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CIVIL ENGINEERING
INDIANA
DEPARTMENT OF HIGHWAYS

JOINT HIGHWAY RESEARCH PROJECT

JHRP-84-12

GEOTECHNICAL ENGINEERING PROBLEMS
IN THE KARST REGION OF SOUTHERN
INDIANA

FINAL REPORT

Francis T. Adams and C. W. Lovell

PURDUE UNIVERSITY

JOINT HIGHWAY RESEARCH PROJECT

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Final Report

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IN THE KARST REGION OF SOUTHERN INDIANA

To: H. L. Michael, Director
Joint Highway Research Project

July 3, 1984
Project: C-36-51B

File: 1-5-2-66

FROM: C. W. Lovell, Research Engineer,
Joint Highway Research Project

Attached is the Final Report on the JHRP approved research study "Geotechnical Engineering Problems in the Karst Region of Southern Indiana" by F. T. Adams and C. W. Lovell of our staff. It was prepared under my direction.

This report may be used in conjunction with JHRP-8-7, "Engineering Soils Map of Lawrence County, Indiana" by F. T. Adams to produce guidance in decisions of location, design and construction in areas of limestone bedrock. The report is a complete and concise statement of existing knowledge relative to geotechnical problems and their resolution in karst areas. We expect that it will be a very valuable reference for the Indiana Department of Highways, and others.

Respectfully submitted,

C. W. Lovell

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IN THE KARST REGION OF SOUTHERN INDIANA

by

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Joint Highway Research Project
Engineering Experiment Station
Purdue University

In Cooperation with the

Indiana Department of Highways

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the material presented herein.

Purdue University
West Lafayette, Indiana
July 3, 1984

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TABLE OF CONTENTS

	Page
LIST OF TABLES	vi
LIST OF FIGURES	vii
LIST OF ABBREVIATIONS AND SYMBOLS	xi
HIGHLIGHT SUMMARY	xvi
INTRODUCTION	1
Engineering Problems in Karst Regions	1
Objectives	3
ASSESSMENT OF ENGINEERING PROBLEMS OF LAWRENCE COUNTY	6
Rating of Engineering Problems	6
Description of the Mitchell Plain	10
ORIGIN OF ENGINEERING PROBLEMS IN KARST REGIONS	15
Definition of Karst Topography	16
Chemical Weathering Process in Limestone and Dolomite	16
Prerequisites for Karst Development	19
Karst Landforms	22
Development of Subsurface Cavities	27
Evolution of Sinkholes	35
Classification of Sinkholes	41
Irregular Weathering of the Bedrock Surface	42
Development of Plastic Residual Soils	43
SITE EVALUATION IN KARST REGIONS	45
Methodology	47
Preliminary Studies	50
Reconnaissance Surveys	52
Geologic Indicators of Sinkhole Development	57
Preparation of an Engineering Soils Map	59
Preparation of a Sinkhole Density Map	61
Identification of Lineaments	63
Selection of Preliminary Site(s)	68
Field Site Investigation	69
Preliminary Reconnaissance Techniques	70
Ground-Probing Radar	70
Terrain Conductivity	72

	Page
Detailed Surveys	73
Gravity	73
Magnetics	75
Seismic Refraction	76
Electrical Resistivity	77
Other Methods	80
Evaluation of Geophysical Methods	82
Location of Borings	86
Depth of Borings	89
Boring Types	90
Evaluation of a Probabilistic Approach	90
 HYDROLOGIC ASPECTS OF PROBLEMS IN KARST REGIONS	 92
Case Histories	93
Groundwater Withdrawal	93
Alteration of Surface Drainage	95
Mechanisms of Subsidence and Collapse	96
Solutions and Construction Techniques Applied to	
Hydrologically-Induced Problems	99
Evaluation of Hydrologic Data	100
Corrective Procedures	101
Methods Dealing with Groundwater Withdrawal	102
Control of Surface Drainage	104
Subsidence and Collapse Problems	104
Flooding Problems	108
Location of Wells in Karst Regions	113
Conclusion	115
 ENGINEERING DESIGN AND CONSTRUCTION IN KARST REGIONS . .	 117
Engineering Classification of Carbonate Materials	118
General Foundation and Stability Requirements	121
Foundation Failure Modes	123
Allowable Bearing Pressure	125
Slopes and Underground Excavations	125
Qualitative Assessment	127
Design Guidelines	129
Construction Methods	140
General Foundation Schemes	140
Compacted Soil Fill	144
Arching Theory	145
Design Method	148
Foundation Stabilization	154
Reinforced Earth Slab	155
Sinkhole Repair	157

Page

CONCLUSION	163
----------------------	-----

Summary	163
Conclusions	166
Recommendations for Further Study	168

LIST OF REFERENCES	170
------------------------------	-----

APPENDICES

Appendix A - Additional Ratings of Agricultural Soils of Lawrence County	181
Appendix B - Evaluation of Accessible Explorations and Drilling Methods	183
Appendix C - CSIR and NGI Classification Systems for Jointed Rock Masses	187
Appendix D - Suggested Special Provisions for Sinkhole Areas	194

LIST OF TABLES

Table	Page
1. Rating of Highway Soil Considerations for Engineering Soil Units of Lawrence County.	9
2. Associations Between Parent Material, Landform, Physiographic Subsection, and Agricultural Soils.	11
3. Characteristics of the Limestone Formations of the Mitchell Plain.	13
4. Known Caves of Lawrence County Corresponding to Figure 9.	34
5. Details of Methodology of Site Investigation in a Karst Region.	49
6. Airphoto Patterns of a Limestone Plain.	53
7. Geologic Indicators of Sinkhole Collapse in the Mitchell Plain of Southern Indiana.	60
8. Evaluation of Geophysical Techniques for Site Investigation in a Karst Region.	83
9. Suggested Gradation Characteristics of Drainage Stone. (From Royster, 1984).	111
10. Procedure for Evaluating Hydrologic Aspects of Problems Encountered in Karst Regions.	116
11. Allowable Contact Pressure on Carbonate Rock Based on RQD. (From Peck, et. al., 1974)	126
12. Allowable Contact Pressure on Carbonate Rock from Various Building Codes. (After D'Appolonia, et. al., 1975; and Peck, et. al., 1974)	126
13. Qualitative Stability Index of Karst Caverns. (From Jingmin and Zhizhong, 1982)	130
14. Approximate Strength Criteria for Intact and Jointed Carbonate Rock. (From Hoek and Brown, 1980a). . .	138

Appendix
Table

Page

A1. Frost Potential, Corrosivity Ratings and Spring Water Table Depths of Agricultural Soils of Associated with Landforms of Lawrence County. . . .	182
B1. Evaluation of Types of Accessible Explorations. . .	184
B2. Evaluation of Drilling Methods.	185
C1. CSIR Geomechanics Classification of Jointed Rock Masses.	188
C2. The Effect of Joint Strike and Dip Orientations in Tunnelling.	188
C3. Classification of Individual Parameters Using the NGI Tunnelling Quality Index.	190

LIST OF FIGURES

Figure	Page
1. Physiographic Map of Indiana. (After Wayne, 1956)	7
2. Idealized Profile in a Karst Region. (From Scholle, et. al., 1983)	20
3. Distribution of Karst Areas in Relation to Carbonate and Sulfate Rocks in the United States. (From Davies and Le Grande, 1972)	21
4. Karst Areas of Central Kentucky and Indiana. (From Davies and Le Grande, 1972)	23
5. Examples of Terra Rosa and Lapies. (From Wallace, 1982)	25
6. Schematic Cross-Sections of a Doline and Collapse Sink. (From Wallace, 1982)	26
7. Idealized Block Diagram Showing Karst Features of the Crawford Upland and the Mitchell Plain. (From Powell, 1961)	28
8. Models of Solution Channel Evolution. (After Powell, 1976)	32
9. Map of Known Caves in Lawrence County. (After Powell, 1961)	33
10. Process of Sinkhole Development. (From Wallace, 1982)	36
11. Sudden Sinkhole Collapse in Soil Overburden. (From Wallace, 1982)	39
12. Sudden Collapse of Bedrock Roof. (From Wallace, 1982)	40
13. Hypothetical Cost-Uncertainty Curve of a Site Investigation in a Karst Region.	46
14. Proposed Methodology of Site Investigation in a Karst Region.	48

Figure	Page
15. Sinkhole Density Map of Lawrence County.	62
16. Bedrock Geology Map of Lawrence County. (After Gray, et. al., 1970)	64
17. Lineament Pattern of Sinkholes in South-Central Lawrence County (T4N, R1W).	65
18. Character of Jointing in South-Central Lawrence County. (From Palmer, 1969)	67
19. Central Electrode Configuration. (After McDowell, 1975)	79
20. Decline of Water Table from Pumping of Well in a Carbonate Terrain.	97
21. Cross-Section Through Sinkhole Area Covered with Blanket Drain. (From Weber and Darnell, 1984) . . .	106
22. Schematic Cross-Section of the Tennessee S.R. 76 Subsidence Problem and Solution. (From Royster, 1984)	107
23. Schematic Cross-Section of Sump Design for the Tennessee S.R. 12 Sinkhole. (From Royster, 1984) . .	110
24. Simplified Cross-Section of a Sinkhole Used for Drainage. (From Strate, 1972)	112
25. Occurrence of High Permeability Zones for Well Location in Carbonate Rock. (After Freeze and Cherry, 1979)	114
26. Classification of Limestones, Limestone-Dolomite Mixtures, and Calcareous Sedimentary Rocks. (From Dearman, 1981)	119
27. A General Engineering Classification of Carbonate Rocks Based on Elastic Modulus and Unconfined Compressive Strength. (After Dearman, 1981)	120
28. Typical Weathering Profile and Classification Scheme for Carbonate Rocks. (From Dearman, 1981)	122
29. Foundation Failure Modes on Carbonate Rocks. (From Dearman, 1981; after Sowers, 1976)	124
30. Weathering Features Influencing the Stability of Road Cuts in Carbonate Rocks. (From Dearman, 1981) .	128

Figure	Page
31. Mohr-Coulomb Failure Envelopes for Failure (a) in Intact Rock and (b) along Discontinuity.	133
32. Shear Strength Envelope and Variation of Strength Parameters with Normal Stress for Hoek and Brown (1980a) Strength Criteria.	136
33. Alternative Methods of Founding on Carbonate Rock. (From Partridge, et. al., 1981)	142
34. Simplified Analysis of Arching Effect in Soil above a Developing Cavity in Limestone Bedrock.	146
35. Simplified Theory of Two-Dimensional Compacted Soil Arch. (After Partridge, et. al., 1981)	149
36. Reinforced Earth Slab Constructed over Cavities to Support a Highway Embankment. (From Steiner, 1975) .	156
37. Repair of Sinkhole Using Excavation - Concrete Plug Method. (From Weber and Darnell, 1984)	159

Appendix

Figure

D1. Typical Sinkhole Formations in Pennsylvania. (From Skea, et. al., 1963)	198
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LIST OF ABBREVIATIONS AND SYMBOLS

a	- distance between current sink and potential electrodes
A	- empirical rock strength constant
AASHTO	- American Association of State Highway and Transportation Officials
b	- distance between potential electrodes
B	- empirical rock strength constant; half span of compacted soil arch
c	- strength intercept
c'	- effective strength intercept
c _i	- instantaneous value of strength intercept
c ₁	- strength intercept along discontinuity
c ₁ , c ₂	- current sink electrodes
cm	- centimeter
°C	- degrees Centigrade
CaCO ₃	- calcite
Ca(HCO ₃) ₂	- calcium bicarbonate
CO ₂	- carbon dioxide
CSIR	- South African Council for Scientific and Industrial Research
d	- distance between current sink and potential electrodes

dc (DC)	- direct current
et. al.	- "and others"
E	- elastic modulus
f_r	- resisting forces
f_s	- sliding forces
FS	- factor of safety
ft	- feet
gpm	- gallons per minute
gwt	- groundwater table
GPa	- gigapascal
h	- depth to water table
h_w	- depth from water table to bedrock
H_a	- height of soil arch at abutments
H_c	- height of soil arch above crown
H^+	- hydrogen ion
HCO_3^-	- bicarbonate ion
H_2CO_3	- carbonic acid
H_2O	- water
i. e.	- "that is"
in.	- inch
K	- coefficient of lateral earth pressure
K_a	- coefficient of active earth pressure
m	- meter; empirical rock strength constant
mi	- mile
$MgCa(CO_3)_2$	- dolomite
$MgCO_3$	- magnesium carbonate
MPa	- megapascal

N	- normal force; north
No.	- number
N_a	- residual lateral stress due to compaction at abutments of soil arch
N_c	- lateral stress above crown of soil arch
NASA	- National Aeronautics and Space Administration
NGI	- Norwegian Geotechnical Institute
psf	- pounds per square foot
psi	- pounds per square inch
pvc	- poly-vinyl chloride
pH	- measure of acidity/alkalinity
P	- line impact of mechanical roller
P_1, P_2	- potential electrodes
P-wave	- primary or compression wave
q_a	- allowable contact pressure
q_u	- unconfined compressive strength
R	- radius of soil arch; measured resistivity
RQD	- rock quality designation
RlW	- land survey range (Range 1 West)
s	- empirical rock strength constant; soil shear strength in terms of effective stress
sq ft	- square feet
S	- shear force
SLAR	- side-looking airborne radar
SP	- self potential
S.R.	- State Route
tsf	- tons per square foot

T_v	- vertical shear force
T5N	- land survey township (Township 5 North)
USCS	- Unified Soil Classification System
USGS	- United States Geological Survey
w_{opt}	- optimum moisture content
W	- weight
z	- depth; lift thickness
α	- angle of failure plane with horizontal
β	- angle of discontinuity with horizontal
γ	- unit weight of soil
γ_m	- moist unit weight of soil
γ_w	- unit weight of water
θ	- angle of inclination
ρ_a	- apparent resistivity
μm	- micrometer
σ_c	- uniaxial compressive strength of rock
σ_h	- horizontal effective stress
σ_n	- normalized applied stress (σ/σ_c)
σ_t	- tensile strength of rock
σ_{tn}	- normalized tensile strength of rock (σ_t/σ_c)
σ_v	- vertical stress
σ_1	- major principal stress
σ_{1n}	- normalized major principal stress (σ_1/σ_c)
σ_3	- minor principal stress
σ_{3n}	- normalized minor principal stress (σ_3/σ_c)
τ	- shear stress
τ_m	- maximum shear stress

τ_n	- normalized shear stress (τ/σ_c)
ϕ	- angle of internal friction
ϕ'	- effective angle of internal friction
ϕ_i	- instantaneous angle of internal friction
ϕ_l	- angle of friction along discontinuity
$>$	- greater than
$<$	- less than

HIGHLIGHT SUMMARY

The weathering of soluble carbonate rock produces a highly irregular landscape and subsurface profile referred to as karst topography. Diversion of surface water to underground solution-enlarged passages in the bedrock erodes the overlying soil leading to ground subsidence and eventual collapse, which is often catastrophic. The geologic literature describing karst geomorphology is extensive, and the processes responsible for the geotechnical problems associated with karst features are fairly well understood. Despite this, site evaluation, engineering design, and methods of construction which relate to these problems are not very well developed. An extensive review of the literature has been conducted to provide a concise statement of the geologic, hydrologic, climatological, and man-made factors responsible for the development of karst features and their related engineering problems.

