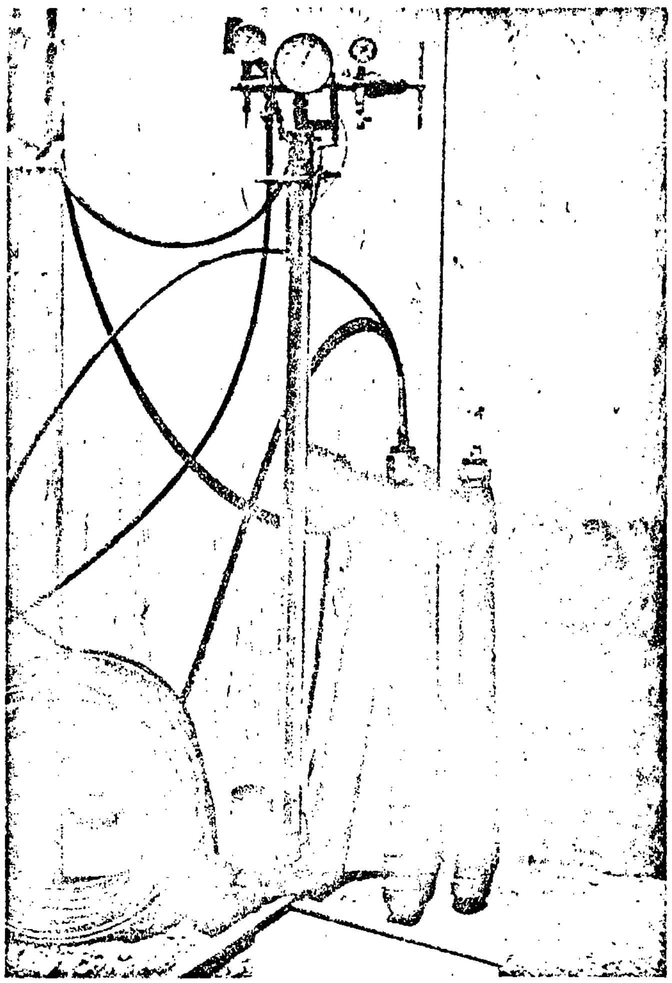


Figure 33. Menard pressuremeter apparatus (complete probe with urethane sheath and bare probe on right).

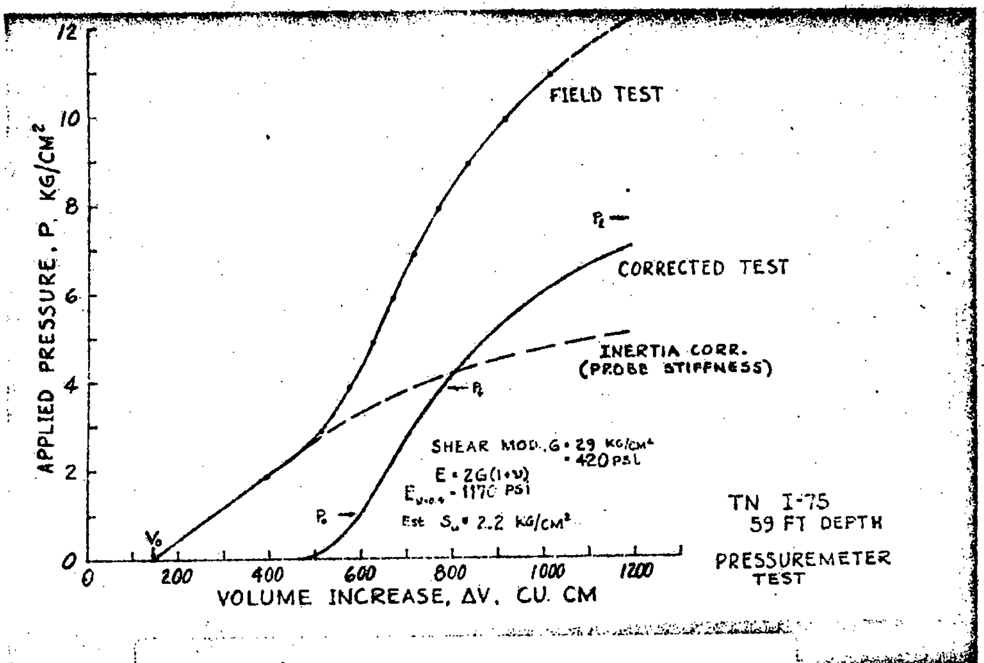


Figure 34. Pressuremeter test results at 59 ft depth, I-74, TN.

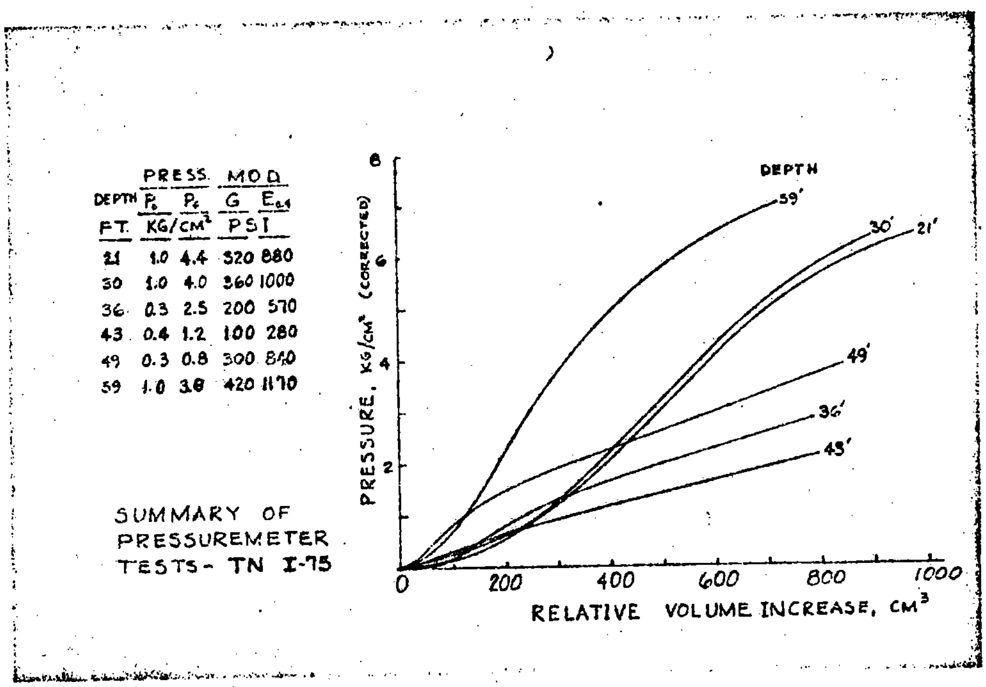


Figure 35. Summary of pressuremeter results, I-74, TN.

The estimated undrained shear strength is calculated from pressure and volume changes near the P_f pressure and is about one half of P_f.

d. The fluid line to the probe from the surface caused a hydrostatic head of 1.8 kg/cm^2 at a depth of 59 ft. With an applied pressure of 1.0 kg/cm^2 at the ground surface, the total pressure of 2.8 kg/cm^2 at the probe increased the probe diameter to 3.0 in. At a depth of 100 ft the hydrostatic head alone is enough to increase the diameter to 3 in. Thus the pressuremeter is limited to depths of about 100 ft in soft soils to prevent excessive expansion of the borehole wall by hydrostatic pressure before the test is started.

e. The six valid tests (out of 10 tried) at the Tennessee site are summarized in Figure 35. The pressure and modulus values at the left side of the plot fall into two groups and are somewhat lower than expected.

Conclusion

13. Based on the recent field tests, it appears that shale deterioration in problem embankments is not as great a factor as originally believed. Mixing of soil and fine-grained particles of shale with large chunks of hard shale, siltstone, sandstone or limestone which prevent adequate compaction in thick lifts is a major cause.

a. The results of the laboratory investigation being started will better define the role of shale deterioration and provide information needed to better utilize shale materials in embankments.

b. The laboratory investigation will include petrographic examination of undisturbed samples, triaxial compression and shear tests on selected undisturbed samples, slake-durability and compaction tests on the parent shales used in the embankments, and development of a rapid aging test on compacted parent shale to aid in predicting long-term deformation and strength characteristics. This information and that gathered from the states and other agencies will provide technical guidance in the form of a manual for use by the operating engineer.

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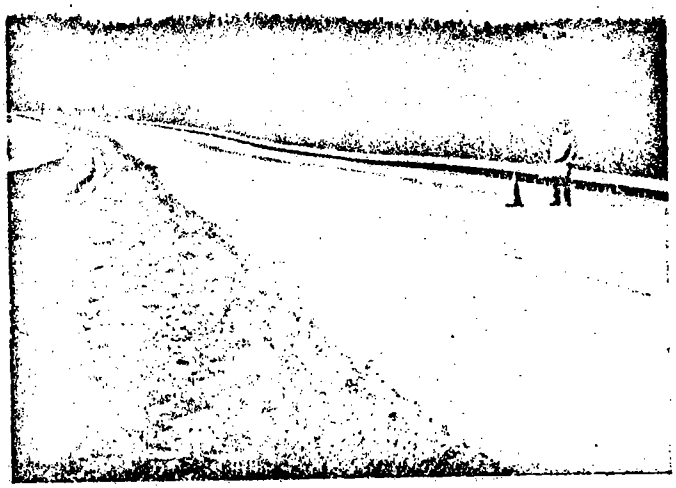

Figure 1. Overlaying required to maintain grade.

Figure 2. Wood pile and guard rail retaining structure.

Figure 3. Addition of berms to flatten slopes.

CURRICULUM VITAE

WILLIAM E. STROHM

Mr. Strohm has been engaged in a variety of soil mechanics research projects at WES since 1956. Research areas included drainage of base courses and improvements in design of subdrains for airfields, compaction and shear strength of cohesionless materials and earth rock mixtures, design and construction of earth and rock-fill dams, dewatering for deep excavations, liquefaction of sands and flow slies in Mississippi Riverbanks, stability of excavated slopes in clay shales, and stability of rock slopes.

He is the author of a number of WES reports and has prepared several Corps of Engineers design manuals.

Mr. Strohm holds an MS degree in engineering from the University of California, Berkely. He is a member of the American Society of Civil Engineers, Society of American Military Engineers, American Society for Testing & Materials, and a registered professional engineer in Mississippi.

Mr. Strohm is the project leader for the shal embankment research being conducted for the FHWA at WES.

SHRINKAGE FACTOR
FOR FILL CONSTRUCTION
IN IOWA

by

DONALD A. ANDERSON

and

SUBODH KUMAR

Presented at the

27th Annual Highway Geology Symposium

Orlando, Florida

May 19 - 21, 1976

SHRINKAGE FACTOR FOR FILL CONSTRUCTION IN IOWA

BY

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SUBODH KUMAR, Assistant Professor, Department of Engineering, University of Arkansas, Little Rock, Arkansas.

ABSTRACT

In any roadway construction project, soil is excavated from one site and placed and compacted at another. To obtain the volume of compacted fill, the volume of excavated material must be adjusted by a "shrinkage factor". Since numerous factors effect it, the exact value of shrinkage factor remains largely indeterminate. In practice, its values are assigned on the basis of engineering judgement.