Lawrence County, located in the karst region of southern Indiana known as the Mitchell Plain physiographic subsection, is chosen as the study area. The geotechnical engineering problems of the area are briefly described and

are associated with engineering soil units classified according to landform and parent material. Since karst-related problems dominate, this research focuses on their evaluation and solution.

A careful study of site investigation techniques specifically applicable to karst regions is performed to develop an efficient methodology of site evaluation. Such a methodology is proposed and includes preliminary studies, reconnaissance surveys, geophysical techniques, and boring location schemes. New concepts are introduced, such as preparation of a regional sinkhole density map. Others are evaluated, such as the identification of lineaments from aerial photography as an aid for site evaluation.

Several remedial and preventive techniques dealing with hydrologic aspects of sinkholes are reviewed. Factors relating to engineering design of foundations and embankments in karst regions are discussed. Failure modes and strength criteria for carbonate rock are described. A relatively new technique for analyzing a compacted clay fill is developed. Although many simplifying assumptions are used, the method has great potential as an efficient and economic alternative to costly treatment of individual features. Finally, a procedure for sinkhole repair is recommended.

INTRODUCTION

Areas underlain by carbonate rock such as limestone and dolomite have unique characteristics which present difficult problems for geotechnical engineers. A "karst" landscape is produced by chemical and mechanical alteration of soluble carbonate rocks. Specific geologic, hydrologic, and climatological circumstances are necessary for geomorphic processes to fully develop solution features. The Mitchell Plain of southern Indiana satisfies these requirements, and karst features such as subsurface cavities and sinkholes are common.

Engineering Problems in Karst Regions

Three major engineering problems are encountered in karst areas. The first is the development of an irregular soil-bedrock contact. Carbonate rocks are susceptible to solutioning by mildly acidic groundwater, and activity is concentrated along structural defects such as joints, fault zones, and bedding planes. Weathering is therefore enhanced in these weaker zones, and a rough bedrock surface results, with many rock pinnacles separating deep, soil-filled

cavities. The bedrock surface is difficult to contour, and problems arise when planning transportation routes and their consequent cuts, fills, and deep foundation elements.

The second problem is the formation of solution features such as caves and sinkholes. Solutioning of carbonate bedrock results in migration of soil particles from the overlying residuum into the rock cavities. Voids form in the overburden, and a soil arch results. Changes in the groundwater regime alter the stresses acting within the soil. Underground erosion occurs as soil is carried away by flowing groundwater. These changes may be either natural or man-induced. General subsidence and collapse of the ground surface over these cavities results, endangering structures in the vicinity. Localized flooding is a secondary problem which results when surface drainage characteristics are altered by construction and land development.

The third problem is the weathering of the carbonate rock to a highly plastic residual soil. Relict joints and bedding planes, and an open soil structure provide good internal drainage in the natural residuum developed from limestone and dolomite. However, reworking the soil alters structural characteristics and destroys the natural open internal fabric. The resulting material is difficult to compact and often provides poor foundation and subgrade support.

Objectives

Extensive information about karst geomorphology and the resulting landforms is available in the literature. The geologic and hydrologic processes responsible for development of the unique features of karst regions are fairly well understood. However, application of this knowledge to engineering problems encountered in these areas is not very advanced. Consequently, reports in the engineering literature addressing these problems have been scarce until quite recently. Construction techniques are still somewhat of an art, and designs are based on qualitative, rather than quantitative factors.

The objective of this research is to develop a state-of-the-art summary of techniques of site investigation, construction, and remedial treatment applied to karst regions. Lawrence County, located in the Mitchell Plain of southern Indiana, is chosen as the study area. A detailed description of the area is found in a previous Joint Highway Research Report, No. 84-7 (Adams, 1984). Factors directly affecting the geomorphic processes responsible for the karst landscape can be assessed from this information. The county is mapped according to landform-parent material associations, and all major engineering problems with respect to transportation route selection, design, and construction are described.

Since most of the county is underlain by Mississippian Age limestone, the primary engineering problems encountered are related to the developed karst topography. Review of pertinent geologic literature is performed to provide a detailed discussion of the origin of these problems. The engineer must have a thorough understanding of the processes involved so that appropriate techniques for site investigation and construction are chosen. A methodology of site evaluation in karst regions is proposed based on the most up-to-date techniques available. Preliminary studies, remote sensing surveys, geophysical techniques, and the development of boring location schemes are discussed in detail. Hydrologic aspects of land subsidence, collapse, and flooding are considered, and several examples of preventive or remedial treatment based on these factors are included. Finally, state-of-the-art construction techniques dealing with sinkhole repair and embankment construction are described. Some simple quantitative methods of estimating soil stress and deformation characteristics in carbonate terrain are developed. Recommended procedures are outlined, and limitations of the various methods are discussed.

This study concerns the karst region of southern Indiana, specifically the Mitchell Plain physiographic subsection. Concepts and methods are applicable to other areas with similar geologic, hydrologic, and climatological characteristics, such as the karst regions of Kentucky,

Tennessee, Pennsylvania, and to a lesser extent, Missouri. Karst problems in Florida, Georgia, and Alabama have a slightly different origin, and this must be kept in mind when attempting to transfer the technology discussed in the following sections to these areas. However, a review of studies conducted in the latter geographical region is included, and some of the techniques can be applied there, if their limitations as well as differences in the geomorphology of the area are considered.

The following report presents a detailed review of the current understanding of karst geomorphology and the state-of-the-art investigative and construction techniques as described in the literature. The concepts and methods were not physically explored or tested during this research. However, their application to the karst region of southern Indiana should be successful if all of the factors described below are considered and proper engineering judgment is exercised.

ASSESSMENT OF ENGINEERING PROBLEMS OF LAWRENCE COUNTY

Lawrence County is shown on the physiographic map of Indiana, which is illustrated in Figure 1. The Mitchell Plain covers most of the county, and exhibits a karst landscape. Some karst features are also found in the Crawford Upland, where limestone outcrops between shale and sandstone units. These two physiographic subsections are part of the Highland Rim Section of the Interior Low Plateaus Province (Witzcak, 1970). A variety of engineering problems are encountered in Lawrence county because the diverse geology of the region. A rating of these problems for each engineering soil unit indentified on the engineering soils map of the county, found in Adams (1984), is described in the following section.

Rating of Engineering Problems

To thoroughly assess potential hazards on a regional basis, a preliminary rating system is recommended. Sisiliano (1970) introduced the concept of rating the potential seriousness of highway engineering problems for a particular parent material within a physiographic region. This idea has been adapted to the engineering soil units identified in

Figure 1. Physiographic Map of Indiana.
(After Wayne, 1956)

the county. The original recommendations from Table 14 of Sisiliano (1970) are incorporated into Table 1, prepared specifically for Lawrence County. Some modifications have been made to take into account special problems encountered in the study area. In addition, some new considerations have been added under the heading "Miscellaneous" including: shale fills, availability of aggregate, subsurface irregularity, corrosion of steel and concrete, and the presence of a high water table in the spring.

The intention of Table 1 is to provide a preliminary guide as to which potential problems the engineer should expect in a specific soil unit. The designations L, M, and H indicate that there is a low, average, or high likelihood of a particular problem developing, respectively. A high probability means that the problem deserves detailed consideration during planning, design, and construction of a transportation route. Of course, this concept is applicable to other engineering facilities as well. The rating is based on the engineering judgment of Sisiliano (1970) and this author, taking into account experience as well as available laboratory and field data.

This rating system is meant to accompany the engineering soils report of Lawrence County (Adams, 1984). Information from the "Soil Survey of Lawrence County, Indiana" (1984) was also used to compile parts of Table 1. The Tables in the Appendix of Adams (1984) give geotechnical

engineering data of the agricultural soils of the county. Table 2 presents associations between the engineering soil units, physiographic subsections, and agricultural soils. Since each agricultural soil is correlated with a particular engineering soil unit, the data can be directly applied to the soils identified on the engineering soils map of Lawrence County. Table A1 of Appendix A provides a basis for the rating of frost potential and corrosivity as compiled from Thomas (1981).

Description of the Mitchell Plain

More than half of the county is covered by limestone bedrock and the residual soil developed from it. To set the stage for a discussion of the karst region of southern Indiana, the geology of the Mitchell Plain physiographic subsection is briefly described below.

The Mitchell Plain is composed of a series of limestone units of Mississippian Age. The oldest is the Sanders Group containing the Harrodsburg and Salem Formations. These limestones are coarse-grained, and do not have extensive karst features along their outcrop, as localized solutioning is not intense. The Harrodsburg Limestone is rather impure, containing large quantities of chert. The Salem Limestone is massive and relatively free of joints, and is quarried extensively as a building stone. It is less resistant to degradation than the underlying Harrodsburg, but is not

Table 2. Associations Between Parent Material, Landform, Physiographic Subsection, and Agricultural Soils.

ORIGINAL	PARENT MATERIAL	LANDFORM	PHYSIOGRAPHIC SUBSECTION	PREDOMINANT SOIL TYPE	ASSOCIATED AGRICULTURAL SOILS	SLOPES	TOPOGRAPHIC EXPRESSION
FLUVIAL	ALLUVIUM	FLOOD PLAIN ALONG EAST FORK WHITE R.	NORMAN UPLAND MITCHELL PLAIN CRAWFORD UPLAND	SILT, CLAY, SAND	NOLIN NEWMARK PETROLIA CHAGRIN	0-3%	FLAT PLAIN
		FLOOD PLAIN ALONG TRIBUTARIES OF EAST FORK WHITE R.	NORMAN UPLAND MITCHELL PLAIN CRAWFORD UPLAND	SILT, CLAY, SAND	HAYMOND STENDAL WILBUR BURNSIDE	0-4%	FLAT PLAIN
	ALLUVIAL AND EOLIAN DRIFT	SANDY TERRACES ALONG EAST FORK WHITE RIVER	NORMAN UPLAND MITCHELL PLAIN CRAWFORD UPLAND	SAND, SILT	ALVIN ABSCOTA BLOOMFIELD TYNER	0-50%	LOW TERRACE SOME LOW HILLS
		BEDROCK-DEFENDED TERRACES	NORMAN UPLAND MITCHELL PLAIN CRAWFORD UPLAND	SILT, CLAY, SAND	ELKINSVILLE VARIET PEKIN BURNSIDE	0-25%	LOW TERRACE SOME DISSECTION
EOLIAN	EOLIAN DRIFT	SAND/LOESS OVER LIMESTONE OR SANDSTONE-SHALE	NORMAN UPLAND MITCHELL PLAIN CRAWFORD UPLAND	SAND, SILT	PRINCETON MARTINSVILLE	0-50%	ROLLING UPLAND SMALL DUNES
LACUSTRINE	LACUSTRINE DEPOSITS	LAKEBED AND SLACKWATER PLAIN	NORMAN UPLAND MITCHELL PLAIN CRAWFORD UPLAND	SILT, CLAY	BARTLE HENSHAW	0-3%	FLAT TO UNDULATING PLAIN
		LACUSTRINE TERRACE	NORMAN UPLAND MITCHELL PLAIN CRAWFORD UPLAND	SILT, CLAY	MARKLAND McGARY	0-70%	LOW TERRACE HIGHLY DISSECTED
	INTERBEDDED SANDSTONE-SHALE BEDROCK	SANDSTONE-SHALE UPLANDS	NORMAN UPLAND CRAWFORD UPLAND	SILT, CLAY	EBAL GILPIN HOSMER WELLSTON	0-70%	PLATEAU UPLAND HIGHLY DISSECTED
		SANDSTONE-SHALE SIDESLOPES	NORMAN UPLAND CRAWFORD UPLAND	SILT, CLAY	BERKS GILPIN WEIKERT	0-80%	STEEP SIDESLOPE
RESIDUAL	INTERBEDDED SANDSTONE-SHALE LIMESTONE	SANDSTONE-SHALE OVER LIMESTONE	CRAWFORD UPLAND	SILT, CLAY	EBAL, GILPIN HOSMER WELLSTON CANEYVILLE	0-80%	FLAT UPLAND TO STEEP SIDESLOPE
	LIMESTONE BEDROCK	LIMESTONE UPLANDS	MITCHELL PLAIN	SILTY CLAY, CLAY	BEDFORD, CRIDER FREDERICK HOSMERVILLE MUREN	0-60%	ROLLING PLAIN NUMEROUS SINKHOLES
		LIMESTONE SIDESLOPES	MITCHELL PLAIN	SILTY CLAY, CLAY	CANEYVILLE FREDERICK	2-75%	STEEP SIDESLOPE

easily distinguishable from it where exposed at the surface (Malott, 1922).

Finer-grained carbonate rock of the Blue River Group includes the St. Louis, Ste. Genevieve, and Paoli Limestones. They are characteristically thin-bedded and highly jointed, resulting in the development of classic karst topography along the outcrop. There are occasional thin layers of shale and impure limestone horizons. Chert is very conspicuous in the upper portion of the St. Louis Limestone. The Ste. Genevieve and Paoli Limestones are oolitic in nature, being composed of small sand-sized spheres of calcium carbonate called oolites. Karst development is most pronounced in the St. Louis and Ste. Genevieve Formations, and their outcrops form a pock-marked plain with many sinkholes and subterranean passages. The upper members of the Blue River Group also exhibit numerous sinkholes, and form low hills in the western third of the Mitchell Plain (Malott, 1922). Brief descriptions of the lithology, karst expression, and collapse potential of the five limestone formations which outcrop in the Mitchell Plain are presented in Table 3.

Engineering problems encountered in karst areas are fairly well understood. The following section describes in detail the geologic processes responsible for the unique character of the soil and bedrock of limestone regions. With this background, techniques of site investigation and

Table 3. Characteristics of the Limestone Formations of the Mitchell Plain.

FORMATION	CHARACTERISTICS	KARST EXPRESSION	COLLAPSE POTENTIAL
Paoli Limestone	Tan fine- to medium-grained dense thin-bedded LS, locally oolitic; lower portion crystalline; some shale and chert. Thickness: 0-40'	Sinkholes common where lower parts of formation outcrop; hilly topography; solutioning mod. active. ² Sinkhole Density: 0-100/mi ²	Moderate to High Sinkholes are common where the formation outcrops in karst valleys.
Ste. Genevieve Limestone	Lt. tan fine-grained dense thin-bedded crystalline and oolitic LS; beds of shale, calcareous SS, and dolomite common. Thickness: 70-170'	High density of sinkholes; numerous caves; subsurface drainage extensive; classic karst topography. Sinkhole Density: 0-1000/mi ²	High Sinkholes are common throughout the plain.
St. Louis Limestone	Upper: lt. tan fine- to medium-grained thin-bedded LS; lower: brown fine-grained dense LS; beds of gypsum and anhydrite. Thickness: 90-300'	High density of sinkholes; numerous caves; subsurface drainage extensive; classic karst topography. Sinkhole Density: 0-200/mi ²	Moderate to High Sinkholes are common throughout the plain.
Salem Limestone	Gray to tan medium- to coarse grained fossiliferous LS; massive, unbedded, with few joints. (Bedford Limestone) Thickness: 20-80'	Rolling topography; surface drainage dominates; springs are common, but small; limited karst expression. ² Sinkhole Density: 0-50/mi ²	Low to Moderate Sinkholes conspicuously absent in most areas.
Harrodsburg Limestone	Lt. tan to gray medium- to coarse-grained LS; hard, crystalline, thin to massive, unevenly bedded; cherty. Average thickness: 70'	Rolling topography; surface drainage dominates; springs are common, but small; limited karst expression. ² Sinkhole Density: 0-100/mi ²	Low to Moderate Sinkholes confined to interstream spaces.

Description of characteristics and karst expression compiled from Powell (1961); and Pinsak (1957).

design and construction methods in karst areas are then presented.

ORIGIN OF ENGINEERING PROBLEMS IN KARST REGIONS

Geotechnical engineers are faced with unique challenges when building structures or locating transportation routes in karst regions. Three major problems encountered are (a) a highly irregular soil-bedrock contact; (b) development of solution features such as caverns and sinkholes; and (c) weathering of the rock to a highly plastic residual soil. The geologic processes which govern the development of the distinct topography of karst regions must be understood if the engineer is to have insight into the problems that are faced when building in such areas. Specific engineering properties and structural characteristics of the rock, such as orientation of bedding planes, joint spacing, presence of fault zones, folds, and other types of rock, coupled with climate and land-use, influence the weathering processes and the subsequent problems that develop. This chapter describes the basic chemical and mechanical weathering processes in limestone and dolomite and discusses the prerequisites for the development of landforms of karst regions.

Definition of Karst Topography

The name karst comes from a narrow strip of limestone plateau in Yugoslavia and Italy bordering on the Adriatic Sea (Thornbury, 1954). Many features unique only to limestone, and to a lesser extent dolomite, are found there, and other regions of the world having similar characteristics are given the name karst topography. Such a landscape exhibits an irregular topography characterized by streamless valleys, sinkholes, and streams that disappear underground, all developed by the action of groundwater (Leet, et. al., 1978). Depressions are numerous, surface drainage is erratic or absent, and cavern development is extensive. Many other features are often present, and these are described later.

Chemical Weathering Process in Limestone and Dolomite

Limestone is the general name for a sedimentary rock composed primarily of the mineral calcite, CaCO_3 . It has several different forms depending upon the texture, and is formed by both organic and inorganic processes. Crystalline formations are found in nature, but granular-textured limestones whose particles range in size from silt to shell fragments up to an inch across are more common. Many inorganic processes are responsible for limestone deposition. A familiar form is travertine, or dripstone, which is formed in caves from the evaporation of mineral-laden water. Tufa

is a spongy rock formed from precipitation of calcite from spring water. Another inorganic process is precipitation of calcium carbonate from seawater to form small sand-sized particles of almost pure calcite called oolites. Such a deposit is termed oolitic limestone, examples of which are found in portions of the Paoli and Ste. Genevieve Limestones (see Table 3).

The organic processes which form limestone usually occur in a marine environment. Accumulation of organic sediments from shells and other undecomposed debris occurs over a long period of time, and eventually the material becomes cemented to form a rock. This limestone consists of sand-sized particles, but other forms occur. Chalk is a soft, fine-grained, fossiliferous form of calcium carbonate. Coquina is a coarse-grained, fossiliferous limestone composed of shell fragments and other organic debris (Leet, et. al., 1978).

Dolomite is another carbonate rock, and is composed of a large portion of the mineral dolomite, $\text{MgCa}(\text{CO}_3)_2$. Pure dolomitic stone contains a ratio of 40 to 44 % MgCO_3 and 54 to 58 % CaCO_3 . However, it is common to refer to any stone with more than about 20 % MgCO_3 as dolomite (Boynton, 1966). Substitution of magnesium for calcium in calcite deposits is believed to be the primary process in the development of dolomite. Metamorphism, resulting from the intrusion of magnesium-rich igneous materials into limestone, is another

theory of transformation. The majority of dolomite formations are formed by an increase in the amount of magnesium in the groundwater percolating through a limestone formation (Leet, et. al., 1978). The resulting rock is less soluble than the original limestone, but karst development may still be extensive, as evidenced by the dolomites of the Far West Rand and southwestern Transvaal regions of South Africa (Kleywegt, 1980; and Wagener, 1982).

Limestone and dolomite are among the most chemically stable earth materials. Pure water flowing through them does not seriously alter the rock. Decomposition is also minimal for temperatures between absolute zero and 600°C. However, most groundwater contains minor amounts of dissolved CO_2 , which reacts with H_2O to form carbonic acid, H_2CO_3 . The acid reacts to dissolve calcite and forms calcium bicarbonate, $\text{Ca}(\text{HCO}_3)_2$, a highly unstable compound. The reactions are expressed in equation form:

Similar reactions occur for magnesium carbonate in dolomite. Calcium bicarbonate is about thirty times more soluble than the original calcite, and is carried away by water flowing through the formation. Other acids and acid gases react with calcite, usually in an exothermic reaction which liberates CO_2 gas (Boynton, 1966).

Although this weathering process is slow, extensive cavities develop if there is a steady flow of groundwater available. Solutioning is intense along joints, bedding planes, and fault zones (Howard, 1963). Loss of support of soil and rock above the void results in further enlargement. Figure 2 shows an idealized karst profile with the different groundwater zones illustrated.

Prerequisites for Karst Development

There are many significant limestone and dolomite regions of the world. Davies and Le Grand (1972) estimate that about 15 % of the continental United States has limestone or other soluble rocks at or near the surface (Figure 3). However, karst features do not develop unless certain physical conditions are present. Thornbury (1954) cites four prerequisites for the development of karst topography: (a) the presence of a soluble rock at or near the surface; (b) a dense, highly fractured, thinly bedded rock; (c) entrenched major valleys within the upland; and (d) at least a moderate amount of rainfall. Limestone, dolomite, gypsum, and salt are some of the earth materials that are subject to the solutioning process. A highly jointed and thinly bedded rock permits concentrated flow and accelerates solutioning. The large hydraulic gradient provided by deeply entrenched valleys allows water to flow through the rock before it becomes saturated with bicarbonate. Figure 3 reveals that karst development occurs on only a limited number of the

Figure 2. Idealized Profile in a Karst Region.
(From Scholle, et. al., 1983)

Figure 3. Distribution of Karst Areas, in Relation to Carbonate and Sulfate Rocks in the United States. (From Davies and Le Grand, 1972)

areas susceptible to solution, since these physical conditions are not present everywhere.

The Mitchell Plain satisfies these requirements since (a) limestones of the Sanders and Blue River Groups outcrop extensively; (b) the St. Louis and Ste. Genevieve Limestones are dense, highly jointed, and thinly bedded; (c) the East Fork White River, Salt Creek and other streams have cut major valleys in the plain; and (d) the area receives an average of 45 in. of rainfall a year. The karst regions of southern Indiana and central Kentucky are shown in Figure 4.

Karst Landforms

The presence of karst features must be anticipated by the engineer when planning a site investigation. The significance of each on the design of foundations and transportation routes should be understood. Descriptions of common karst landforms are given as summarized from Thornbury (1954).

The mantle of red residual soil from the weathering of limestone and dolomite rock is called terra rosa. It extends down into solution zones in the rock. Soil thickness is highly variable, but is generally thickest in the flat uplands and valley bottoms, and thinnest on the sideslopes. Exposed limestone or dolomite bedrock which is etched, grooved, or fluted is termed lapies. Geologists argue over the origin of this landform, but for engineering

Figure 4. Karst Areas of Central Kentucky and Indiana.
(From Davies and Le Grand, 1972)

purposes the term can be applied to any rugged outcrop of bedrock devoid of terra rosa. Profiles illustrating terra rosa and lapies are presented in Figure 5.

Saucer-shaped collapse depressions which develop in the residual soil are called sinkholes. Dimensions vary from several feet to more than 100 ft deep, and several feet to over 400 ft across. Sinkholes vary in shape and relative dimensions. In southern Indiana and Kentucky they are broad, funnel-shaped, shallow depressions; while in Missouri they are generally cylindrical with steep sides up to 60 ft deep and 25 ft in diameter (Williams and Vineyard, 1976). Two types of sinkholes are distinguished by geologists. Dolines are sinks which develop downward by solution beneath the residual soil cover. Collapse sinks result from the collapse of rock and soil above an underground cavity. They are steep-sided and may be partially filled with water. Dolines are much more common than collapse sinks, and are focal points for surface drainage, often having openings at the bottom called swallow holes or swallets. Karst lakes or sinkhole ponds may result if the opening becomes clogged with clay or other debris. Profiles of a doline and a collapse sink are shown in Figure 6.

Features resulting from the collapse of extensive roof sections over an underground watercourse are termed uvalas. They appear as small stream valleys on the limestone plain, but end abruptly at the point where no further collapse has

(a) Typical terra rosa profile.

(b) Lapies surface.

Figure 5. Examples of Terra Rosa and Lapies.
(From Wallace, 1982)

(a) Doline.

(b) Collapse sink.

Figure 6. Schematic Cross-Sections of a Doline and Collapse Sink. (From Wallace, 1982)

occurred. Small streams which develop on a karst plain and terminate at a sinkhole are called sinking creeks. One that has occupied the same location for a long period of time produces a deep valley upstream which terminates at a swallow hole, and is referred to as a blind valley. Stream valleys which are entrenched in limestone but surrounded by non-carbonate rocks are called karst valleys. Surface drainage is evident on the uplands but the valley floor exhibits sinkholes and other karst features. An idealized block diagram of a portion of the Mitchell Plain and Crawford Upland illustrating these features is shown in Figure 7.

Sections of a surface stream which have been diverted underground are called subterranean cutoffs. Such a stream disappears into a swallow hole and reappears downstream in the form of a spring. Often, a dry bed having flow only during periods of heavy rainfall is associated with it. Indian Creek in Lawrence County, the Lost River in Orange County, and Bogard Creek in Crawford County are examples of a subterranean cutoff (Malott, 1922).

Development of Subsurface Cavities

The chemical weathering of carbonate rock takes hundreds of years to develop a void of any significance. The process is too slow to observe easily, therefore geochemical and geomorphic models are used to estimate rates of solu-

Figure 7. Idealized Block Diagram Showing Karst Features of the Crawford Upland and the Mitchell Plain. (From Powell, 1961)

tioning. Several studies of limestone solutioning have been conducted in a wide range of geologic settings and climate. Tests performed in Puerto Rico estimated that limestone may dissolve at rates as high as 1.7 cm thickness per 100 years under the right conditions (United States Atomic Energy Commission, 1974) . Another study in Kentucky yielded solution rates of about 1.5 cm per 100 years (Legget and Karrow, 1983). However, Jennings (1983) reports a hydrogeologic study in Australia giving much lower values. In general, limestones are dissolved at rates on the order of 1 cm (0.4 in.) per 100 years. The actual numbers are usually not important, except to illustrate the magnitude of time required for cavern development.

Solution channels form as groundwater flow dissolves the bedrock along joints, bedding planes, and fault zones. Many theories have been proposed explaining the morphology of limestone caverns. The prevailing opinion prior to 1930 centered around seven key points: (a) caverns are formed above the water table through the action of diverted surface water; (b) water may circulate below the water table, but is not significant in the solution process; (c) the base level of local streams controls the downward movement of voids; (d) joints are the primary features along which caverns develop; (e) mechanical erosion may be significant; (f) one level may be abandoned as the stream valleys cut vertically downward; and (g) after a cavern is abandoned, travertine

development becomes the dominant process (Thornbury, 1954). Davis (1930) challenged many of these ideas with his two-cycle theory of cavern development. In summary, his theory rejects the notion that little solutioning takes place below the groundwater table. He believes active solutioning occurs within the phreatic zone, and subsequent lowering of the water table initiates the second phase, which is the development of cave travertine.

At the present time, there is still disagreement on all of the factors responsible for cave development. Ford (1971) reviewed some of the theories on the genesis of limestone caverns. He states that each theory is plausible if the right conditions are present, but none of them is sufficient to explain the origin of caverns in general. Ford refers to caves developed according to the pre-1930 notions as vadose caves. Caverns developed according to Davis' theory are termed deep phreatic caves. Much of the speculation of the past few decades has centered around the "water table cave" theory. It suggests that cavern formation occurs within 200 ft of the water table, while Davis' explanation allows for the random development of cavities at greater depths. From an engineering standpoint, much of the debate is academic, since the developed morphology is the same.

Powell (1976), in a study of the limestone formations of southern Indiana, postulates several models for the

evolution of underground solution channels. He considers lithologic, structural, and hydraulic factors which influence geomorphic development in the Mitchell Plain and Crawford Upland. Several sequences of solution channel formation under varying geologic conditions are illustrated by the block diagrams in Figure 8. They show a simplified progression from initial groundwater conditions, through an intermediate phase, and into a "final" form typically observed in the region. The models are not exclusive and are not intended to represent cyclical development or an equilibrium state in nature. They only show how idealized groundwater flow conditions may influence cavity development along joints and discontinuities in simple stratigraphic situations.

Many caves have been mapped in Lawrence County. They are especially prevalent in the St. Louis and Ste. Genevieve Limestones. Figure 9 shows the major cave systems that have been mapped in Lawrence County. Table 4 lists the caverns located in Figure 9. Several are mapped and described in detail in Powell (1961). Such information is useful for conducting preliminary studies for site investigation in a limestone region.

Figure 8. Models of Solution Channel Evolution. (After Powell, 1976)

Scale 1 : 250,000

LEGEND

- 120 Numbers correspond to caves listed in Table 4.
- Rocks of Pennsylvanian Age
Mostly shale and sandstone, includes some thin limestone, coal beds, and clay.
- Rocks of middle and late Mississippian (Meramec and Chester) Age
Mostly limestone in lower part, shale and sandstone in upper part.
- Early Mississippian and older rocks
Mostly shale and siltstone.

Figure 9. Map of Known Caves in Lawrence County.
(After Powell, 1961)

Table 4. Known Caves of Lawrence County
Corresponding to Figure 9.