Estimates of shrinkage factors for various soils in Iowa were made from the statistical analyses of the data from completed highway projects. Shrinkage factors for most soils were found to vary with the volume of earthwork. In addition, variations in them were found to depend on the geologic distribution for the glacial soils and on the principal soil associations for the other soils.

INTRODUCTION

Any highway construction project consists of excavation of soil from one site and its placement and compaction at another. The volumes of excavation and of fill can be determined with reasonable accuracy from their geometrical configurations. Only in rare cases, a certain volume of excavated soil occupies the same volume in a roadway fill. During compaction, expulsion of water and air from the soil mass reduces its volume and increases its density. To obtain the volume of the fill material, the volume of the excavated material must be adjusted by a factor commonly known as the 'Shrinkage Factor'. The relationship used for the adjustment is:

$$\text{Volume of fill material} = \frac{\text{Volume of excavated material}}{1 + \dfrac{\text{Shrinkage Factor, \%}}{100}}$$

During actual construction of a highway, the volume measurements of the excavated material and of the compacted material are affected by many factors and account must be taken in one form or another of approximations used in design, loss of soil during transportation, loss of soil due to erosive action of wind and water, type and amount of compaction, compaction characteristics of soil types, the moisture content at which the soil is compacted, subsidence of surface under the load of equipment, settlement of surface under the embankment load, physico-chemical properties of the soil types involved, errors in quantity measurement, etc. Some of these factors may be difficult to take into account even though their effects may be significant.

After the completion of survey and before the start of earthwork, the changes that take place in the topography of the area are seldom accounted for and many a time small changes are made in roadway alignment and grade without the benefit of new survey data. That some amount of soil is lost during transportation is a well known fact. Prior to and during construction soil is lost also due to the erosive actions of wind and water. The effects related to compaction have been studied in detail and shrinkage due to them can be determined reasonably accurately using standard test like that specified by AASHTO designation T 99-70 (ASTM designation D 698-64 T). The effects of other factors have been studied at length but any calculations based on them remain time consuming and extremely expensive.

At the present time the values of shrinkage factors are based on engineering judgement and past experience with the construction in the area under consideration.

As yet, however, there is no rational way to determine the values of shrinkage factors. Furthermore, on a small roadway project, any attempt to establish values for shrinkage factors from laboratory and field tests would be economically expensive and practically futile. Determination of the values of shrinkage factors, however, remains an important item not only from the engineer's point of view but also from those of the construction contractor and the contract awarding agency.

METHOD OF INVESTIGATION

In an attempt to determine the values of shrinkage factors, data was collected from roadway construction projects completed by the Highway Division of the Iowa Department of Transportation. An investigation of the data revealed that the shrinkage factors were greatly affected by the type of soil. In Iowa, the soils, that are most frequently used for fill construction can be classified in the following three (3) types:

1. Silty Clays: These include all soils classified as topsoil, loess, clay, silty clay, silty clay loam, clay loam, loam, silt loam, silt and certain weathered glacial clays and are characterized by the predominance of the finer fractions of soil particles.

2. Sandy Soils: These include all soils classified as sandy clay, sandy clay loam, sandy loam, loamy sand, sand, silty sand and gravelly sand and are characterized by the predominance of the coarser fractions of soil particles and, properties associated with the good internal friction and drainage.

3. Glacial Soils: These are the soils transported to their present location, in the geologic past, by the agency of glaciers and have been preconsolidated to a dense state by ice loads; these include all soils classified as till, gumbotil, glacial till and glacial clay, clay loam or loam. These soils usually are heterogeneous, unsorted, non-stratified mixtures of all sizes of soil fractions.

Further classification of silty clays is made here, on areal basis, by means of Principal Soil Association Areas[4] (Fig. 1). On the same basis Glacial Regions[1] (Fig. 2) are used to further classify the glacial soils. Within one area or region soils can be assumed to be fairly uniform for the purpose of fill construction. Use of similar information is recommended by Crawford and Thomas[2] and Johnson[3]. Johnson[3] feels that the research of previously composed data available along the highway location has a tenfold return on the time invested. Robnett and Thompson[5] have used such information to develop soil stabilization recommendations for Illinois.

The data obtained was depicted graphically for each soil association or glacial area. Data from soils showing similar graphs were combined together and then analyzed statistically. It was decided to use linear regression analysis as far as possible and then reduce the relations obtained to simple equations. This way the information could be kept in forms simple to use.

An interesting observation made during the analysis of the data was that for certain soils the values of shrinkage factors remained practically constant, irrespective of the volume of the material involved (Figs. 5, 6, 7 & 8). On the other hand, for some soils the values of shrinkage factors decreased to an extent, as the per mile volume of excavated material increased (Figs. 3, 4 & 9). The results obtained from the analysis were:

1. Silty Soils:

(a) Soils CKL, GH, OMT and SSM (Fig. 3)

$$S = 24 - 0.06\ v$$

(b) Soils D, F, KFC and TM (Fig. 4)

$$S = 45 - 0.133\ v$$

(c) Soils M and MIH (Fig. 5)

$$S = 35.5 \pm 3.0$$

(d) Silty Soils (Fig. 7)

$$S = 20.0 \pm 9.0$$

(In figure 1, the area CKL is shown as C)

2. Sandy Soils (Fig. 7)

$$S = 25.0 \pm 2.5$$

3. Glacial Soils

(a) Kansan Glacial Soils (Fig. 8)

$$S = 9.0 \pm 5.0$$

(b) Iowan and Tazewell Glacial Soils (Fig. 9)

$$S = 35 - 0.05 v$$

In these equations:

S = Shrinkage factor in percent and
v = Volume of excavated material in thousand cubic yards per mile
$\pm n$ = Indicates one standard deviation variation.

For the areas not covered in this analysis, no data or scant data was available. The data from Cary and Mankato glacial area showed too much spread in values and has not been included in this analysis. The reliability of any regression equation included in this paper is obvious from the variation shown by the points plotted on the shrinkage factor - volume relationship graphs (Figs. 3 through 9).

SUMMARY AND CONCLUSIONS

The data for this report was collected from the completed highway projects in various counties in Iowa. An attempt has been made here to make the determination of shrinkage factors possible by means other than pure engineer judgement and guesswork. The results of this investigation can be used as a more reliable guide than those presently available. There is, however, no guarantee that the values other than those predicted by the given equations will not be encountered in actual roadway fill construction.

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5. Robnett, Q.L. & Thompson, M.R., Concepts for Developing Stabilization Recommendations for the Soils of an Area, Highway Research Record 315, Highway Research Board, Washington, D. C., 1970, pp. 1-13.

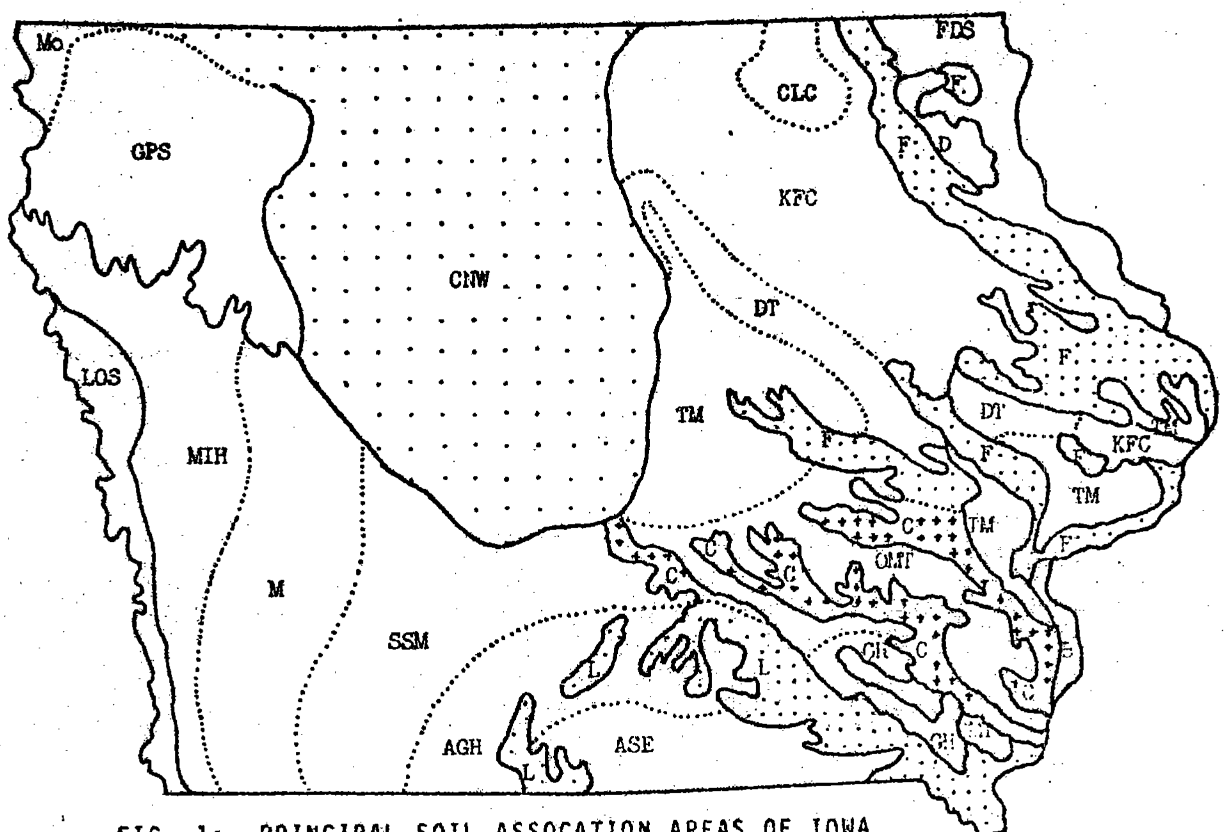

FIG. 1: PRINCIPAL SOIL ASSOCATION AREAS OF IOWA
(From Reference 4)