121. Armstrong Cave	158. Kern's Pit
122. Bailey Cave	159. Lackey Cave
123. Baker's Cave	160. Linden Pit
124. Bantley's Cave	161. Lynn's Cave
125. Bear Cave	162. Mitchell Crushed Stone Co. Cave
126. Beaver Creek Swallow Hole Cave*	
127. Bedford Spring Cave	163. Jim Moore Cave
128. Bex Pit	164. Moreland Pit Cave
129. Blue Spring Cave*	165. Pinnacle Pit
130. Joe Bone Cave	166. Pless Cave*
131. Bootlegger's Pit Cave	167. Popcorn Spring Cave
132. Boulton Sink Cave*	168. Rainey Cave
133. Brazzell Cave*	169. Ramsey Cave*
134. Bronson's Cave*	170. Ray Spring Cave*
135. Buzzard's Cave	171. Reeder Cave
136. Colglazier Sink Cave*	172. Rock Lick Cave*
137. Connerly's Cave*	173. Salt Creek Cave
138. Crying Cave	174. Salt's Cave*
139. Denney Cave	175. Seven-Up Pit
140. Dixon's Pit Cave*	176. Sherman Pit
141. Dog Hill Cave*	177. Shiloh Cave*
142. Donaldson's Cave*	178. Silverville Pit Cave*
143. Donnehue's Cave*	179. Speed Hollow Cave
144. Dry Cave*	180. Storm's Pit
145. 18th Street Cave*	181. Sullivan Hill Pit
146. Evans Cave	182. Sullivan's Cave*
147. Fishing Creek Cave	183. Sweet Potatoe Cave
148. Formation Cave	184. Todd Cave
149. Fox Den Cave	185. Towe's Cave
150. Gory Hole*	186. Trailside Cave
151. Grinstaff's Cave*	187. Twin Caves*
152. Gyger Bend I Cave*	188. Valley Cathedral Cave*
153. Gyger Bend II Cave	189. Wagoner Cave*
154. Hamer's Cave*	190. Whistling Cave
155. Harrison Cave	191. Wilson Grotto
156. Hornecker Cave*	192. Wind Cave
157. Indian Pipe Pit Cave	193. Wolf Cave
	194. Wood's Ferry Spring Cave

*Described in Powell (1961)

Evolution of Sinkholes

Two types of mechanisms are responsible for the development of sinkholes. The first, and by far the most common, is an upward ravelling of soil over a cavity in the bedrock (Sowers, 1978). Figure 10 shows the development of a soil arch over an opening in the rock, which may be a small cave, an enlarged joint or fracture, or a narrow vertical pipe. Cylindrical solution pipes are often responsible for the sinkholes common to central Florida (Fountain, 1976). The appearance of a sinkhole at the surface is really the final step of a long process of chemical solution of the bedrock and mechanical erosion of the overlying soil (Howard, 1963; Allen, 1969; and Sowers, 1978).

The position and variability of the water table is a key aspect in sinkhole development since it affects the stresses acting on the soil and rock. The stress imposed by the weight of the soil above the void is resisted by the shear strength of the soil in the vicinity of the arch, and is transferred to the bedrock. When the water table is located above the cavity, the buoyant, or effective unit weight of the soil is acting on the arch. If the water table suddenly drops, the slight decrease in unit weight of the soil due to water loss is more than negated by the large increase in effective stress due to the removal of the pore water pressure. The shear strength of the soil is exceeded by the stress imposed on the soil arch, resulting in

(a) Solution cavities exist, no downward movement of soil.

(b) Soil arch develops, soil migrates into cavity.

(c) Arch increases in size, subsidence evident at surface.

(d) Overburden migrates downward, dish-shaped depression at surface.

Figure 10. Process of Sinkhole Development.
(From Wallace, 1982)

failure; i.e. soil begins to spall into the void until the stresses are redistributed such that the arch can again support the overburden.

For a case in which the water table is located within or below the void, the free water surface rises during periods of heavy rainfall. The shear strength of the soil is weakened as it becomes saturated, but the decrease in effective stress imposed by the soil overburden usually compensates for this. However, when the water level drops again, the effective stress increases, and soil ravelling occurs as the stresses are redistributed. A depression at the surface in the form of a doline results. Eventually, the size of the arch increases until the arched zone nears the surface. Once again, a redistribution of stresses occurs. If equilibrium cannot be maintained, the soil arch collapses and a sinkhole forms at the surface.

The catalysts of sinkhole collapse are both natural and man-made. Seasonal and long-term fluctuations in the groundwater table are a natural occurrence. Sudden surges of water occur during periods of heavy rainfall. These fluctuations are responsible for the development of natural sinkholes. Recently, however, human activities are increasingly more of a factor in the development of sinkholes, especially collapse sinks. In Alabama, for example, an estimated 4000 induced sinkholes and related features have developed since 1900, while only 50 or so natural sinkholes

have been reported during the same period (Newton, 1976). Lowering of the groundwater table due to pumping is a primary factor in sinkhole development in many regions. The large collapse in Winter Park, Florida in 1981 is attributed to excessive groundwater withdrawal (Jamaal and Associates, 1981). In Hershey, Pennsylvania, the development of over 100 sinkholes during a period of two months in 1949 was directly linked to pumping of the groundwater at a nearby quarry (Foose, 1953).

The second mechanism responsible for sinkhole development is collapse of the soil and rock roof above a cavity. Such a phenomenon is rare, however (Howard, 1963). A typical failure mode is the collapse of a cone or trapezoidal wedge of soil and rock into a cavern below. Since the sides of the collapse are often unstable, erosion continues until a stable feature is established, usually in the form of an inverted cone (Sowers, 1978). The collapse is usually catastrophic, and this should be considered whenever a new load is applied over an area with solution-enlarged openings in the bedrock. In addition to the new load applying an increased stress to the soil mantle and its supporting bedrock foundation, there is evidence that this increased stress accelerates the solution process in carbonate rocks, analogous to stress cohesion in metals (Sowers, 1978). The sequence of events during collapse of the soil overburden and a rock arch are illustrated in Figures 11 and 12.

(a) Soil arch exists, similar to doline.

(b) Sudden collapse of overburden.
Dashed lines indicate areas of
additional sloughing of soil.

Figure 11 . Sudden Sinkhole Collapse in Soil Overburden.
(From Wallace, 1982)

(a) Large cavern exists
beneath Limestone roof.

(b) Failure of roof causes sudden
collapse of overburden.

Figure 12. Sudden Collapse of Bedrock Roof.
(From Wallace, 1982)

Classification of Sinkholes

A classification system for sinkholes is useful so that factors responsible for their development can be studied on a more scientific basis. It is important for the engineer to assess the potential causes of collapse in a given area. Perlow, et. al. (1984), using experience from a study of over 2000 sinkholes near Allentown, Pennsylvania, proposed a classification system of five types of sinkholes based upon the most probable failure mechanism. They include: (a) naturally occurring sinks in undeveloped areas; (b) sinks related to deep weathering, faulting, and fractures; (c) construction-related sinks resulting from altered surface drainage and topography; (d) collapses caused by leaking, failed utility lines in urban areas; and (e) sinks triggered by dewatering, mine pumpage, or blasting operations.

The percentage of naturally occurring sinkholes varies considerably from one region to another. In southern Indiana, for instance, the majority of collapses are natural since land development is not intense. In the Allentown study, about 45 % of the sinks investigated were classified as natural. Williams and Vineyard (1976) estimate that 51 of 97 catastrophic collapses in Missouri since the 1930s are natural. However, Newton (1976) concludes that only 50 of the 4000 collapses in Alabama reported since 1900 were completely due to natural processes.

Sinkholes related to faults or other geologic structure can be identified from aerial photography by observing linear trends or clusters of sinks. These sinkholes tend to be deep, extensive, and catastrophic (Perlow, et. al, 1984). Construction-related sinks are common since land development alters surface drainage patterns. Increased surface runoff from a developed site often triggers collapse in adjacent areas during construction. About 54 % of the sinkholes investigated near Allentown (Perlow, et. al., 1984) and 52 % of the man-induced collapses in Missouri (Williams and Vineyard, 1976) were caused by leaky utility lines or altered surface drainage. From these data, it is evident that a significant portion of sinkhole collapses can be prevented if proper engineering design and maintenance is performed. The case studies of sinkholes in Hershey, Pennsylvania and Winter Park, Florida indicate the potential seriousness of altering the groundwater regime.

Irregular Weathering of the Bedrock Surface

The solution process described earlier is concentrated along the joints and bedding planes in the limestone and dolomite bedrock. When seepage is not sufficient to carry away the residual soil developed in the solution-enlarged openings, a pinnacled soil-bedrock contact results. The variation in rock profile often becomes significant. For example, in Tennessee, a 10 to 40 ft difference in depth to sound rock along a 50 ft horizontal distance is not uncommon

(Kemmerly, 1984). Cost overruns are frequent when attempting to predict the amount of rock to be excavated in cut and fill operations for transportation routes. The design and construction of a deep foundation presents a difficult challenge. During a site investigation, increasing the number of borings is expensive and provides little additional information because of the erratic subsurface. Exploration techniques which provide subsurface data of large aerial extent are efficient in planning the location of borings. Geophysical methods are useful in this regard, and are discussed in a later section.

Development of Plastic Residual Soils

The weathering processes in limestone and dolomite result in the development of plastic residual soils, usually clayey silts and silty clays. The clay fraction is often highly plastic, and is responsible for foundation problems for highways and other structures. The more plastic soils are usually located in the residual soil mantle closest to the weathered bedrock. Generally, the soils are siltier near the surface, since much of the clay fraction is leached into the lower horizon. The frequency of plastic clay horizons is evident from engineering data compiled from the region. For example, Frederick soils developed on both uplands and sideslopes in the Mitchell Plain have deep layers of plastic clays and silts (USCS CH and MH soils), with liquid limits approaching 85 and plasticity indices as

high as 55 (Adams, 1984).

Residual soil derived from limestone and dolomite in its natural state is not necessarily poor from an engineering standpoint. The high plasticity may actually reduce the risk of sinkhole collapse, as discussed in the next chapter. The soil is usually free-draining because of the relict joints and bedding planes retained from the original bedrock. However, reworking destroys the natural fabric of the soil and alters the engineering properties. Permeability is decreased, resulting in pavement pumping and poor support when the soil is used as a highway subgrade. High shrink-swell potential, high compressibility, and low shear strength are but a few of the problems associated with these soils.

SITE EVALUATION IN KARST REGIONS

When planning a transportation route or selecting a site for a structure in a karst region, the engineer is faced with many unique problems which can best be solved only with experience gained from past similar situations. However, it is unfortunate that even with such experience, many competent engineers are often fooled by the highly erratic nature of the soil and bedrock associated with karst areas. It is therefore imperative to select the right techniques for site investigation and to know their limitations so that correct interpretations are made from the collected data.

A methodology is proposed which enables the engineer to make a step-by-step evaluation of the site(s) under consideration. The objective is to provide a thorough, but economic geotechnical evaluation of the subsurface conditions. Figure 13 shows two hypothetical cost-uncertainty curves for a site investigation in a karst region. The methodology combines preliminary office studies with appropriate field exploration techniques. It should be possible to reduce the uncertainty of data interpretation while maintaining a reasonable cost, as shown by the lower curve.

Figure 13. Hypothetical Cost-Uncertainty Curve of a Site Investigation in a Karst Region.

A review of the current techniques of remote sensing and geophysical exploration was conducted to determine the most useful methods of site investigation in karst regions. The advantages and limitations of each technique are discussed and evaluated on the basis of cost, reliability, complexity of the survey, and difficulty of interpreting the compiled data. Lawrence County is used to illustrate some of the preliminary study techniques.

Methodology

The proposed methodology of site investigation for locating highway routes or other engineering structures in karst regions is presented in Figure 14. Although the steps in the general outline are not new, such a logical approach to site evaluation is often neglected. Table 5 provides a more detailed description of the six steps of the methodology and suggests alternative techniques which are appropriate for site investigations in karst terrain. Methods of remote sensing and geophysical exploration are listed in the table and are reviewed subsequently. The following sections expand upon and explain the logic of Table 5. Since none of the methods can be used in all situations, the physical conditions and types of problems for which each is best suited are emphasized.

Figure 14. Proposed Methodology of Site Investigation in a Karst Region.

Table 5. Details of Methodology of Site Investigation in a Karst Region.

CONDUCT PRELIMINARY STUDIES	<p>Gather all available data such as</p> <ul style="list-style-type: none"> • physiographic maps • engineering soils maps/reports • agricultural maps/reports • unconsolidated and bedrock geology maps • topographic and drainage maps • overburden thickness or bedrock topography maps • existing airphotos or other imagery
COLLECT RECONNAISSANCE DATA	<p>Conduct airphoto or other remote sensing survey to supplement above sources</p> <ul style="list-style-type: none"> • black and white or color photography • black and white or color infrared photography • thermal infrared imagery • side looking airborne radar (SLAR) • airborne microwave radiometer imagery • multispectral imagery
CHECK GEOLOGIC INDICATORS	<p>Correlate mappable data from above sources with geologic indicators of potential sinkhole collapse</p> <ul style="list-style-type: none"> • prepare engineering soils map • prepare sinkhole density map • check thickness and nature of overburden • check plasticity and drainage characteristics • check mottling on airphotos • check lineaments
SELECT PRELIMINARY SITES	<p>Based on interpretation of above data, select a suitable site or sites for the project based on</p> <ul style="list-style-type: none"> • distance from existing transportation routes • cut and fill requirements • access to exploration and construction equipment • drainage characteristics • solution activity/collapse risk • proximity to engineering construction materials
PLAN FIELD SITE INVESTIGATION	<p>Preliminary reconnaissance techniques</p> <ul style="list-style-type: none"> • ground probing radar • terrain conductivity <p>Detailed surveys</p> <ul style="list-style-type: none"> • gravity • magnetics • seismic refraction • electrical resistivity
PLAN BORING LOCATIONS	<p>Plan an efficient boring scheme that explores anomolous areas identified from previous surveys; combination of soundings, sample borings, and observation wells based on economics, design requirements, and acceptable risk is planned; routine sampling should cover all areas; minimum requirements for highways: 1 boring every 500 ft, 1 sounding every 100 ft.</p>

Preliminary Studies

The importance of preliminary studies to a successful and efficient site investigation cannot be overemphasized. The first step noted in Table 5 is to determine the physiographic characteristics of the area under consideration. For example, Lawrence County is located in the Highland Rim Section of the Interior Low Plateaus Province (Witczak, 1970), and has three physiographic subsections: the Crawford Upland, the Mitchell Plain, and the Norman Upland. The Mitchell Plain is a notorious karst region with many active solution features (Malott, 1922; and Thornbury, 1965). From this information, the engineer can immediately anticipate encountering problems unique to karst terrain.

Many states have mapped the surficial soils on the basis of parent material, landforms, or some other general engineering classification. The maps are usually prepared by county, and are available from the state Department of Transportation. Such maps have been completed for 73 of the 92 counties in Indiana. Other states that have prepared engineering soils maps include Arizona, Kentucky, Illinois, and Rhode Island. They provide the engineer with a quick synopsis of the possible problems in the area by associating parent material and landform with general engineering behavior.

A more comprehensive program of soil mapping is being carried out by the Soil Conservation Service of the U. S. Department of Agriculture. Pedologic surveys of many of the counties in the United States have been completed. Although the soils have been mapped for agricultural purposes, the pedologic soils can be correlated with general engineering properties.

Surficial and bedrock geology maps are good sources of information. State geological surveys in cooperation with the U. S. Geological Survey (USGS) have prepared these maps at a scale of 1:250,000. The extent of a limestone area can be seen immediately from such a map. USGS topographic quadrangle maps are used to determine accessibility to the area. Elevation contours can be compared to the soils and geology maps to determine patterns of weathering, influences of folds and faults, and to locate surface depressions so often found in limestone residual soil.

Where available, drainage maps are useful in locating sinkholes, swallow holes, and subterranean cutoffs. The state of Indiana has completed drainage maps for every county, and it was from such a source that a sinkhole density map was prepared for Lawrence County, to be discussed later. Hydrologic studies are an important source of predicting how construction may alter the groundwater regime. Overburden thickness and bedrock topography maps estimate the depth to bedrock.

Existing airphotos, especially those with stereoscopic coverage, are an invaluable tool for delineating features of a site and for getting an initial feel for the area without actually visiting it. Aerial coverage in the form of standard black and white, or color photography; or black and white, or color infrared imagery is available from state or local planning agencies. Landsat satellite imagery at scales of 1:250,000 to 1:1,000,000 can be obtained from NASA for large site selection studies. It provides computer-enhanced images in four wavelength bands. Although it is not able to distinguish individual solution features because of the very small scale, regional trends such as lineaments and fault zones may be observed (Beaven and Lawrance, 1982).

Reconnaissance Surveys

In the event that the information discussed above is limited or not available, or more detailed data are desired, a remote sensing survey is suggested. The most common type of imagery is either black and white or color photography. Scales range between 1:20,000 and 1:62,500, although larger scales may be desirable for smaller projects (Alexander, et. al., 1974). An experienced airphoto interpreter can delineate subtle topographic or reflectance features which can help locate problem areas such as solution zones. The elements of form and tone are two important patterns which the interpreter looks for, and those observed in a limestone region are summarized in Table 6.

Table 6. Airphoto Patterns of a Limestone Plain.

<u>ELEMENTS OF FORM</u>	
Topography	<ul style="list-style-type: none"> •Basin-studded plain •Narrow U-shaped valleys •Very rolling topography •Scalloped effect around river systems
Drainage	<ul style="list-style-type: none"> •Swallow-hole drainage pattern •Gullies, if present, are short and gradient is steep •Streams entrenched in bedrock exhibit rectangular pattern
Erosion	<ul style="list-style-type: none"> •Solutioning predominates •Knife-like V-shaped notches at tops of gullies •Valleys have deep vertical sides and are flat-bottomed
<u>ELEMENTS OF TONE</u>	
Vegetation	<ul style="list-style-type: none"> •Woodlands have dark tone and are located in steep sideslopes and in areas of poor drainage •Rectangular spacing of trees indicates orchard
Land-Use	<ul style="list-style-type: none"> •Light rectangular tones indicate cultivated areas •Orchards may be present •Limestone quarries common •Karst lakes and ponds
Soil	<ul style="list-style-type: none"> •Soil and rock exhibit light to white tones due to free-draining soil characteristics •Mottled tone indicates bedrock pinnacles

Black and white, or color reflective infrared photos (about 0.7 to 0.9 μm in wavelength) have been used to map slight differences in soil and/or moisture content. Color infrared photos are especially useful if a combination of films and filters are employed. For example, the blue-green band reveals areas of increased water penetration, such as solution zones, and is used for mapping siliceous rocks; the red band enhances cultural features; and the near infrared band amplifies land-water contacts, topography, and differences between exposed carbonate units. Another clue for detecting solution activity in limestone is that vegetation may be stressed in the vicinity of a developing cavity, and a corresponding change in reflectance in this part of the spectrum can be seen (Warren and Wielchowsky, 1973). Near-surface structural features such as faults, fractures, and joints can also be delineated, which is significant since these are the channels along which active solutioning takes place (Alexander, et. al., 1974). Such features are especially enhanced if taken during the leafless season after a rainfall (Warren and Wielchowsky, 1973). Color and color-infrared (4.5-5.5 μm) imagery was used to identify sink-holes and solution zones at potential sites for a large industrial plant in carbonate terrain in Alabama (Barr and Hensey, 1974). Borings confirmed about 85 % of the inferred solution zones.

Thermal infrared imagery records emitted radiation in the 8 to 14 μm wavelength band. Sinkholes are linked to subsurface drainage paths, which have air and water flowing through them, often at temperatures different from the ambient temperature. These thermal anomalies can only be detected under certain conditions at particular times of the day. Early morning and late afternoon are thought to be the best times, since the difference between the ambient air temperature and the subsurface temperature is usually the greatest. However, this need not always be the case, depending on the climate and time of year. The engineer is advised to set up an observation plan or to consult others who have performed a prior survey in the area. Besides finding the optimum time for flying, it is often difficult to distinguish ground patterns, and any air movement greatly alters the thermal pattern (Rinker, 1975). Some studies have been performed using thermography to distinguish differences in soil texture with good results (Kiefer, 1972). Both Stohr (1974) and Rinker (1975) conclude that thermal infrared imagery cannot distinguish sinkholes by itself, but that stereoscopic airphoto coverage and/or ground control is a required supplement.

Side-looking airborne radar (SLAR) is a technique which has shown only limited application for detecting solution zones and cavities. Its value is that it can be used to map solution-enlarged fracture and fault traces. Such a survey

is usually cost-effective only for very large projects. The chief advantages of SLAR are that it can penetrate cloud cover and the survey can be conducted at night, since it is an active system which provides its own radiation source (Warren and Wielchowsky, 1973).

Airborne microwave radiometer surveys have been conducted in the past in an attempt to detect subsurface voids. The detection system records the radiometric temperature of the ground surface, which is believed to be influenced, at least in part, by the presence or absence of subsurface voids. Dedman and Culver (1972) have concluded that there is a correlation between temperature lows and the location of the subsurface voids from radiometric temperature profiles taken over a previously mapped cave. Although the data were not perfect, the technique shows sufficient promise to warrant more research.

A relatively new technique of remote sensing is multispectral imagery, which requires an airborne line scanner capable of recording data in several synchronous spectral bands. The radiation bands are in the visible and near infrared regions of the spectrum. Because of the amount of data which is collected, computer processing techniques are required. The terrain information is recorded directly onto magnetic tape for immediate processing upon completion of the flight survey. To date, the primary use for this tool has been in mapping engineering soils. The technique has

not been perfected and can be considered reliable only on bare soil areas. Good discussions are given by Wagner (1972), West (1972), and Mathews, et. al. (1973). Its applicability to the detection of solution features in karst terrain appears to be limited at best, but it may be a valuable, if expensive tool for delineating residual soil boundaries.

Geologic Indicators of Sinkhole Development

Williams and Vineyard (1976) have compiled a list of geologic indicators of potential collapse in areas of limestone and dolomite in Missouri. They conclude that catastrophic collapse is enhanced if the residual soil cover (a) is between 40 to 100 ft thick; (b) contains the relict structure of the parent rock; (c) has a low plasticity clay fraction (USCS ML soil; AASHTO A-7-5 soil); and (d) is poorly drained. Areas where surface water is diverted to the subsurface and which show a high density of natural sinkholes may also indicate a higher potential for collapse. Colluvial and alluvial deposits and glacial till generally have a lower risk of collapse. Deeply incised valleys indicate a steep hydraulic gradient which increases rates of solutioning and underground erosion, thereby increasing the potential of sinkhole development.

Perlow, et. al. (1984) developed some general correlations for potential collapse in the Allentown, Pennsylvania

area. The data revealed that man-induced sinkholes from construction and leaky utilities occurred more often in soil where the depth to bedrock is only 5 to 10 ft. A moderate risk exists in soils 10 to 15 ft deep, while the hazard is low where the residuum is greater than 20 ft, especially for construction-related sinks. Other conclusions from the study are that a highly fractured bedrock indicates a moderate to high potential of collapse, and sinks related to faulting and pumping tend to be catastrophic.

The Allentown study also pointed out potential clues for assessing the risk of sinkhole formation using remote sensing data, especially aerial photography. A mottled tone on the airphotos of a limestone region indicates a pinnacled bedrock surface. Differences in the moisture content of the soil due to variable subsurface drainage characteristics are responsible for this pattern. Moderate to severe mottling indicates a medium to high risk of natural and construction-related sinkhole development. Trend-lines and clusters of sinks can be identified from airphotos and indicate areas of high risk. Another important clue to potential sinkhole development is the formation of ghost lakes after periods of heavy rainfall. They often indicate the existence of filled solution channels which could result in collapse if the infill soil is eroded or piped out. Remnant ghost lakes are seen when frost occurs after freezing and thawing. Solutioning is also more active along natural drainageways.

Some minor differences in the details of the geologic indicators are evident from the above discussion. For instance, depth of the residuum where collapse risk is highest differs from 5 ft in the Allentown study to over 40 ft in Missouri. However, after comparing the geology of southern Indiana with that of Missouri and eastern Pennsylvania, a list of geologic indicators of collapse potential in the Mitchell Plain was compiled based on these two studies, and is presented in Table 7.

Preparation of an Engineering Soils Map

The preparation of an engineering soils map from available preliminary or reconnaissance data is useful for delineating parent materials and landforms of the area under consideration. A suitable map scale, depending upon project size, must be chosen. For example, the county engineering soils maps in Indiana are prepared at a scale of 1 in. to a mile. Methods of airphoto interpretation are described in Frost, et. al. (1953). For large projects, preparation of a soils map is the first detailed step for identifying specific areas where sinkhole development and other karst related problems are most likely to be encountered.

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Table 7. Geologic Indicators of Sinkhole Collapse
in the Mitchell Plain of Southern Indiana.

INDICATOR OF HIGH COLLAPSE RISK
<ul style="list-style-type: none"> •Soluble bedrock near surface (limestone or dolomite) •Overburden 5 to 20 ft thick •Undisturbed residuum with relict structure of parent bedrock •Residual soil with low plasticity clay fraction (ML;A-7-5) •Poorly draining soil •Highly fractured/jointed, thinly-bedded rock •Deeply incised stream valleys •Swallow-hole type drainage •Extensive subsurface drainage network •High density of natural sinkholes •Depressed areas or natural drainageways •Formation of ghost (remnant) lakes •Severe mottling on airphotos •Intersections of lineaments

(Modified from studies by Williams and Vineyard, 1976; and Perlow, et. al., 1984)

Preparation of a Sinkhole Density Map

Another example of the use of preliminary information or data to aid in site selection is the sinkhole density map of Lawrence County shown in Figure 15. It was developed from the 1953 drainage map of the county (Magnusson, 1953), which was prepared from 1938 photography at a scale of approximately 1:20,000. Each sinkhole identified from these photos is represented on the drainage map with a dot or small circle. The map of Figure 15 was prepared by simply counting the number of sinkholes in each Land Survey Section (a square mile).

Although the density of sinkholes is high in parts of the Mitchell Plain, new collapses are infrequent because land development is not intense. For this reason, the sinkhole density map is current even though it was prepared essentially from 1938 photography. The contours of 20, 50, and 100 sinkholes per square mile are arbitrary, but seem to define areas of low, moderate, intense, and very intense sinkhole development. Such a map can be prepared at other scales, depending on the size and scope of the project.

Figure 15 is particularly useful for planning a new transportation route in Lawrence County. Regions with a high risk of collapse can be avoided or at least identified. Correlation of sinkhole density with limestone formation is also possible by comparing the map with a bedrock geology

Approximate Scale 1 : 250,000

LEGEND

SINKHOLES PER SQUARE MILE

0 - 20

20 - 50

50 - 100

> 100

N

Figure 15. Sinkhole Density Map of Lawrence County.

map as shown in Figure 16. The finer-grained limestone of the Blue River Group has a much more pronounced development of collapses than the coarser-grained limestone of the Sanders Group. Such a correlation can be extrapolated to areas of the same geology outside the county if the engineer keeps certain factors in mind. Changes in climate and land-use, as well other factors listed in Table 7 may make such a conclusion invalid. Williams and Vineyard (1976) stress that areas which show little indication of natural sinkhole development or subsidence may still be hazardous.

Identification of Lineaments

Benson (1984) and Ruth and Degner (1984) illustrate the potential use of surface lineaments identified from aerial photography for indicating areas of future sinkhole collapse. They mapped lineaments primarily from soil and vegetation tonal alignments and linear trends exhibited by old sinkholes. An area of high sinkhole density in southern Lawrence County was examined to assess the potential of using this technique in the Mitchell Plain. Linear patterns from soil and vegetation were not easily discernable from the photos (scale 1:24,000), but several trends in sinkhole alignment were observed, as shown in Figure 17.

Some of the identified lineaments are admittedly questionable, but the linear pattern exhibited by most of the sinkholes is unmistakable. This is even more so in light of

Scale: 1 : 250,000

LEGEND

	Borden Group
Siltstone, shale, sandstone,
and some limestone		West Baden Group
Shale, sandstone, and
limestone, isolated deposits
semiconsolidated sand
	Sanders Group
Mostly coarse-grained
limestone		Stephensport Group
Limestone, sandstone,
and shale
	Blue River Group
Mostly fine-grained
limestone		Raccoon Creek Group
Shale, sandstone, lime-
stone, clay, and coal

Figure 16. Bedrock Geology Map of Lawrence County.
(After Gray, et. al., 1970)

Figure 17. Lineament Pattern of Sinkholes in South-Central Lawrence County (T4N, R1W).

the information in Figure 18, which shows the mean trends of major joint sets of the five major limestone formations which outcrop in the Mitchell Plain in south-central Lawrence County. The study area is primarily in outcrops of the St. Louis, Ste. Genevieve, and Paoli Limestones, and the joint trends are shown in the top left corner of Figure 17. The northwest-southeast and east-west strikes of the majority of lineaments is obvious, and correlates well with the joint trends of these limestone formations. It should be noted that a large portion of sinkholes occur at the intersection of lineaments.

Figure 17 was prepared by marking the location of sinkholes on an acetate sheet superimposed over the airphoto. Stereoscopic viewing aided in identifying all topographically depressed areas believed to be sinkholes. Another acetate sheet was placed on this, and linear patterns were identified and marked. It should be noted that the particular linear trends in the joint sets of these formations were not known by the author previous to this exercise.