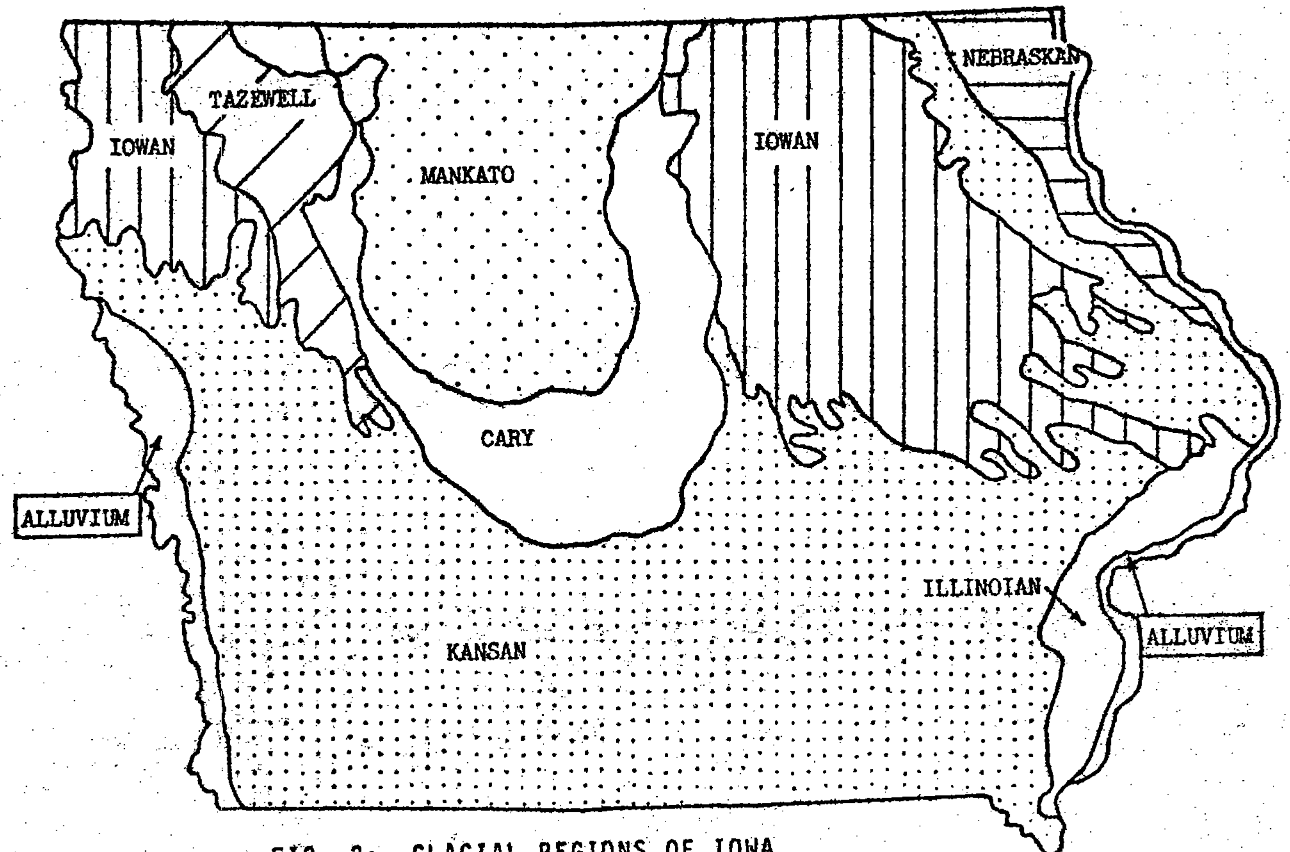

FIG. 2: GLACIAL REGIONS OF IOWA
(From Reference 1)