Another aspect of Figure 17 is the almost total absence of sinkholes in the sandy areas overlying limestone and in the flood plain, supporting some of the conditions in Table 7. Although it is not obvious from the Figure, the limestone areas devoid of sinkholes are, almost without exception, topographic highs, supporting the Ruth and Degner (1984) contention that sinkhole formation is more likely to

<u>FORMATION</u>	<u>CHARACTER OF JOINTS</u>	<u>MEAN TREND OF MAJOR JOINT SETS</u>
Chester limestones	Prominent joints, with consistent directional trends. Most large joints extend through the entire thickness of a formation, and commonly have spacings between 10 and 25 feet. Nearly all joints are vertical.
Paoli and Ste. Genevieve Limestones	Numerous small joints with relatively diverse orientations. Spacing of 2-10 feet is most common. Some oblique joints with dips as low as 30° .
St. Louis Limestone	Well defined directional trends. Vertical extent of joints commonly limited because of heterogeneous lithology. Nearly all joints are vertical.
Salem Limestone	Large master joints with spacing from 10 to 50 feet, with very pronounced directional trends. Many joints probably extend the full thickness of the formations. Nearly all joints are vertical.
Harrodsburg Limestone	Prominent joints in the upper beds similar to that of the Salem, decreasing in size and spacing toward the bottom of the formation. Nearly all joints are vertical.

Figure 18. Character of Jointing in South-Central Lawrence County. (From Palmer, 1969)

occur in depressed areas where surface runoff is concentrated. It is recommended that, if available, larger scale photography be used to better identify soil and vegetation tonal alignments, and to reduce the crowded appearance of sinkholes, which tends to obscure more local trends in alignment.

Selection of Preliminary Site(s)

In the event that the actual site has not been established beforehand, the engineer should have enough information at this point to select the best one or two areas for the facility. The selection should be made by considering the following points. The site should: (a) be located within reasonable distance of existing transportation routes; (b) have an approximate balance of cut and fill in the case of highways and other transportation corridors; (c) be accessible to exploration and construction equipment; (d) have a minimum of solution activity; (e) have favorable drainage characteristics; (f) contain a minimum of potential solution zones such as caves, joints, fractures, or faults; and (g) be located in reasonable proximity to engineering construction materials such as fill from borrow pits, water, and granular soils for the construction of drains. If it is impossible to find a site that is favorable with respect to all of the points listed above, the engineer must carefully balance the positive and negative factors to ensure that the best available site is chosen.

Field Site Investigation

When the site or sites have been selected, a field investigation is performed. Problem areas can be identified from the preliminary reconnaissance work, and it is within these that much of the subsurface investigation should be concentrated. However, no area should be ignored, since what is construed to be a "safe" area may prove to be troublesome later.

As mentioned earlier, the highly erratic subsurface profile of a limestone area necessitates the use of efficient investigation techniques which provide data of a large areal extent. Geophysical techniques are most efficient, since they can provide a variety of subsurface data from surveys conducted at the surface. Clayton, et. al. (1982) cite five factors which must be weighed before any particular geophysical technique is selected:

(a) What will the survey tell the engineer? Will it distinguish boundaries, reveal a buried feature, or expose an underground cavity? (b) Are the physical properties which are to be measured sufficiently different to allow for accurate detection of subsurface anomalies? (c) Does the subsoil condition and geometry resemble the simple model upon which the interpretation of the data will be made? (d) Will interference such as noise, old services, metal refuse, lack of space, power lines, or physical changes in the atmosphere adversely affect the survey? (e) Is previous subsurface data available so that correlations can be made with the geophysical data?

Points (a), (b), (d), and (e) can probably be answered sufficiently, but point (c) is often difficult to assess.

Day and Wagener (1981) recommend that site investigations be performed in two or three phases, especially in regions unfamiliar to the engineer. Each phase is planned separately, based on the results of the previous phase. This assures that correct investigation techniques are chosen, and that one is not committed to a method which, after analysis, yields unsatisfactory results. Techniques of field investigation are grouped into two phases: preliminary reconnaissance and detailed surveys. A later section discusses a third phase, which is the planning of boring locations.

Preliminary Reconnaissance Techniques

It is often advantageous to conduct a preliminary reconnaissance survey to identify general anomolous areas worthy of more detailed investigation. Two relatively new geophysical techniques are ground probing radar and terrain conductivity. They are fast and provide continuous subsurface measurements across a site. For large projects, preliminary borings or soundings may also be used to confirm anomolous areas identified from the geophysical survey.

Ground-Probing Radar

Impulse or ground probing radar is a relatively new technique which is the subject of some recent studies. Spencer, et. al. (1976) employed this method in an attempt to delineate soil-rock contacts and to detect old mine

workings in a chert bedrock. The results were inconclusive until they were coupled with electrical resistivity data. Penetration depth was limited to 8 ft because of the presence of a moist clay soil. Fountain (1976) reports three cases in which radar was used for detecting subsurface cavities. A cave site in Florida was surveyed with moderate success; some large cavities and soil-rock interfaces were detected to a depth of 15 ft. At another test site, the radar was able to penetrate more than 30 ft, but the signal wavelength was too long to detect the small cavities at the site. At a site in Alabama penetration was very limited due to high electromagnetic attenuation in the soil.

Ballard (1983), in a report by the U. S. Army Corps of Engineers, discusses the results of radar techniques applied to three test sites by four independent investigators. He concludes that the method should be considered only where earth materials allow sufficient transmission of electromagnetic signals, such as sites (a) with an overburden of sand or silty sand; (b) where the water table is below the area of interest; and (c) where the bedrock is composed of low conductivity rock, such as soft limestone or granite. Materials with very high dielectric constants such as moist clays and loess deposits severely limit propagation of electromagnetic signals. Cross-borehole techniques were also used and successfully detected both air and water-filled cavities as long as they were regular in shape and the

bedrock was not highly fractured.

The reliability of this technique is dependent on the electromagnetic properties of earth materials at the site, the structure of the bedrock, and the shape and size of the cavities. The equipment must be chosen so that the output signal wavelength is comparable in size to the cavities which are to be detected. If these features are not too deep, the technique should be given serious consideration.

Terrain Conductivity

Terrain conductivity is a relatively new geophysical technique valuable for quick identification of subsurface anomalies. The system operates on the principle of electrical conductivity, which is directly proportional to the amount of moisture, salts, and free ions present in the subsurface (Perlow, et. al., 1984). The method is analogous to electrical resistivity, except that it measures conductivity, the inverse of resistivity. It has an advantage over resistivity because it does not require electrode contact with the ground and therefore surveying is much faster. Speeds up to five miles per hour are attainable (Benson, 1984).

The conductivity method is particularly effective in locating lateral anomolous conditions and deep zones of weathering. Moist residual soils show a high conductivity as opposed to a massive sound limestone bedrock, which

exhibits a much lower conductivity. Preparation of contours from the measured values is useful in identifying areas of deep weathering, which are indicated by anomolous highs. Correlation with depth to bedrock is also possible (Perlow, et. al., 1984).

Detailed Surveys

Anomolous areas identified by the reconnaissance geophysical surveys can now be mapped in more detail by more sophisticated geophysical methods, combined with subsurface borings. Conventional methods include gravity, magnetics, seismic refraction, and electrical resistivity. Specific applications, advantages, and limitations of each are discussed below.

Gravity

Gravimetric techniques detect anomalies in the earth's gravity field due to differences in density among earth materials (Griffiths and King, 1981). Colley (1963) demonstrated the potential of this method for detecting underground cavities twenty years ago. Omnes (1976) describes a microgravity survey which was successful in establishing the presence of a 6 by 8 ft adit dug in a granite hill at a depth of up to 20 ft. He also discusses other surveys which located cavities and underground quarries in limestone. Brooke and Brown (1975) describe a gravimetric survey over a previously mapped cave. They concluded that cavities could

be detected near the surface, but the complexity of the survey and data reduction rapidly increases the cost. Fountain (1976) reports on gravity studies at locations in Florida and Alabama. The technique was successful in locating only the largest rooms of an extensive cave system in Florida and was unsuccessful because of ground vibrations induced by heavy traffic at the site in Alabama.

Butler (1983), in a study of Medford Cave in Florida, concludes that three conditions limit the application of microgravity to site investigations in limestone regions: (a) extreme topographic variation; (b) lithological noise, i.e. spacial fluctuations in soil type and depth to bedrock; and (c) close spacing (<1.2 times the depth) of cavities. The technique was successful in delineating the plan of the cave system, whose roof varied in depth from 10 to 30 ft. It also indentified unknown solution features, small clay pockets, and limestone pinnacles. A simple geometric model was assumed to estimate the depth and size of the cavities. Butler, et. al. (1983) used gravity to delineate the general outline of water-filled cavities at a depth of 90 ft at Manatee Springs, Florida. Details were not as clear as at Medford Cave because of higher lithological noise and greater depth. A study by Day and Wagener (1981) concluded that gravity methods are unable to identify narrow features in the rock surface at large depths (>50 ft). Gravity may be a useful technique for large projects if coupled with

another method. It is expensive and often not reliable enough by itself.

Magnetics

Magnetic surveying is a technique which has shown limited potential in locating subsurface cavities (Clayton, et. al., 1982). McDowell (1975) describes a magnetic survey conducted over clay-filled sinkholes in chalk. Although the method was successful in locating many of the sinkholes, he concludes that the slight difference in susceptibility of the two materials required the site to be free of any metal refuse which would interfere with the signal. In addition, the dimensions of the feature must be greater than the depth of burial. A study by Butler (1983), using magnetic surveying at the Medford Cave site in Florida, found that the technique is not useful for the general detection of voids, except for clay-filled cavities or pockets at very shallow depth (<10 ft). The size of the feature must be of the same order as its depth of burial, and the magnetic susceptibility contrast between the clay and limestone must be large. The primary applications of magnetics are detecting old mine shafts and prospecting for minerals (Griffiths and King, 1981).

Seismic Refraction

Seismic refraction is a useful technique for delineating soil-bedrock interfaces in a karst region, and it provides a quantitative measure of the depth to bedrock. The method has some serious drawbacks since the theory on which the data interpretation is based assumes two parallel layers, which almost never exists in a limestone residual soil. Clayton, et. al. (1982) list some of the uses of seismic refraction as (a) aiding in "rippability" assessments, (b) locating buried channels, faults, and fault zones, and (c) profiling the depth to bedrock. The problem of highly irregular contacts will produce ambiguous travel time data. Buried channels may give results similar to those from a three layer system. Setting up longer shots should reduce this problem (Clayton, et. al., 1982).

Curro (1983) reports a study of several seismic refraction techniques used at the Medford Cave site. The constant-spacing surface method was successful in delineating cavities larger than 4 ft up to a depth of 30 ft. Cross-hole P-wave velocity tests were also successful, and used correctly, may reduce the required number of boreholes by a factor of two (for investigating cavities larger than a few feet in diameter).

Some interesting data were reported by Brooke and Brown (1975) in which refraction was used to locate a small cave

and fault trace north of Santa Cruz, California. Anomalous time delays were believed to reveal the presence of the cave, and a time offset was attributed to the fault. Such data is encouraging since they indicate the many applications of the technique. However, in the case of limestone, soil-bedrock profiling is the most efficient use of seismic refraction. Other methods are more applicable for detecting of subsurface cavities.

Electrical Resistivity

Much of the recent literature on detection of subsurface cavities in limestone has concluded that electrical resistivity methods are very efficient. Fountain (1976) gives detailed discussions of three resistivity surveys in which the pole-dipole electrode configuration was used. The Medford Cave site was thoroughly investigated using resistivity. Narrow vertical solution pipes were detected at another Florida site. A number of resistivity highs at a site in Alabama were found to be soil-bedrock interfaces. Spencer, et. al. (1976) describe a constant depth resistivity traverse and geoelectrical sounding survey using the Wenner configuration, which was the most successful of three geophysical techniques in delineating the subsurface profile at a site in Kansas. The results were superior to those of passive microwave radiometer and impulse radar profiling techniques. Brooke and Brown (1975) provide data from a resistivity survey using the Wenner array that were

successful in detecting a small cave.

Bates (1973), in a comprehensive study of geophysical techniques used for detecting subsurface cavities, concludes that electrical resistivity methods give the best results. He recommends the modified Bristow method in exploration, which is equivalent to the pole-dipole array. The configuration is a form of the central electrode array, but the current sink electrode is placed an infinite distance from the source electrode, or essentially 5 to 10 times greater than the desired penetration depth. The array is shown in Figure 19, where ρ_a is the apparent resistivity of the material. The configuration allows both vertical sounding and horizontal profiling and is very rapid. Good discussions on the details of the technique are given by Fountain (1976), Clayton, et. al. (1982), and Bates (1973). Interpretation procedures can be found in Bates (1973) and in the U. S. Army Corps of Engineers manual "Geophysical Exploration" (1979).

Of course, as with any geophysical technique, there are limitations. Both high and low resistivity anomalies may indicate subsurface voids, depending on the presence and nature of interstitial water and debris. In some cases, a balance between cavity size and amount of moisture present may result in no resistivity anomaly (Brooke and Brown, 1975). It is advantageous to know the local groundwater conditions before interpretation of resistivity data is

$$\rho_a = \frac{2\pi R}{b \left[\frac{1}{a(a+b)} + \frac{1}{d(d+b)} \right]}$$

For General Configuration

$$\rho_a = \frac{2\pi R a(a-b)}{b}$$

For Modified Bristow Array
Where R = Measured Resistivity

Figure 19. Central Electrode Configuration. (After McDowell, 1975)

completed. Highly heterogeneous ground introduces errors due to lateral changes when conducting an expanding survey for vertical sounding. Electromagnetic coupling between current and potential electrode cables may occur, so care must be taken that they are kept separated. Interference from overhead wires, buried services, metal fences, etc., may be a serious problem, especially in urban areas (Clayton, et. al., 1982).

Other Methods

Passive microwave radiometer field studies have not been very successful in mapping underground cavities. In a study to evaluate several geophysical techniques of cavity detection, Spencer, et. al., (1976) conclude that microwave penetration is very limited except in the driest natural environments. The measured brightness temperature could not be correlated with the presence of an underground opening on a consistent basis. A report by the Federal Highway Administration (1972), based on the same study, concludes that the technique is not currently applicable to void detection, but recommends more research.

Cooper (1983) reports on a Corps of Engineers study of three new methods of cavity detection: acoustic resonance; sonar; and self-potential. The acoustic resonance technique employs an energy source, such as a loudspeaker, which is lowered into an air-filled cavity. The source is tuned to

the estimated resonant frequency of the void, and ground amplitudes are measured at selected locations with very sensitive geophones. Accessibility to the cavity, either through an opening at the surface or through a borehole, is required. Data interpretation is based on the assumption that amplitude highs are located directly above the cavity. Results of an acoustic resonance study at Medford Cave indicate that the method is useful for delineating shallow (<20 ft) air-filled cavities less than 10 ft in horizontal dimension. Increasing geologic complexity and depth of burial drastically reduce data resolution.

The sonar technique operates on the same principle as acoustic resonance, but relies on a sonar source to excite water-filled cavities. Depth of submergence of the equipment was the main limitation during a study at the Manatee Springs site. Results indicate that signals could be easily detected at distances of 200 ft, but correlation between amplitude highs and the plan of the main cavity system was impossible. Cooper recommends that another sonar source more suitable to the site conditions be used.

The self-potential (SP) method operates by measuring direct current (dc) earth currents produced by the movement of pore water and spacial variation in chemical composition of the strata and pore fluid. An electric field is measured at the surface, and anomalies have been used previously to detect ore bodies, seepage zones in dams, and for geothermal

studies (Cooper, 1983). Cavity detection is possible since water flowing through underground channels creates a net negative electrical charge in the surrounding mineral crystals. Fresh water produces a significant variation in the SP field measured at the surface. A reference electrode is placed at a large distance from the survey area, preferably upstream if the direction of groundwater flow is known, and a survey electrode is traversed along a predetermined grid pattern. DC potential measurements are made between the two electrodes using a high resistance voltmeter (to minimize polarization effects). A study at Manatee Springs was successful in detecting a large solution channel at a depth of 90 ft, and a crude delineation of the feature was achieved. However, depth determination is impossible with the current technology.

These three techniques, while deserving additional research, are not currently applicable to everyday site investigation studies. Acoustic resonance requires access to air-filled voids; sonar is limited by equipment; and SP does not provide sufficient data resolution for accurate mapping. None of the methods provides estimates of cavity depth.

Evaluation of Geophysical Methods

Table 8 presents advantages and limitations of each geophysical technique discussed above. It also lists the

Table 8. Evaluation of Geophysical Techniques for Site Investigation in a Karst Region.

METHOD	PRINCIPLE/ INTERPRETATION	APPLICATIONS	ADVANTAGES	LIMITATIONS	RELATIVE COST	RATINGS*
Ground probing or Impulse Radar	Receives analogue signal based on dielectric constant of earth material/ Qualitative	<ul style="list-style-type: none"> ● Mapping of soil layer boundaries ● Detection of buried drums ● Detection of sub- surface voids 	<ul style="list-style-type: none"> ● Quick survey can be performed from vehicle or by hand ● Profile can be obtained in the field 	<ul style="list-style-type: none"> ● Penetration depth may be restricted by moist soil layer ● Signal wavelength must not be greater than the subsurface void dimension 	Low	35/35
Terrain Conductivity	Measures relative electromagnetic conductivity of soil and rock/ Qualitative	<ul style="list-style-type: none"> ● Mapping of weathered fracture zones ● Detection of solution pits ● Correlation with depth to bedrock 	<ul style="list-style-type: none"> ● Quick survey can be performed from vehicle or by hand (up to 5 mph) ● Profile can be obtained in the field 	<ul style="list-style-type: none"> ● Penetration depth is limited ● Interference from buried metallic objects 	Low	35/35
Gravity	Measures earth's gravity field; detects density variations/ Qualitative; but some depth and size determinations possible using simplified models	<ul style="list-style-type: none"> ● Mapping geologic structure ● Locating near- surface cavities 		<ul style="list-style-type: none"> ● Field techniques are slow ● Many corrections of field data are required ● Accurate topographic survey often necessary ● Anomalies are usually small 	High to Intermedi- ate	35/35

Table 8. Continued.

METHOD	PRINCIPLE / INTERPRETATION	APPLICATIONS	ADVANTAGES	LIMITATIONS	RELATIVE COST	RATINGS*
Magnetics	Measures magnetic field intensities; detects differences in magnetic susceptibility of earth materials/ Qualitative	<ul style="list-style-type: none"> ● Mapping of clay-filled sinkholes ● Location of abandoned mine shafts, buried services, and subsurface voids ● Location of step faults and fault zones 	<ul style="list-style-type: none"> ● Equipment compact and portable ● Large areas may be covered quickly ● Data corrections simple and easy to apply 	<ul style="list-style-type: none"> ● Interference from power lines and iron objects ● Magnetic remanance ● Temporal variations and magnetic storms ● Highly complex geology makes data interpretation difficult 	Low to Intermediate	0/25
Seismic Refraction	Measures travel time of induced seismic waves refracted at soil-rock boundaries/ Quantitative; can detect bed thickness and dip angle in simple cases	<ul style="list-style-type: none"> ● Depth profiling of soil and rock ● Location of buried channels and faults ● Determination of throw of step faults ● "Rippability" assessments ● Evaluation of rock mass fracture state ● Determination of dynamic elastic moduli of soils and rocks 	<ul style="list-style-type: none"> ● Accuracy high at shallow depths ● Depth and velocity contrast easy to determine ● Field techniques are simple ● Equipment compact and portable 	<ul style="list-style-type: none"> ● High background noise level ● Velocity contrast between layers may be poor ● Velocity inversion ● Hidden layer problem ● Constant velocity increase with depth ● Water table may interfere ● Ground-coupled air waves (in loose/soft surface layer) 	Intermediate to High (but covers a large area)	4/2

Table 8. Continued.

METHOD	PRINCIPLE/ INTERPRETATION	APPLICATIONS	ADVANTAGES	LIMITATIONS	RELATIVE COST	RATINGS*
Electrical Resistivity	Measures relative electrical resis- tance of soil and rock in terms of resistivity/ Qualitative; but some depth deter- minations are possible	<ul style="list-style-type: none"> ● Mapping of both vertical and lateral variations in lith- ology ● Determination of thickness and nature of surface deposits ● Location and mapping of buried channels, sinkholes, mine shafts, and sub- surface cavities ● Groundwater investigations ● Location of con- struction materials (sand and gravel) ● Porosity and water salinity assessments 	<ul style="list-style-type: none"> ● Equipment compact and portable ● Field techniques are rapid ● Interpretation of data is straight forward 	<ul style="list-style-type: none"> ● Interference from electric power cables ● Hard ground surface ● Insufficient elec- trical coupling between electrodes and ground ● Poor resistivity contrast between layers due to high degree of overlap of resis- tivity ranges for soils and rocks ● Not well suited for complex subsurface conditions ● Principles of equivalence and suppression in depth sounding ● Dipping interfaces in depth sounding 	Intermediate (but covers a large area; best suited for medium to large projects)	3/4

*Ratings:

0. Not applicable

1. Limited application

2. Could be used, but not best approach

3. Excellent potential, but not fully developed

4. Excellent approach; state-of-the-art well developed

5. Should be supplemented with another method to clarify data

The first rating is for the
location of the soil-bedrock
interface; the second is for
detection of subsurface
cavities

primary uses of each and gives a qualitative assessment of the cost of a survey. The table was compiled primarily from Clayton, et. al., (1982), Butler, et. al., (1982), and "Geophysical Exploration" (1979), although conclusions were drawn from other references discussed in the text.

Location of Borings

The efficient location of borings is perhaps more critical in karst regions than in any other geologic setting. It requires a sufficient knowledge of the site as interpreted from the data compiled during the preliminary study, reconnaissance survey, and geophysical investigation. Clayton, et. al. (1982) list four principal factors that should be considered for a successful drilling program: (a) a relationship between structure, borehole layout, frequency, and depth; (b) a need for sample quality and quantity related to the required geotechnical parameters, and the soil type and variability; (c) site supervision, to ensure that drilling and sampling are carried out correctly and good records are kept; and (d) prompt sample description and preparation of borehole and pit records so that the drilling program can be modified as work proceeds.

It is recommended that the site be broken into four zones, each with a higher risk of collapse or other instability problem. Zone I includes all areas that are away from the proposed structure and are not influenced by the

foundation, and which contain no anomalies or suspected areas of instability. Zone II includes all areas away from the structure that exhibit signs of potential instability or have anomalies indicated by the survey data. Zone III includes the area within the influence of the structure but which shows no sign of incipient collapse or other instability. Zone IV encompasses the area within the perimeter of the structure and which exhibits collapse features, anomalous data, or other indicators of potential instability. It is obvious that an optimal site location minimizes the area of Zone IV.

Once the location of the structure, transportation route, or facility has been established (often before any field exploration), these four zones are mapped based on the accumulated data. Zone I is explored with the least detail, but still must not be overlooked since the initial data always have some uncertainty, and future expansion of the facility may occur. The rule of thumb recommended by Hvorslev (1949) of a regular spacing of 100 ft, or one boring for every 10,000 sq ft, is usually adequate. If the material is found to be very uniform, the spacing of sample borings may be increased to 400 to 500 ft. However, soundings should be taken at a maximum spacing of 100 ft. If unexpected conditions are encountered, the spacing is decreased at the discretion of the engineer.

Zone II should have routine borings located as specified for Zone I. In addition, soundings or sample borings are used to verify anomalies identified from the geophysical survey and potential areas of instability as revealed from remote sensing data. A minimum anomaly reading should be chosen, above which a direct boring is used for verification. This value depends on the geophysical method used, the minimum dimensions of a cavity that can be tolerated (missed) at a given depth, and other geologic and hydrologic factors. The choice should be left up to the judgment of a geophysicist experienced with the geology of the area. Although vertical borings are most often used, inclined borings are recommended if there is some suspicion that a cavity is aligned in a vertical direction. The width of the feature can then be estimated.

Zone III requires more detailed investigation than Zones I or II, even though no anomalous areas are indicated, since overlooking any solution feature may prove disastrous later. Maximum spacing of sample borings should be 100 ft (10,000 sq ft area) for relatively uniform conditions, and reduced accordingly when erratic conditions are encountered. If subsequent exploration reveals unexpected solution features, the area should be reclassified as Zone IV. More detailed investigation is then performed.

Zone IV is the most critical area of the site, and careful investigation is required. Spacing of sample

borings should be less than 100 ft, and preferably in the range of 25 to 50 ft, depending on the erratic nature of the subsurface (Hvorslev, 1949). If areas classified as Zone IV are numerous and the project budget limited, soundings should be taken where significant anomalies are located. Accessible explorations such as test pits and trenches are recommended for direct assessment of the subsurface conditions. These are most applicable where the anomalous features are located close (<15 ft) to the ground surface, but deeper excavations are possible. Table B1 of Appendix B outlines the applications and limitations of several types of accessible explorations reported by Price, et. al. (1981).

Depth of Borings

The depth of borings for verification of a subsurface anomaly should extend through the anomaly and into sound material. Depth for routine borings depends on the type and size of the proposed facility. For foundations, this is usually taken as 1.5 times the effective width of the foundation below the proposed founding level (Clayton, et. al., 1982). For highways, the depth should be at least 15 ft, where the final grade is close to the existing ground surface; 15 ft below the finished road level for cuts; and at least 1.5 times the embankment height in fill areas (Clayton, et. al., 1982). In addition, Couch (1984) recommends at least one deep exploratory boring to assess the potential

for a large collapse at the site. This is usually unsampled, and should extend into a relatively unweathered zone of the bedrock. Such a boring is recommended about every 1000 ft for transportation routes.

Boring Types

Several types of drilling are used in karst areas. Perlow, et. al. (1984) recommends air-track drilling for rock sounding and verification of cavities since it is fast and economical, and may be used to establish approximate rock quality. Percussion drilling is used extensively in the dolomite regions of South Africa (Day and Wagener, 1981). Sampling of soil and rock is performed with rotary core drilling. The most common drilling types, and their applications and limitations are presented in Table B2 of Appendix B (Price, et. al., 1984).

Evaluation of a Probabilistic Approach

Recent studies at the Massachusetts Institute of Technology (Grant, 1973; and Baecher, 1978) attempt to assess the risk of overlooking a cavity by using probabilistic methods. Grant (1973) specifically looks at exploration of subsurface cavities in limestone. Simplified models such as spheres and horizontal cylinders are used to represent small cavities, tunnels, and large caverns. The probability of finding a cavity is estimated based on the spacing of borings (or geophysical readings), and the depth to, and

geometry of the feature. However, extensive boring data are needed to estimate the porosity (ratio of the volume of voids to the total volume) at a given range in depth. A probability density function of both porosity and cavity size is required. Several boring schemes with estimated costs are prepared, and an analysis based on decision logic is performed to obtain the optimum solution.

Such an approach, although possessing great potential, is difficult to apply at this time. Uncertainty in choosing parameters for the analysis, the extensive subsurface data needed beforehand, and the simplified cavity models reduce the practicality of the method, but additional research is warranted.

HYDROLOGIC ASPECTS OF PROBLEMS IN KARST REGIONS

Water is the key factor in the geomorphologic processes responsible for development of karst features on carbonate rock. Fluctuations in the groundwater table and flow of subsurface water erode soil from cavities formed in the residuum over solution-enlarged openings in the bedrock. The resulting depressions are often the focal points of surface drainage, and any alteration of the landscape may adversely affect the drainage characteristics of the surrounding area. It is therefore important for the engineer to consider hydrologic as well as geologic factors for assessing potential land subsidence or collapse when constructing over carbonate bedrock. Several studies are reviewed to illustrate the significance of water in the development of sinkholes in terrain with characteristics similar to the Mitchell Plain. Construction techniques dealing with the hydrologic aspects of sinkholes and their formation are described. Some examples of the use of sinkholes for drainage of surface runoff are also included, and suggested procedures are outlined.

Case Histories

Important insight is gained from studying case histories of man-induced sinkhole collapses triggered by changes in the hydrologic environment. Two separate mechanisms are distinguished: (a) subsidence and collapse triggered by a lowering of the groundwater table and subsequent water level fluctuations; and (b) subsidence, collapse, and flooding caused by alterations in the surface characteristics of the adjacent area. Examples of each are numerous, and several are discussed below.

Groundwater Withdrawal

Land development, construction of transportation routes, mining, and quarrying operations produce changes in the local groundwater system. Water supplies are needed for the expansion of municipalities, residential communities, and industrial centers, and groundwater supplies are often the only alternative available. As new wells tap the subsurface, a decline in the water table results. Dewatering operations for mines and quarries produce deep cones of depression and increase the hydraulic gradient in their vicinity.

Several examples are described in the literature. The large sinkholes which develop in north-central Florida are triggered by groundwater withdrawal from rapid urbanization. The 1981 Winter Park sinkhole is a dramatic testimony to the

suddenness with which collapse can occur (Jamaal and Associates, 1981). Foose (1953) presents a thorough case study of intense sinkhole development near Hershey, Pennsylvania in 1949. A limestone quarry suddenly doubled the rate of pumping, producing over 100 collapses in less than two months. A tragic case is reviewed in Legget and Karrow (1983) in which 29 lives were lost at the West Driefontein diamond mine near Johannesburg, South Africa in 1962. Dewatering of the mine caused a crusher plant to suddenly disappear into a large sinkhole formed in the Transvaal Dolomite.