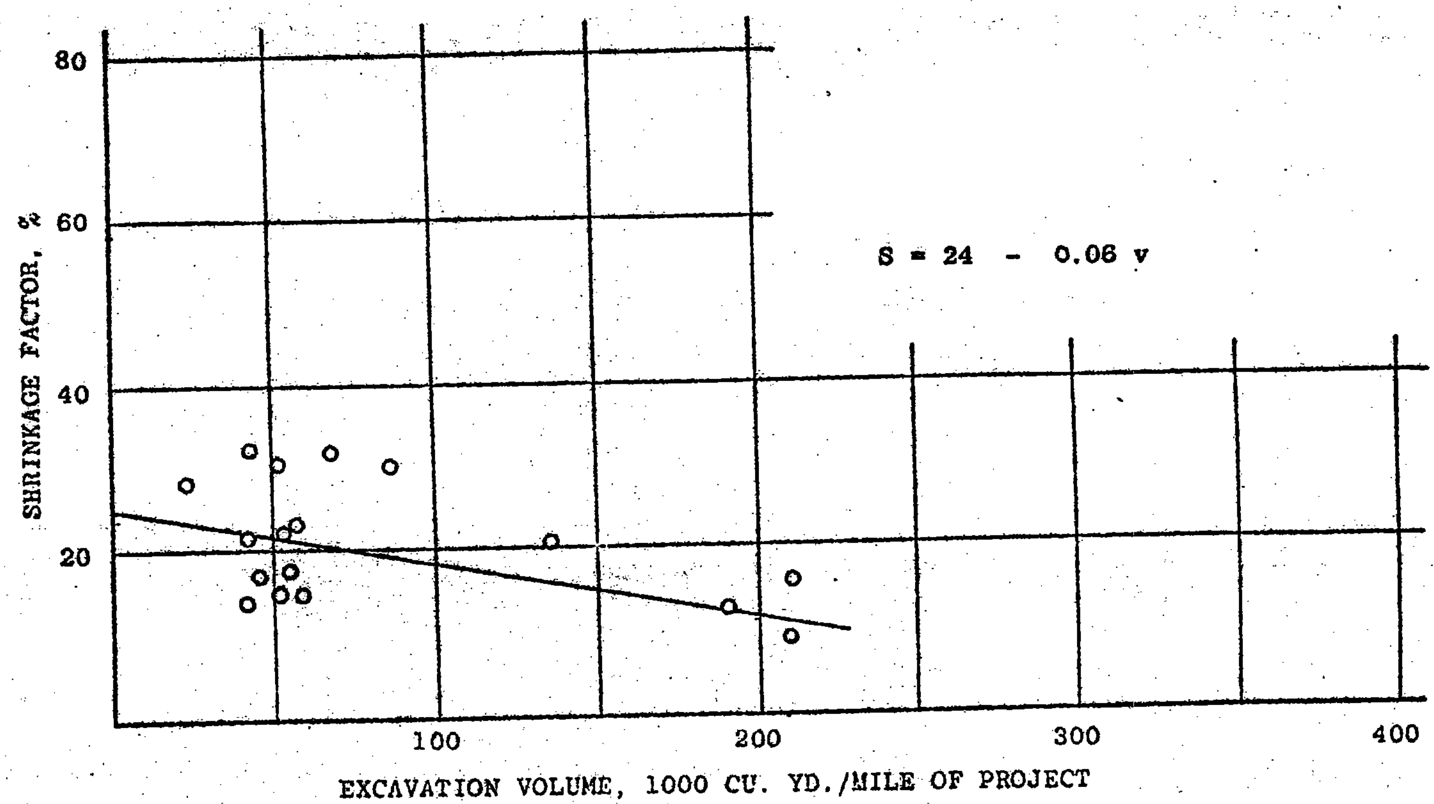

FIG. 3: SHRINKAGE FACTOR - VOLUME RELATIONSHIP FOR CKL, GH, OMT AND SSM SOILS

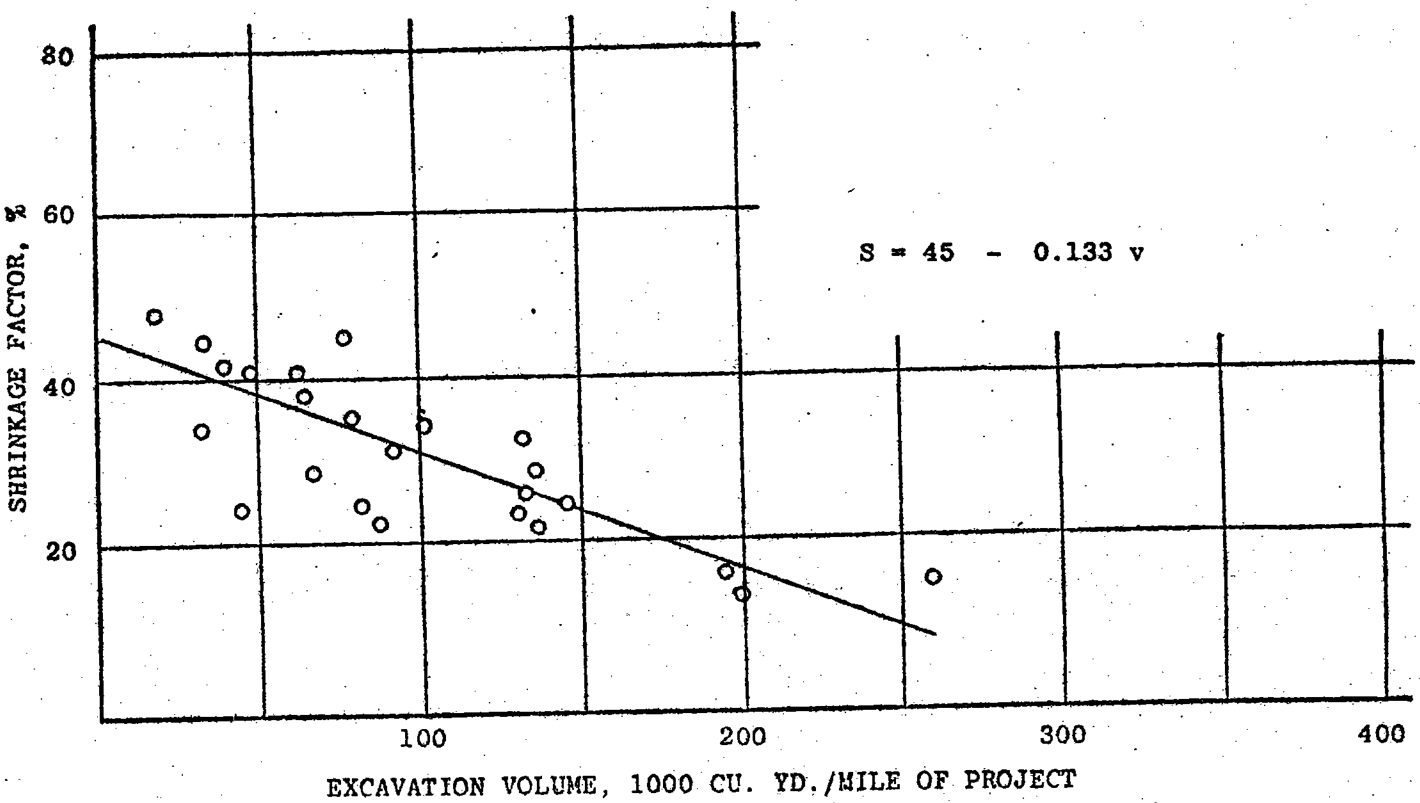

FIG. 4: SHRINKAGE FACTOR - VOLUME RELATIONSHIP FOR D, F, KFC AND TM SOILS

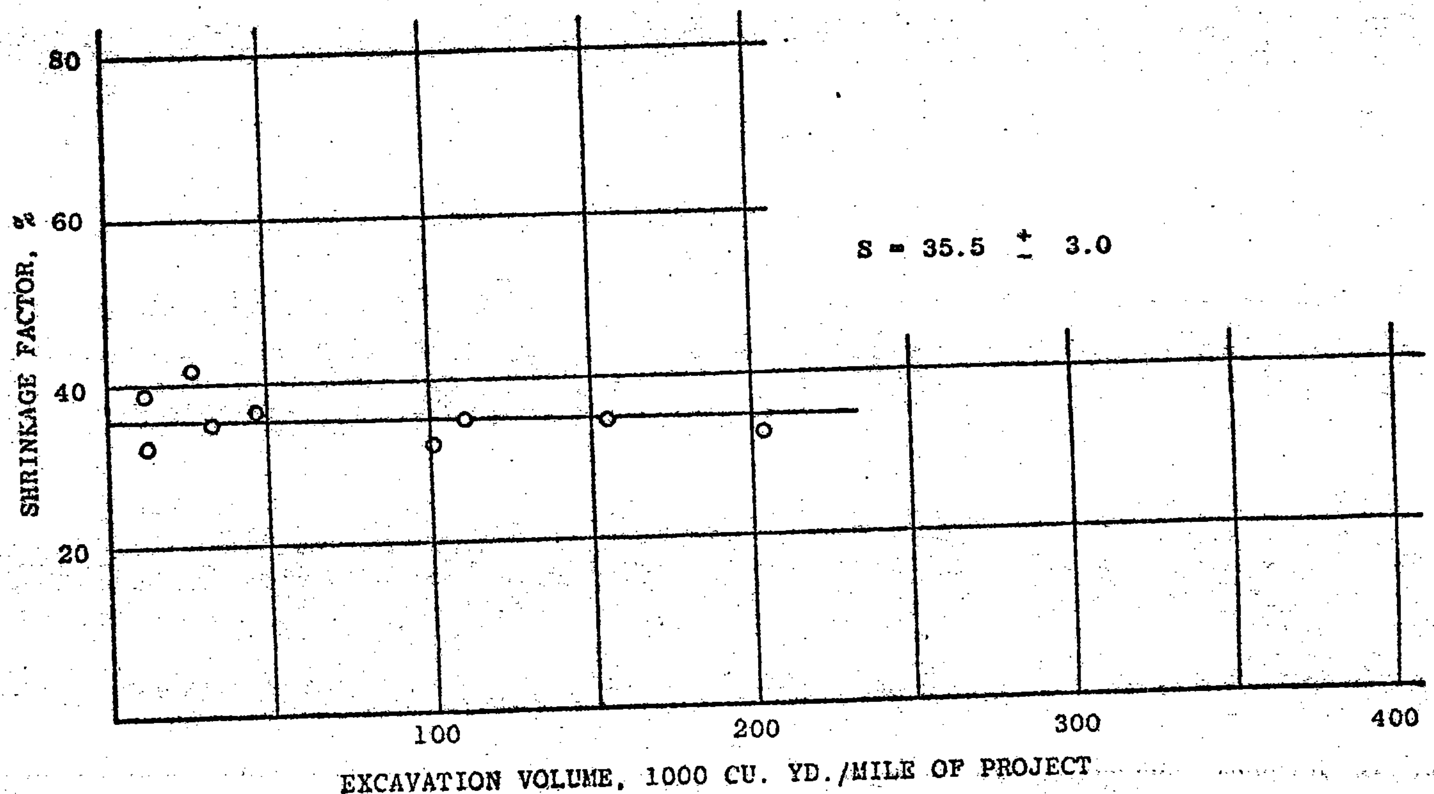

FIG. 5: SHRINKAGE FACTOR - VOLUME RELATIONSHIP FOR M AND MH SOILS

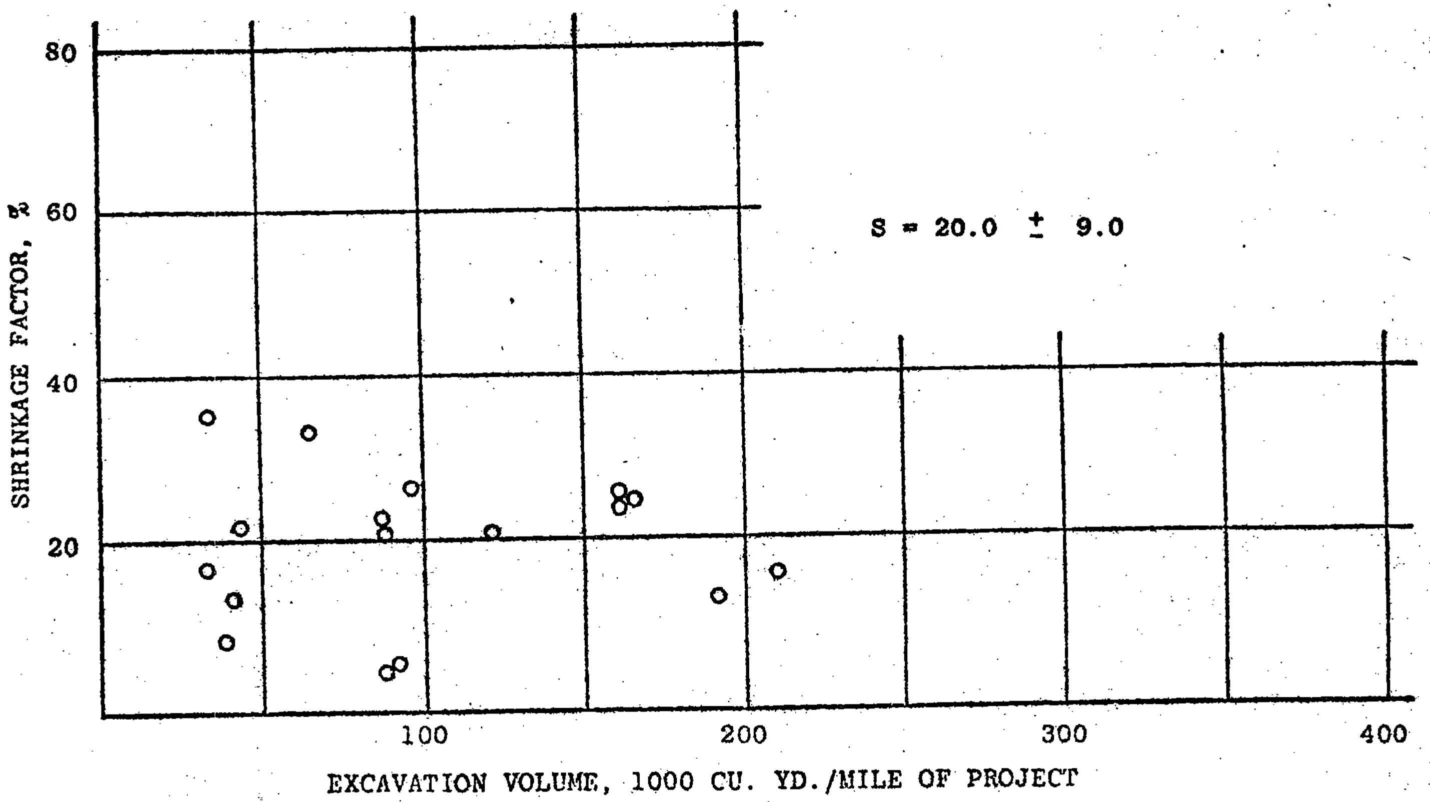

FIG. 6: SHRINKAGE FACTOR - VOLUME RELATIONSHIP FOR SILTY SOILS

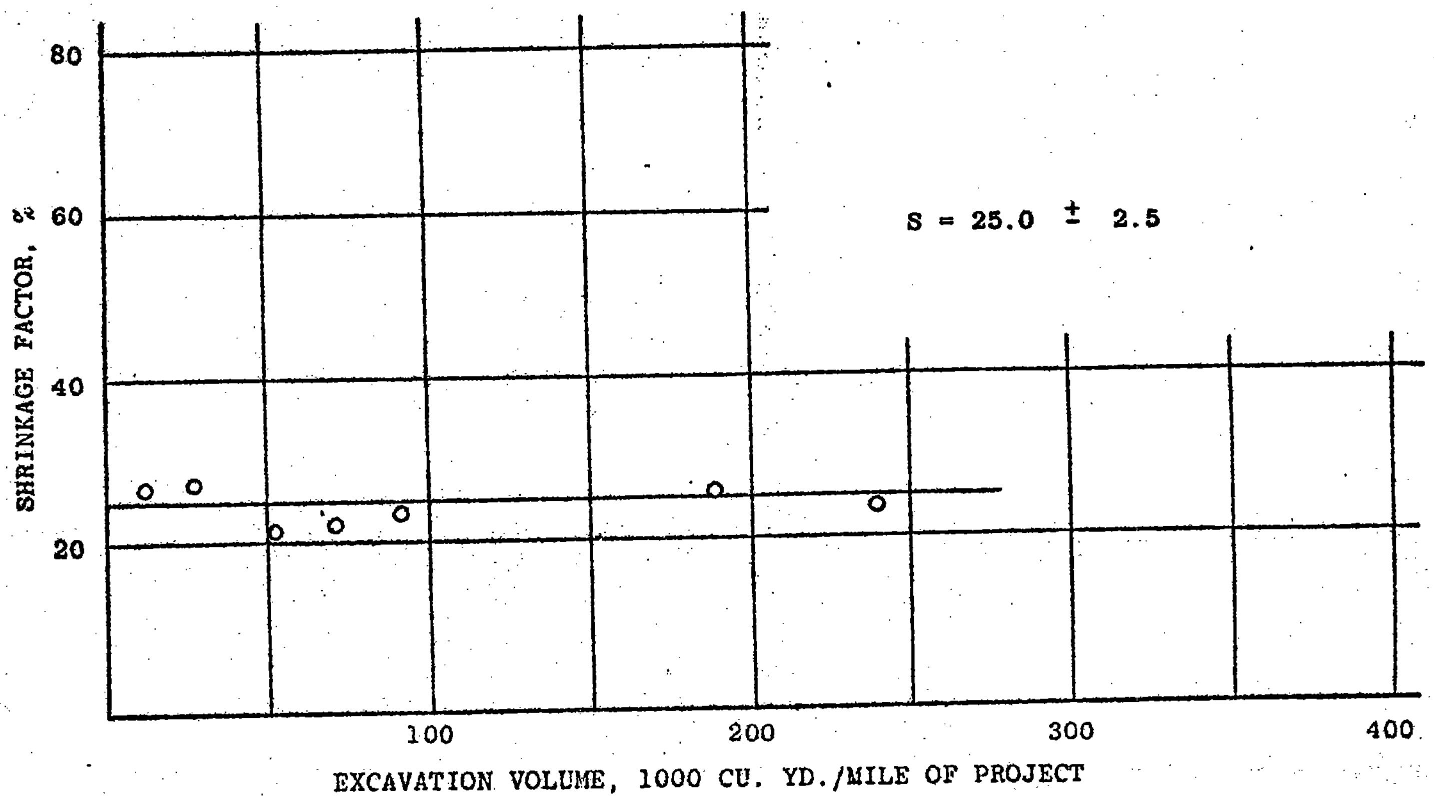

FIG. 7: SHRINKAGE FACTOR - VOLUME RELATIONSHIP FOR SANDY SOILS

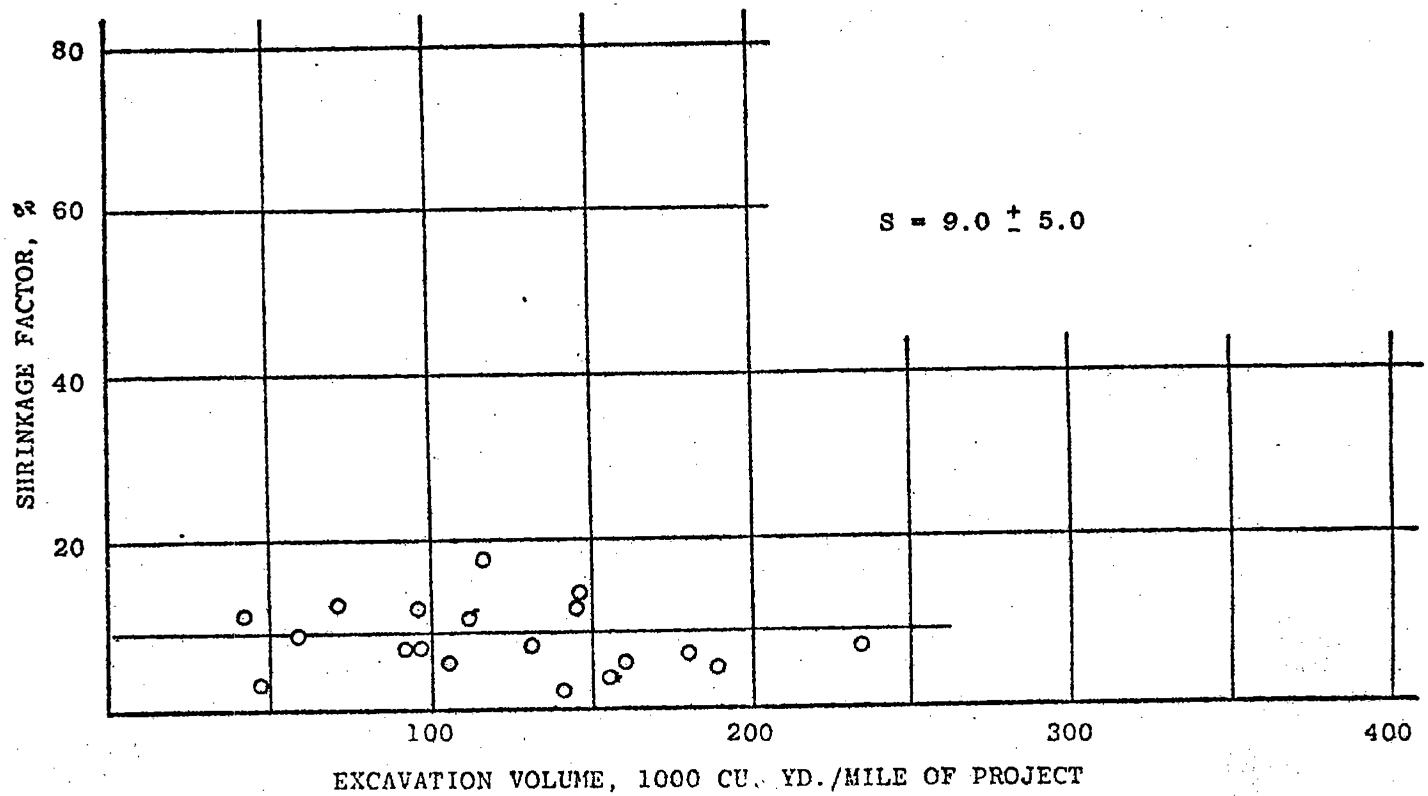

FIG. 8: SHRINKAGE FACTOR - VOLUME RELATIONSHIP FOR KANSAN GLACIAL SOILS

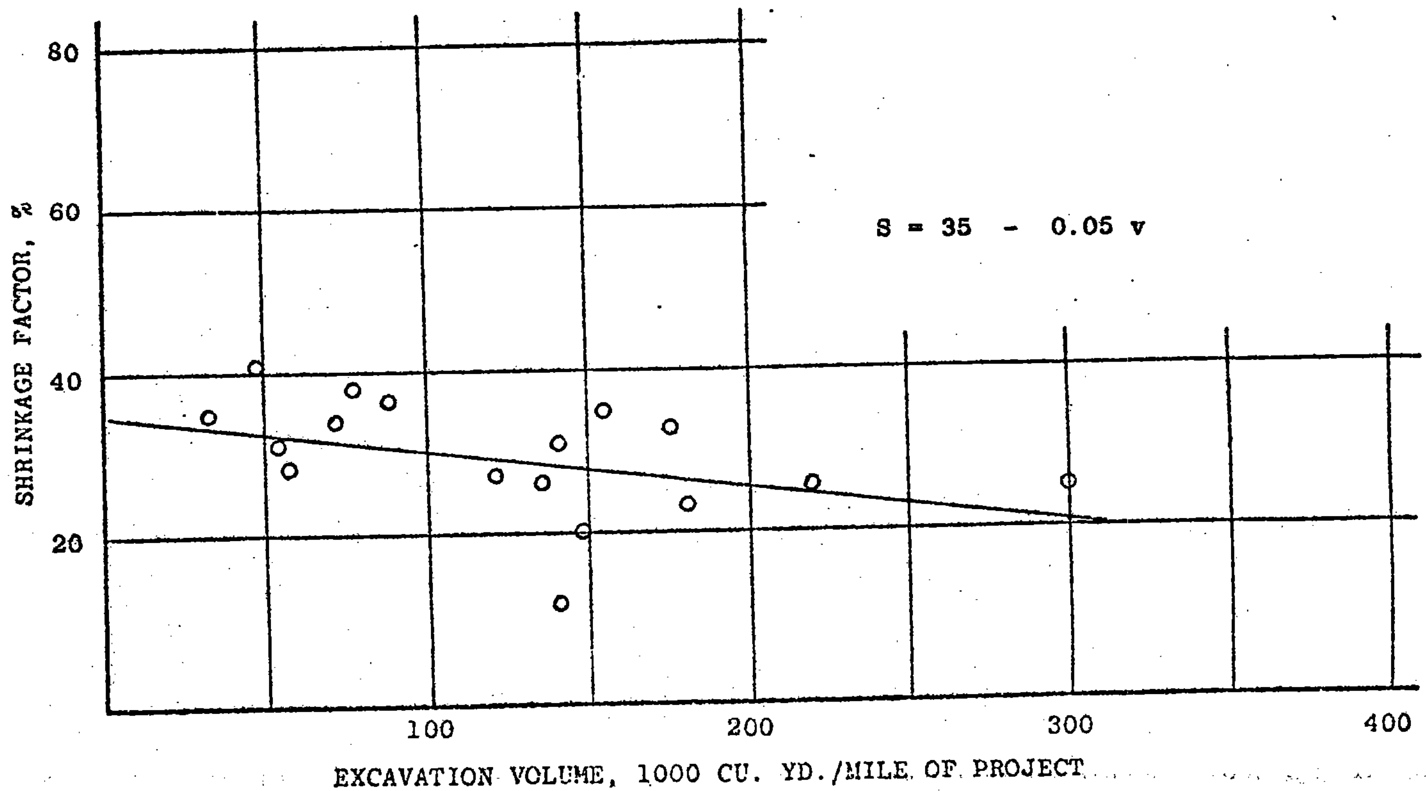

FIG. 9: SHRINKAGE FACTOR - VOLUME RELATIONSHIP FOR IOWAN AND TAZEWELL GLACIAL SOILS

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RESEARCH

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A RESUME OF A STUDY BETWEEN THE SOIL CONSERVATION SERVICE AND THE NATION AERONAUTICS AND SPACE ADMINISTRATION TO DETERMINE IF REMOTE SENSING COULD BE TECHNICALLY AND ECONOMICALLY DEVELOPED AS A BENEFICIAL TOOL TO ASSIST SOIL SCIENTISTS IN SELECTING SPOTS FOR FIELD OBSERVATIONS.

Thomas M. Hammond[1] and Horace F. Huckle[2]

1. National Aeronautics and Space Administration.
2. United States Department of Agriculture Soil Conservation Service.

A resume of a study between the Soil Conservation Service and the National Aeronautics and Space Adminstration to determine if remote sensing could be technically and economically developed as a beneficial tool to assist soil scientists in selecting spots for field observations, delineating soil boundaries, and to expedite soil mapping. This application of space technology, through research and development, experimented with sensory equipment remote from the area to be mapped by utilizing air-borne cameras, infrared detection devices, and multispectral scanning devices. Imagery was acquired by overflying test sites in Valusia County, Florida. Remote sensing data taken of the test sites was viewed to determine if there is a correlation between it and ground truth data.

No Report Available for Publication.

CURRICULUM VITAE

THOMMAS H. HAMMOND

Industrial Engineer - Graduated from the University of Texas. Served as Radar Weapons Controller in the U. S. Air Force. Served as Project Engineer and Chief of the Aircraft Engineering Branch, Kelly Air Force Base, Texas, accomplishing modification and maintenance on B-52's. Served as State Manager on Saturn Space Vehicle States at Kennedy Space Center. Member of the American Institute of Industrial Engineers and American Institute of Aeronautics and Astronautics.

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EVALUATION OF GRAVEL DEPOSITS USING REMOTE SENSING DATA, WABASH RIVER VALLEY NORTH OF TERRE HAUTE, INDIANA[1]

By

T. R. West[2]

S. A. Mundy[3]

&

M. C. Moore[4]

EVALUATION OF GRAVEL DEPOSITS USING REMOTE SENSING DATA, WABASH RIVER VALLEY NORTH OF TERRE HAUTE, INDIANA[1]

By

T. R. West[2], S. A. Mundy[3], and M. C. Moore[4]

ABSTRACT

Valuable gravel deposits adjacent to population centers are in danger of being lost to urbanization. Detailed exploration is needed to locate (within a larger body of coarse-grained sediments) those gravel-rich sites of sufficient size and quality which yield profitable gravel extractions.

Remote sensing provides a reasonably detailed means for rapidly exploring large areas. Satellite imagery (previously ERTS and now LANDSAT) can be used to study large areas of the land surface. Applying surface-materials-mapping techniques, those areas with good drainage (coarse textured soils) can be discerned from those which are more poorly drained. Spectral response of the surface materials in the visible and near infrared portion can be used to subdivide the scene into various categories of materials.

The test site, located north of Terre Haute, Vigo County, Indiana, is a terrace with more than 100 feet of glacial-outwash sands and gravels on the seat side of the Wabash River. In this study, the primary objective was to locate within the 20 square-mile terrace, those areas with the highest concentration of gravel. In general, in excess of 20% gravel (larger than 3/8" diameter) is needed to make a gravel operation profitable in Indiana.

Computer assisted techniques were applied to the LANDSAT imagery using both a supervised and non-supervised approach. Imagery taken at three (3) times of the year (Jan., June, Sept.) was geometrically corrected and overlaid (registered) to provide a single, multiple-channel data tape. A non-supervised analysis of these data proved to be more satisfactory for surface-materials classification.

Field checks of two (2) classes of material, one with a spectral response most similar to known gravel-rich areas, and the other with a very dissimilar response, were performed. Results, though not entirely convincing, suggest a higher gravel-content for those areas designated as gravel-rich on the classification.

Cost of remote sensing analysis as compared to conventional site exploration provided a final evaluation. For the 15,000 ± data points per wavelength band involved in the study the total cost including data aquisition, registration, geometric correction, computer analysis time and personnel, would be about $4900 using current cost figures. This cost, for an area of about 19.5 square miles, equals about one-half dollar per acre.

Cost of exploration for an 80 acre site is estimated at between $1,000-2,000 using conventional drilling which yields a value of $12.50 to $25.00 per acre. Of course, conventional exploration would still be needed after remote sensing studies but the amount of exploration should be greatly reduced. Remote sensing analysis has the potential to reduce exploration time and expenditures significantly for gravel extraction operations.

[1]This work was supported by National Aeronautics and Space Administration, Office of University Affairs under Grant No. NGL 15-005-186.

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INTRODUCTION

Gravel deposits which can be processed economically for construction materials are an important resource. Although gravel is not a rare geologic material it must, owing to the large quantities consumed by construction and its high cost of transportation relative to its low unit value, be located close to market for it to have commercial value. Such a material is said to have a high "place value" (1, p. 93).*

Because of these considerations, a maximum hauling distance of only a few tens of miles is allowed for competitive pricing. To complicate the problem further, gravel is needed close to urban centers where the majority of construction occurs. As urbanization spreads from population centers, gravel-rich areas are commonly covered by housing tracts or commercial development. In addition, as urbanization proceeds, operating gravel pits locally encounter public pressure to minimize noise, dust, traffic, and other environmental factors which act as a deterrent to continued gravel production and future pit expansion.

Economics in Indiana dictate that a viable gravel-extraction operation have a 20-year supply of material and, based on a reasonably small extraction of 200,000 tons per year, an average thickness of 15 feet and a recovery of 80 percent, an area of some 80 acres is needed. For smaller, short-term work, such as a highway construction project in a rural area, much smaller volumes of gravel may prove economical to use. Also, with the steady approach of urbanization in an area, pits with the same thickness of gravel but only 10 acres in size operating for a two (2) year period may prove both economical and necessary.

Sand and gravel production in Indiana had a value of nearly $37 million in 1973 (Table 1) or 11% of the state's mineral production for that year. Some 28 million tons of sand and gravel were produced in 1973 at an average value of $1.30 per ton. The total value of gravel production is significantly greater in many of the more populous states and in some areas a shortage of gravel exits.

*Numbers in ( ) refer to the list of references presented at the end of this paper.

Because of this, the Federal Highway Administration, U. S. Department of Transportation, has recently solicited research proposals on the use of airborne geophysical techniques for the purpose of locating contruction materials (3).

TABLE 1. Gravel Production in Indiana

Year	Total Value of Indiana's Mineral Production ($1000's)	Total Value of Indiana's Sand & Gravel Production ($1000's)	Total Sand and Gravel (1000's of short tons)	Value of Sand & Gravel as % of Value of Mineral Production
1963	202,530	20,683	22,840	10.2
1964	211,783	21,811	24,416	10.3
1965	218,567	22,220	24,867	10.1
1966	230,010	23.542	24,992	10.2
1967	244,921	25,588	26,265	10.4
1968	235,386	26,160	25,774	11.1
1969	241,871	27,438	26,218	11.3
1970	228,786	25,796	23,476	11.3
1971	281,521	29,094	24,982	10.3
1972	322,608	33,290	27,978	10.3
1973	340,915	36,734	28,257	10.7
1974	344,989	34,694	25,421	10.1

Based on data from the U. S. Bureau of Mines (2).

The increasing worth of sand and gravel in Indiana is illustrated in Figure 1. Sand and gravel production in 1,000's of tons and in 1,000's of dollars of value is plotted for the years 1963-74 based on data from Table 1. The cross over point in 1967-68 can be noted where sand and gravel worth exceeded $1.00 per ton. Of major significance is the slope of the two (2) curves from 1970-74. The comparative steepness of the curves indicates the rapid growth in value per ton of sand and gravel caused mostly by inflation. Hence it is to be expected that the worth of sand and gravel will continue to increase considerably per ton in the next decade. The effects of reduced construction activity is shown by the 1974 data. However, the slope of upper plot is not as steep as that of the lower one for 1974, showing an increase in unit cost.

Because of the increasing demand for construction materials, the loss of gravel extraction sites due to urban sprawl, and the high place-value of gravel deposits, it had become necessary for gravel producers to increase their exploration efforts to locate new sources. Cost of exploration for an 80-acre site is estimated at between $1000 and $2000 using conventional drilling augmented by subsurface geophysical techniques. With these comparative costs in mind, it is apparent that if remote sensing analysis can reduce the costs for exploration by designating prime areas for detailed study, a valuable savings can be made.

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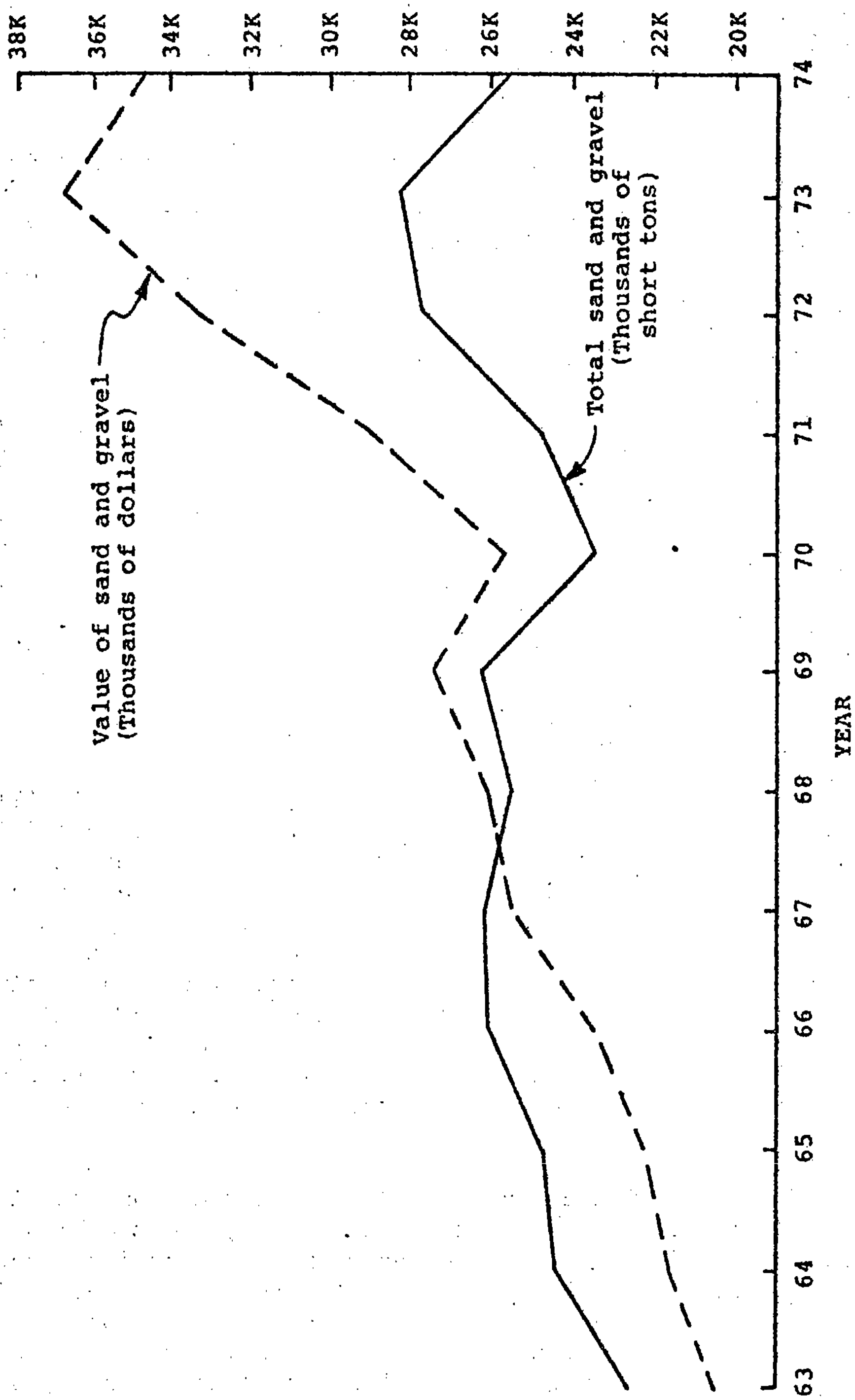


Figure 1. Comparison of Indiana's sand and gravel production by year versus sand and gravel value.

Evaluation of Gravel Deposits Using Remote Sensing Data, Wabash River Valley North of Terre Haute, Indiana.

SETTING OBJECTIVES

A test site located north of Terre Haute in Vigo County, Indiana was selected for study (Fig. 2). The use of remote sensing to locate gravel supplies was suggested by LARS and the Indiana Geological Survey because of the significance of the problem and the special potential of remote sensing for solving it. Study results would influence decision making relative to resource inventory, mineral extraction and land use planning.

The study site is a river terrace on the east side of the Wabash River and immediately north of Terre Haute (Fig. 2). The bedrock valley of the Wabash River was filled with more than 100 feet of sand and gravel (glacial out-wash), which was deposited as the glacial melt-water flow diminshed. The surface of this deposit in the Wabash Valley is known as the Shelbyville Terrace, and lies about 75 feet above the present Wabash River.

Massive glaciers advanced into Indiana at least three (3) times during the Pleistocene Epoch. The earlier invasions extended far south of Vigo County, but the last ice sheet stopped just to the north of Terre Haute. Each glacier deposited till from the ice and also produced large quantities of sand-and-gravel-bearing outwash. From a meandering stream, such as the present-day Wabash, the coarsest gravel is deposited on the insides of the bends, and the deposit builds toward the center of the channel along bars. However, a glacial meltwater stream, which is highly charged with sediment, deposits the coarse material within the center of a broad flat, braided channel in the form of diamond-shaped bars that are elongated in the direction of flow.

In this study the primary objective was not merely to locate sandy materials which hopefully contain some gravel because it was already known that essentially all the Shelbyville Terrace at Terre Haute has some gravel present; but instead, to locate the bars bearing the coarsest gravels. The area of study was some 20 square miles, and about 1/2 of it consisted of the Shelbyville Terrace.

The proportion of gravel necessary to yield an economical operation varies with the intended use of the gravel and the market for the remaining sand. Generally for Indiana, few pits operate with less than 25% of the material being larger than 3/8" in diameter. Pits with 80% sand containing a good masonry sand, or with a market for large volumes of fill sand, however, do operate at a profit. Therefore, it was concluded that locating deposits with a minimum of 20 to 25% gravel (larger than 3/8" diameter) would be the objective of this analysis.

PROCEDURE

Remote sensing is the science of aquiring information about objects from measurements taken at a distance without physical contact with those objects. The primary means of transmission from object to observer is by electromagnetic fields. At LARS, studies are concentrated on the portion of the electromagnetic spectrum which includes the visible-reflective infrared-, and thermal infrared wavelengths with special emphasis on machine (computer) processing of these spectral data.

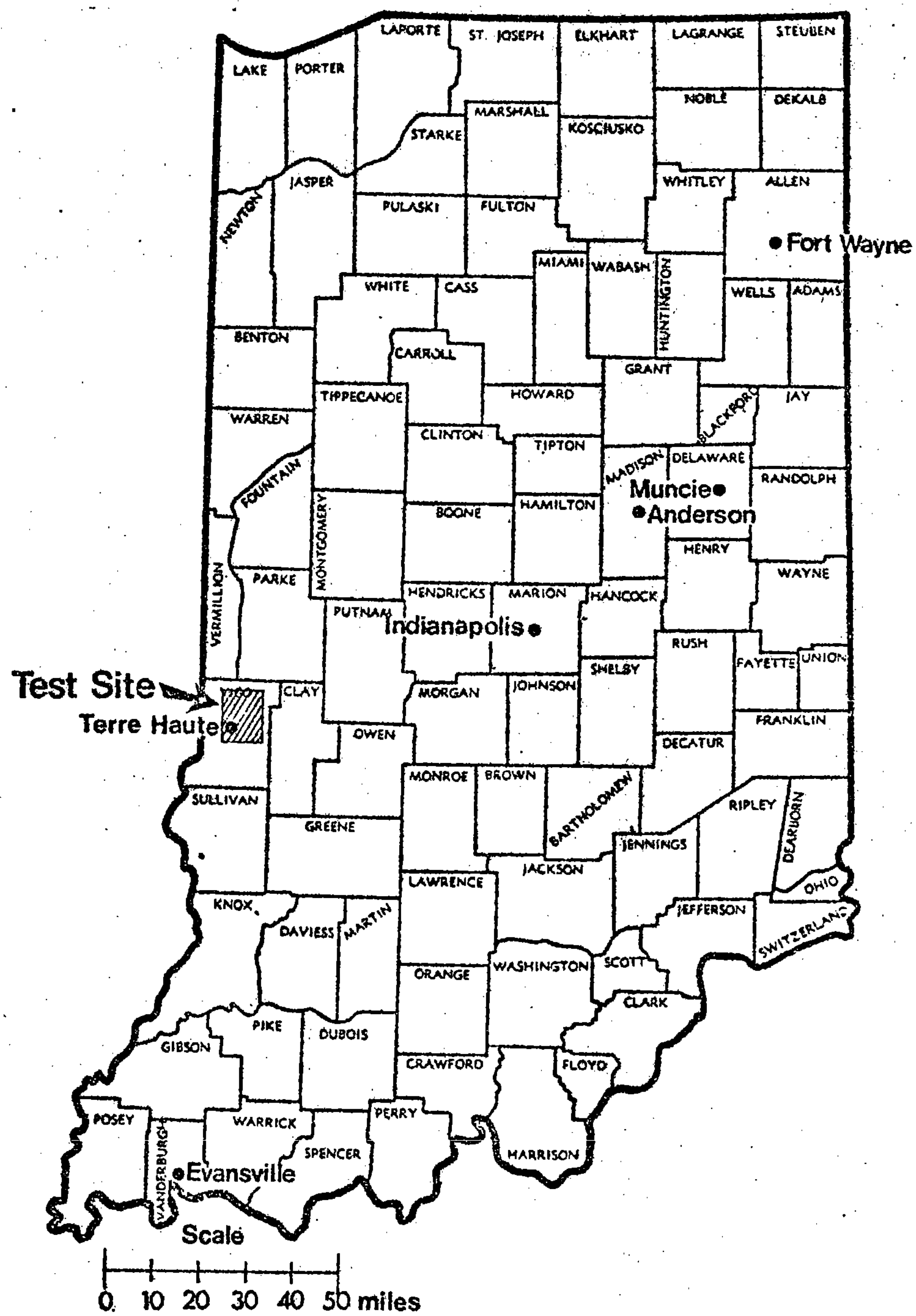


Figure 2. Location map, Terre Haute, Indiana Study Site.

The primary remote sensing data used in this study consisted of three (3) sets of LANDSAT imagery obtained during the months of January, June and September, 1973. Previously, these data had been geometrically corrected and the imagery for the three (3) LANDSAT passes overlaid to provide a single, 12-channel data tape. This combined-data tape served as the basis for land use mapping of Vigo County, Indiana in an earlier study (4) thus reducing data aquisition costs for the current research.

In addition to the LANDSAT data, high altitude (6,000') 1:120,000 scale, infrared photography of Vigo County, flown in May 1971, was also available at LARS. A surface geology map of the area north of Terre Haute was developed by the authors, from published information and compiled on two (2) 7 1/2 minute (1:24,000 scale) topographic maps, the Rosedale and New Goshen quadrangles. A smaller scale, somewhat generalized map, based on that surface geology information, is presented as Figure 3. The locations of eight (8) known gravel-extraction operations are also plotted on the quadrangle maps.

The initial step was to locate accurately the study area in the LANDSAT data. This can be time consuming because of the limited scale and resolution of the imagery. Use was made of the visual display unit at LARS, recognizing the features of the Wabash River and adjacent drainage patterns. Also, the 1:60,000 scale infrared photography proved particularly helpful in determining the specific location within the LANDSAT imagery.

After the study area of 108 lines by 116 columns (4.4 miles by 4.5 miles) was designated in the LANDSAT data, an attempt was made to determine surface features distinctive of gravel deposits. Some of those considered were vegetative cover, drainage pattern, soil cover, and temporal changes in these features in the January, June and September imagery. If distinctive features were found, a unique, spectral signature of the gravel deposits could be isolated through computer-assisted analysis. Figures 4a, 4b, and 4c show the variations between seasons in the Shelbyville Terrace near the center of the study area, using filters to simulate a false color-type image of spectral response. The colors and pattern in the terrace area change with the seasons showing that temporal LANDSAT imagery provides additional detail for purposes of analysis.

The first computer-assisted analysis of data involved a non-supervised or clustering approach. Seven (7) spectral classes were specified with each data point assigned to that cluster class of the seven (7) showing the greatest spectral similarity. Thus, water might be one class, vegetative cover another, bare soil a third, with similar properties comprising other classes. Clustering was performed on the January, June and September parts individually and also on the combined, overlay tape consisting of all twelve (12) channels of data. If a unique gravel class exists, this classification would likely point it out.

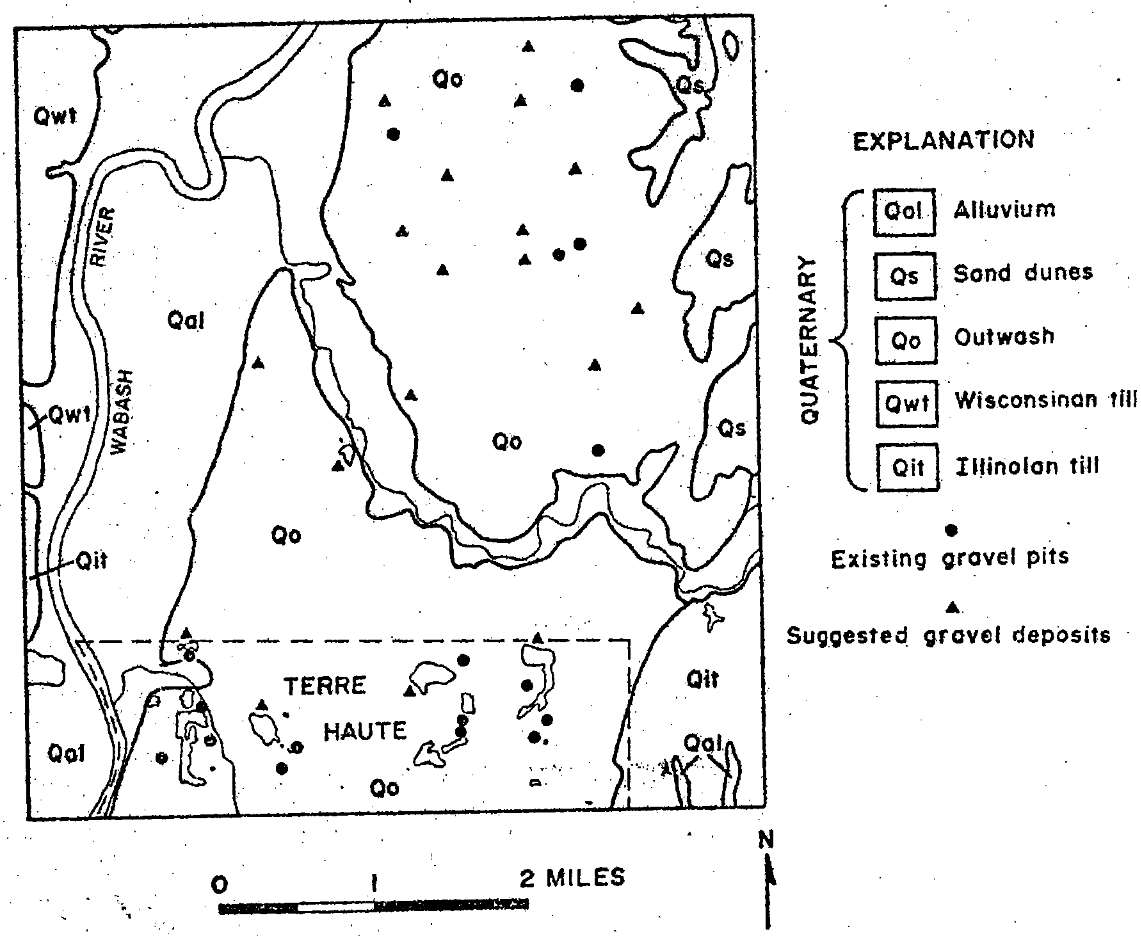


Figure 3. Surface Geology Map of Area North of Terre Haute, Indiana

The second computer-assisted analysis was a supervised approach in which the computer was programmed to recognize a specific signature of known materials outlined for it in the data. Based on the statistics of the designated area (means, standard deviation) and applying a normal or Gaussian distribution, points possessing similar statistical descriptions in other portions of the study area are located. To program the computer, eight (8) sites of present-or recently-abandoned-gravel operations were designated. To obtain the highest location accuracy (position accuracy was within one or two resolution elements) each site was represented by nine (9) pixels (resolution elements), the central one plus its eight surrounding, nearest neighbors. Also, ten (10) pixels correspond to an area of approximately ten (10) acres which is about the minimum size of an economically feasible gravel extraction site, even in a suburban area.

Using the eight (8) known areas containing gravel as training sites for the computer, the entire area was classified. As only one training class was considered (gravel), the entire study area must be included in this single class but to insure that only similar materials would persist, a threshold of 75% was applied to the classification. Thresholding, in effect,deletes the tails of the Gaussian (normal) distribution which in this case leaves only the central portion (25%) of the distribution remaining. This classification was performed using the individual LANDSAT frames for the three (3) seasons plus the combined twelve-channel data tape. Despite the severe curtailment by thresholding, much of the study area was still designated as within the gravel class.

CLASSIFICATION RESULTS AND FIELD EVALUATION.

The supervised approach described in the previous paragraphs did not accomplish the desired results. On all three (3) of the one-season LANDSAT frames, more than half of the total area was assigned to the gravel class despite the 75% threshold. The classification did not single out locations possessing a high gravel content or even designate those where sand and gravel predominated. Instead, not only was the terrace assigned to the gravel class, but also parts of the floodplain and ground moraine. The supervised classification of the twelve-channel overlay of these three (3) frames, however, yielded better results although specific areas of high gravel content were not designated. Outlined instead was the entire sand-gravel Shelbyville Terrace excluding the landforms covered by finer textured soils (floodplain and ground moraine). Even though the desired result of locating high-gravel content areas within the terrace was not accomplished, the results demonstrated that coarse textured materials (sand and gravel) could be distinguished from fine textured ones (silts and clays) using the classification of overlay data. It is believed that the poor results arose from an insufficient number of test sites used, and that those sites included were too small. However, more data would not normally be available in other potential areas.

The non-supervised analysis proved more successful than the supervised. Seven (7) classes were used to cluster the three (3) LANDSAT frames and the 12-channel overlay combination (Fig. 5). As with the supervised classification, the 12-channel overlay data yielded better results. On all four (4) classifications the

4a. January

4b. June

4c. September

Figure 4. Digital display images of January, June, and September, 1973, Landsat data, Terre Haute, Indiana Test Field. Simulated infrared imagery (black and white reproduction of color original).

existing gravel pit locations and the area adjacent to the Wabash River were represented by a common symbol. Apparently the high moisture area of the river had a seasonal variation in spectral response similar to that of the coarse gravel deposits in the terrace. The 12-channel classification was judged superior to the others because it contained less scatter of these "gravel cluster" symbols. Other sites in addition to the existing gravel pit sites were delineated in the terrace deposit by the gravel-cluster symbol (designated by the letter W but blackened over for emphasis). These areas were judged to have a high gravel-to-sand ratio. After reviewing the classification results soil auger holes were drilled at four (4) locations with the Indiana Geological Survey power auger. The results in general seemed to indicate that the gravel-rich classification was reasonably correct as gravel was found in three (3) of the four (4) holes within 8 feet of the surface.

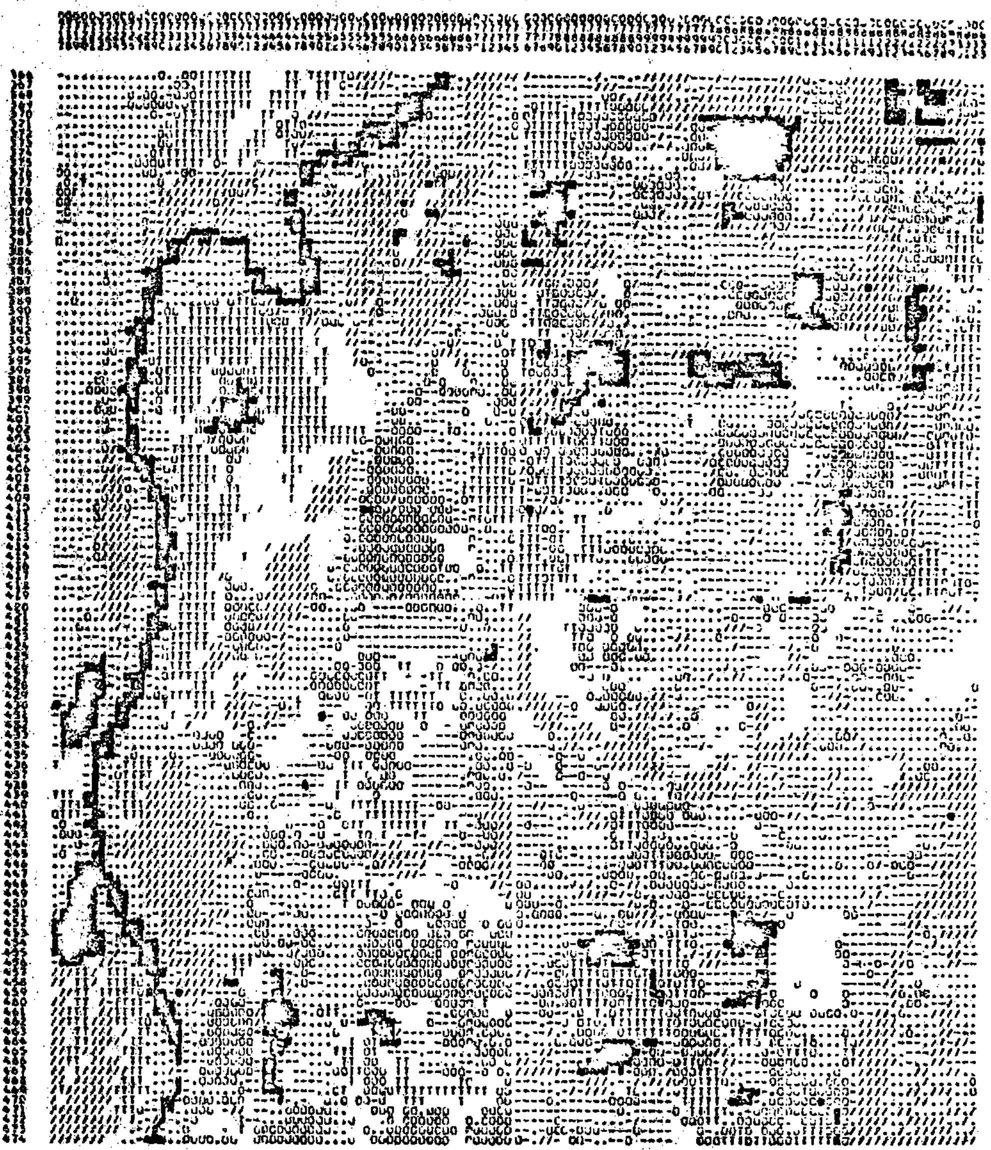
To test the possibility that similar gravel deposits existed in many other locations in the terrace but were not outlined by the classification, the cluster class most dissimilar to the "gravel cluster", (the class among the remaining six (6) which showed the greatest spectral difference from the gravel clusters) was singled out for review to determine if indeed, it corresponded to a different soil material. This class, (designated by T in Figure 5) should have the lowest probability of indicating a gravel-rich area. Seven (7) sites were designated in the terrace where the T cluster symbol was present. The drilling results at these sites in general showed that gravel did not occur within the uppermost 8 feet to as great an extent as it did in the first series of holes.

The evaluation of gravel content in the drilled holes is somewhat subjective as the materials were obtained with a continuous flight auger which is far from ideal for sampling sand and gravel materials. However three (3) of the four (4) "gravel-designated" borings contained the coarsest material throughout the total section drilled while the fourth hole had only a small amount of gravel present. By contrast only one (1) of the "non-gravel" borings had abundant gravel present but this was located below a clayey, surface material.

CRITIQUE OF RESULTS AND RECOMMENDATIONS FOR FUTURE WORK

Though results of the gravel study are encouraging, other aspects deserve consideration. Cost of remote sensing analysis as compared to that of more conventional site exploration is important. Data aquisition of the three (3) LANDSAT frames, image registration (overlay) of the three (3), plus geometric correction for each data point had been accomplished for this site in a previous study at LARS. For the 15,000 $\pm$ data points per channel involved in this Terre Haute study the cost of data aquisition, registration, and correction would total about $3000 using a current costs.*

*The actual cost for the one-fourth ERTS frame covering Vigo County, of which this Terre Haute gravel study represents nearly one-half the area, was approximately $14,000 when performed on an experimental basis in 1974.



Dark areas - suggested sites with high-gravel percentages
Symbol T - suggested sites with low-gravel percentages

Figure 5. Non-supervised classification of 12 channel overlay of LANDSAT data, January, June, and September 1973

Cost of computer time to analyze the data plus the cost of personnel totaled about $1900. Therefore, a reconstructed cost to perform the study, using current LARS charges would total $4900.

Unit cost per site would decrease significantly if a number of such tracts within a county or other large division of the land surface were being studied at one time. The $4900 cost is for an area of about 19.5 square miles which is about $240 per square mile or less than one-half dollar per acre. In addition, if this technique were placed in an operational mode, a larger, faster computer than the IBM 360 Model 67 (now employed at LARS) could be used, resulting in substantial savings in computer charges. Finally, as previously stated, the estimated cost for detailed exploration of an 80 acre tract using conventional methods is from $1000 to $2000 or $12.50 to $25.00 per acre. This total might be substantially reduced if remote sensing analysis was performed first.

Further study should be undertaken to obtain computer training sites where LANDSAT data were obtained prior to extraction of gravel material. In the current study, the training sites were operating and abandoned gravel pits where obviously a high concentration of gravel existed. A more-typical spectral signature would be obtained for an undisturbed area but the existence of high-gravel-bearing material would have to be established for that training site. Better sampling techniques and more sampling points would be needed to obtain a statistically significant evaluation for the presence or absence of gravel in the near subsurface.

The final problem of note is the limited resolution of LANDSAT imagery compared to the detailed desired in this study. Image resolution elements for LANDSAT are approximately one acre in size. In contrast, multispectral scanners mounted in aircraft obtain imagery with much greater resolution. For example, a 3,000 foot flight yields an image resolution element 9 feet by 9 feet. However, there are disadvantages associated with aircraft data. One of the most significant is the narrow, one-mile wide strip of data obtained. In flights at higher elevations, the path width increases but the resolution decreases. For the four (4) mile wide area involved in the Terre Haute study, a 12,000 foot flight would have been idal, yielding resolution elements approximately 36 feet square as compared to LANDSAT resolution elements of one acre or 210 $\pm$ feet square. A consideration of vehicles other than LANDSAT is needed for collecting multispectral data for land surface studies in which minute detail is important.

A follow-on study building on the analysis techniques presented in this paper but applied to multispectral data possessing greater resolution is needed. The results would be evaluated using statistically significant sampling techniques to prove more conclusively the utility of locating gravel-rich areas by analysis of remote sensing data.

CONCLUSIONS

From this study it is concluded that the non-supervised classification of the LANDSAT, 12-channel overlay data was able to designate reasonably well those areas with a high probability of possessing gravel-rich zones near the surface. The supervised classification was also able to designate the broad categories of coarse textured soils (here terrace deposits) from other fine textured soil areas (floodplain and ground moraine).

It is concluded that this technique as augmented by available geologic, agricultural soils and photographic information can be used to locate areas with a higher potential for gravel deposits of an economic value. Subsequently, field studies employing the electrical resistivity method, followed by subsurface drilling can be used for successive refinement of the areas designated as gravel rich, based on the remote sensing study. The combined approach of remote sensing and conventional exploration should result in a savings of time and expense as compared to a totally conventional method.

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UNIVERSITY EDUCATION

Degree	Major	Minor	University	Date
AB	Geology	Chemistry	Washington U.	1959
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MA	Geology	Civil Engineering	Washington U.	1962
MSCE	Civil Engineering	Engineering Geology	Purdue Univ.	1964
Ph.D.	Engineering Geology	Soil Mechanics	Purdue Univ.	1966

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Graduate Assistant, 1959-1961, Washington University, Laboratories in Physical Geology, Historical Geology, Optical Mineralogy, Petrography.

Instructor, 1961-1966, Purdue University, courses taught - Mineralogy, Petrology, Geology for Engineers 1, Physical Geology, Historical Geology, Geology Seminar.

Assistant Professor, 1966-1971, Purdue University, courses taught- 1) undergraduate level, same as for instructor 2) graduage level, Geology for Engineers II, Advanced Engineering Geology, Petrography of Aggregates

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Publications:	Economic geology of aggregate resources Glacial Pleistocene stratigraphy Karst and caves.
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University experience included teaching assistantship in graduate and undergraduate level courses in surveying, photogrammetry, geodesy, and least squares adjustment. Experience also included a research assistantship at the Laboratory for Applications to Remote Sensing (LARS). Summer experience in topographic and construction surveying.