Newton and Hyde (1971) describe the occurrence of over 200 sinkholes during the period 1963-1970 in an industrial subdivision adjacent to two quarries in the Ketona Dolomite near Birmingham, Alabama. Pumping of the groundwater produced a drawdown of as much as 140 ft, and subsequent collapses produced major pollution problems, damaged structures, and resulted in many potentially serious accidents. Subsequent fluctuations in the water table, especially after periods of heavy rainfall, have been directly linked to an increase in the frequency of sinkhole formation. Powell and La Moreaux (1969) report the development of many collapse sinks and ground subsidence responsible for distress to foundations and underground utilities in an area of limestone near Columbiana, Alabama. The ground subsidence and sinkholes are attributed to groundwater withdrawal from two municipal wells and subsequent fluctuations in the water

table level, in an area over the Wilsonville Fault zone. Newton, et. al. (1973) describe the development of over 150 sinkholes during the period 1950-1972 along the right-of-way of Interstate 459 near Greenwood, Alabama. Pumping from nearby municipalities and mines, coupled with a severe drought during the 1950s, induced many collapses in areas underlain by the Tuscumbia Limestone in the vicinity of several fault zones. Cessation of groundwater withdrawal in 1970 has allowed the slow recovery of the water table and was expected to result in a drastic decrease in sinkhole formation.

Alteration of Surface Drainage

Since the development of sinkholes from altered surface drainage is usually not as catastrophic as those induced by groundwater withdrawal, and their occurrence is more localized, few case studies are reported in the literature. However, sinkholes developed in this way are numerous. Perlow, et. al. (1984) has linked over 50 % of the sinkholes developed in the Allentown, Pennsylvania area to construction and leaky utilities. Williams and Vineyard (1976) link 74 % of man-induced sinkhole collapses in Missouri to altered hydrologic characteristics, other than dewatering. This contrasts with the studies by Newton (1976) in Alabama, where most collapses result from groundwater withdrawal, emphasizing the importance of the geologic and hydrologic settings.

Royster (1984) describes several case histories of subsidence and flooding caused by altered surface drainage at sites in Tennessee. In all of the cases reported, increased surface runoff from construction and/or urban development was responsible. In another study of collapses in Tennessee, Kemmerly (1984) reports that 40 % of the nearly 100 collapses investigated in the western Highland Rim area were man-induced, and two-thirds of these were triggered by changes in the surface drainage from residential and commercial development.

Mechanisms of Subsidence and Collapse

General mechanisms of sinkhole formation were discussed previously. Several scenarios are now described which emphasize the hydrologic aspects of subsidence, collapse, and flooding in karst terrain. The two general mechanisms discussed earlier are once again distinguished, since different treatment techniques are applied to each.

A decline in the water table leads to the formation and collapse of cavities in the soil as a result of (a) increased fluctuations in the groundwater level; (b) additional movement of surface water through the overburden into bedrock openings in areas where recharge had previously been rejected; (c) increased velocity of flowing water from increased hydraulic head; and (d) decreased support of the overlying soil (Newton, et. al., 1973). Figure 20 presents

Figure 20. Decline of Water Table from Pumping of Well in a Carbonate Terrain.

an idealized profile of the subsurface in a karst terrain. Solution cavities are extensively developed in the bedrock and several large voids are present in the soil over these openings. The free water table is at a level above the soil cavities, and a buoyant effect is imposed on the submerged portion of the overburden. A lowering of the water level may occur naturally over time, such as during a drought; can be seasonal, such as at the end of the dry season in late summer and early fall; or may be man-induced, such as withdrawal from water supply wells and for mines and quarries. As the water table decreases to a level below the soil-bedrock contact (Figure 20), the effective stress acting on the soil above the voids increases. Water draining into the subsurface through swallets and relict joints and bedding planes erodes soil into the subsurface channels where it is carried away by flowing groundwater. Heavy rainfall produces short-term increases in the groundwater level. As the water rises into the soil cavities, it erodes the soil arch from below, producing the ravelling effect described by Sowers (1978). The same cycle of alternate wetting and drying is repeated until subsidence and eventual collapse results. This scenario is most common when the phreatic surface fluctuates within the zone of soil-bedrock contact.

Surface drainage characteristics are altered by construction and other human activities. Problems associated with these changes include both (a) flooding resulting from

inadequate drainage capacity of sinkholes and the underground network, and (b) land subsidence and localized collapse resulting from piping of soils in the vicinity of the soil-bedrock contact. These problems begin as land development increases surface runoff. Sinkholes, which acted as drains for surface water, are often backfilled during construction. The increased runoff is funneled to other nearby sinks which are often unable to handle the flow during periods of heavy precipitation. Water is ponded near the surface and local flooding occurs. The increased hydraulic head accelerates underground erosion, and old collapse features which were filled during construction are reopened as a result of upward erosion.

Solutions and Construction Techniques Applied to Hydrologically-Induced Problems

In cases where groundwater withdrawal has produced a decline in the water table, prevention of subsidence and collapse is a difficult task. Location of wells away from development or in areas where geology is more favorable is the most obvious solution, but is often not a feasible alternative. Problems induced by alteration of surface drainage are somewhat easier to prevent. An effective measure is to construct impermeable drainage facilities for all new commercial, industrial, and residential development to provide safe disposal of surface runoff. However, treatment of individual features after a problem has developed is more

difficult. Several steps are necessary to ascertain the exact cause of the problem before preventive or corrective measures are designed, and are discussed below.

Evaluation of Hydrologic Data

Royster (1984) recommends that the following factors be considered when assessing hydrologically-related problems in a karst region: (a) geologic structure (joint orientation, direction and angle of dip, etc.); (b) depth, direction of flow, and slope of the groundwater table; (c) thickness and composition of the regolith; (d) degree of solution development and present level of activity; (e) relief and topographic expression; (f) size of area being drained by the sinkhole; (g) location of known or assumed points of discharge; (h) precipitation and flood levels; (i) well locations and level fluctuations; (j) location and size of swallets; and (k) potential for further land development.

For areas where subsidence, collapse, or flooding is occurring, an assessment of the cause of the problem begins with the collection of hydrologic data. Measurements¹ of water levels in existing wells and installation of observation wells are necessary. Magnitudes of groundwater level fluctuations are estimated. By combining these readings with depth to bedrock data, mechanisms of collapse can be postulated. Back flooding of sinkholes indicates inadequate capacity of the subsurface to absorb additional runoff. An

increase in the amount of fines in well water indicates underground erosion. Precipitation data are evaluated to correlate periods of increased subsidence or flooding with periods of heavy rainfall. Monitoring of surface and subsurface points to detect subsidence is recommended for sensitive structures.

In some cases, fluorescein dye is used to determine the direction of groundwater flow (Kemmerly, 1984). Such a method is recommended for evaluating drainage characteristics of large sinks and for areas where the direction of regional or local groundwater flow is difficult to ascertain from available well data. Where the problem is extensive and enough funds are available, a subsurface exploration program following the methodology described in Chapter V is recommended.

Corrective Procedures

Once the groundwater regime and subsurface profile are established and the cause of the particular problem tentatively identified, specific corrective or preventive measures are designed. Weber and Darnell (1984) list three principal design factors that must be satisfied in any remedial or preventive treatment system for sinkholes: (a) maintain satisfactory surface (and sometimes subsurface) drainage within the area; (b) provide structural stability adequate for the intended use of the area; and (c) minimize

the long-term risks of future ground subsidence or collapse. Examples of methods successfully applied in karst regions are described and address three common problems: (a) collapse or subsidence from groundwater table lowering; (b) collapse or subsidence from alteration of surface drainage; and (c) flooding as a result of alteration of surface drainage.

Methods Dealing with Groundwater Withdrawal

The prevention of groundwater withdrawal in areas underlain by carbonate rock is desirable if facilities sensitive to subsidence and ground collapse are present. Development should be restricted adjacent to limestone quarries and mines where deep pumping is required for dewatering. A hydrologic impact study of the area should be conducted whenever a new water supply well is installed whose pumping rate is significant (>100-500 gpm, depending upon risk to development nearby). Location of wells away from existing development is preferred, and other water supply alternatives should be explored if this is not possible and adverse effects to the groundwater regime are expected.

If continuation of groundwater pumping is necessary and relocation impossible, such as with quarries and mines, a plan for recharging the groundwater is recommended. Sources of water for recharge must meet environmental constraints with regard to temperature, dissolved solids, pH level, and

concentration of dissolved ions, in addition to other parameters. Consultation with the United States Environmental Protection Agency and the state department of conservation or other appropriate agency is required. Recharge sufficient to restore the groundwater level close to what it was prior to pumping should be introduced through wells discharging below the soil-bedrock contact. Introduction of water into collapse features at the surface will also provide sufficient recharge but may induce underground erosion and piping leading to further collapse in the vicinity. A thorough evaluation of this possibility is required.

In the case of dewatering operations for quarries, mines, or deep foundation excavations, recharging the groundwater regime in the vicinity of drawdown will, of course, increase the amount of pumping required. A classic example is described in Foote (1953) when groundwater recharge in the Hershey Valley required that the Annville Stone company increase their rate of pumping from 6500 gpm to 8000 gpm to maintain the water level below the bottom of the quarry. After litigation failed to prevent groundwater recharge, the stone company initiated a program of grouting the major solution channels in the vicinity of the mine. This reduced required pumping to 2500 gpm. Groundwater levels returned to normal throughout the valley within a year, and sinkhole formation ceased.

Extending this concept further, large cutoffs can be constructed to isolate areas where withdrawal is severe or subsurface flow must be prevented. A large concrete diaphragm wall was constructed to prevent flow through solution channels in the foundation of the Wolf Creek Dam in Kentucky (Simmons, 1981). Other methods of cutoff can be designed. It is imperative to know the location of the major openings through which the groundwater flows for a successful design.

Control of Surface Drainage

Altering the landscape produces changes in the hydrologic conditions, and can lead to (a) subsidence and collapse; and (b) flooding in karst regions. Specific methods for dealing with each are discussed below.

Subsidence and Collapse Problems

Underground piping and upward erosion within soil cavities from increased subsurface flow is the primary mechanism which must be prevented in any design dealing with sinkhole formation. Specific examples cited all deal with problems of collapse initiated by recharge introduced at points "upstream" on the regional groundwater slope. Underground erosion is enhanced as the water flows beneath the area where sinkholes are developing. The increased recharge may be a result of (a) increased surface runoff from land development; (b) filling of sinkholes which previously acted

as drains; (c) a broken utility line; or (d) opening of old swallets during grading or excavation.

A simple method of bridging over a sinkhole, while maintaining its drainage capability, is presented in Figure 21. A percolation system consisting of a base level of shot rock or crushed stone is placed in the bottom of the sinkhole. Excavation to stable material, either bedrock or a firm soil layer, is recommended. A filter system of progressively finer soil gradation is then constructed, with a compacted fill placed over this. It is obvious that such a system has a high risk with regard to future subsidence and instability. The technique should be considered only if no other economically feasible alternative is available; the facility is not very sensitive to subsidence, such as with small lightweight structures or parking areas; and the soil is shallow with bedrock exposed near the throat of the sinkhole (Weber and Darnell, 1984). Filter fabric placed within the fill and a properly designed rock base help to bridge over any collapse feature which develops.

Figure 22a presents a case of subsidence under S.R. 76 in Tennessee as described by Royster (1984). A sinkhole swallet located under the southbound lane was filled during construction of the highway, and drainage was directed into another swallet just east of the northbound lane. The regional groundwater table sloped in a westerly direction so that runoff from the highway emptied into the swallet and

Figure 21. Cross-Section Through Sinkhole Area Covered with Blanket Drain.
(From Weber and Darnell, 1984)

(a) Subsidence problem.

(b) Final solution.

▽ HIGH WATER TABLE	UNCONSOLIDATED MATERIAL
▼ NORMAL WATER TABLE	LIMESTONE
WATER-FILLED OPENING	← DIRECTION OF WATER FLOW
AIR-FILLED OPENING	SUBSIDENCE
QUARRY-RUN ROCK AND HIGH-SLUMP CONCRETE	CLAY FILL

Figure 22. Schematic Cross-Section of the Tennessee S.R. 76 Subsidence Problem and Solution. (From Royster, 1984)

then flowed back under the embankment. Backward erosion during localized highwater conditions under the highway after periods of intense rainfall produced subsidence over an old filled cavity beneath the southbound lane. After a subsurface exploration program consisting of borings and installation of standpipe piezometers, the cause was ascertained. The solution was to seal the upstream swallet and direct drainage under the roadway to discharge at a point downstream of the groundwater flow (Figure 22b).

Certainty of execution of the chosen design is only possible if the cause of the subsidence or collapse is known or can be predicted beforehand. Simple routing of drainage to the downstream side of the groundwater table with respect to the facility should be a routine procedure in karst areas. Sealing of swallets which could cause subsidence is recommended, as in this example, but the capacity of the downstream drainage basin and existing swallow holes must also be assessed.

Flooding Problems

Localized flooding as sinkholes fill with water during periods of heavy rainfall is treated by improving the subsurface drainage capacity. The installation of drainage wells that are drilled and cased into voids in the bedrock is the simplest technique available. Water which would normally collect and remain ponded at the surface, later to

percolate through and erode the soil residuum, is piped down into the bedrock. A pervious clay seal at the surface is required to direct surface water toward the wells and prevent infiltration. The obvious drawback to such a system is the detailed amount of exploration necessary to locate voids of sufficient size to absorb the recharge. Even in areas where cavity development is extensive, it is difficult to locate borings where they will intersect a large enough void. Environmental factors concerning recharge must also be considered, as discussed previously (Weber and Darnell, 1984).

Royster (1984) describes a case history of flooding along S.R. 12 near Clarksville, Tennessee. During periods of heavy rainfall (>3-4 in. over a day or so), a large sinkhole adjacent to the highway and a developed area filled with water and flooded the surrounding area. A hydrologic assessment of the area was made, and several alternative solutions were considered. It is interesting to note that the sinkhole formed over a limestone pinnacle (Figure 23), where swallets and tubes developed from piping action along the soil-rock contact. To assess the drainage capability of the sink, several 2000 gallon tanks of water were pumped into 5 in. pvc casings inserted through the residuum and set into the bedrock. No rise in water level in any of the piezometers installed at the site was observed, and no lag in flow rate through the casings was noted. This reinforced

Figure 23. Schematic Cross-Section of Sump Design for the Tennessee S.R. 12 Sinkhole. (From Royster, 1984)

the notion that the drainage capability of the sinkhole could be improved. The most economical alternative involved excavating the sinkhole to rock, installing 3 to 4 ft diameter perforated standpipes, and backfilling with drainage stone. A schematic of this design is presented in Figure 23. Other features include filter fabric inserted between the porous stone and the natural soil, grouted rip-rap ditches directed toward the sumps, and siltation fences installed around the sinkhole. Suggested gradation of the drainage stone is presented in Table 9.

Table 9. Suggested Gradation Characteristics of Drainage Stone. (From Royster, 1984)

Size of Stone (Maximum Dimension)	Composition by Weight
1.0 - 2.0 ft	40 - 60 %
0.5 - 1.0 ft	20 - 40 %
2.0 - 6.0 in.	10 - 20 %
minus 2.0 in.	0 - 5 %

The cost of the project was \$60,000, compared to an estimated \$300,000 for construction of a storm sewer to carry the runoff to a nearby creek. Other alternatives were considered, including construction of holding ponds and raising the grade. A cost-benefit analysis as well as evaluating the certainty of execution is required before an appropriate solution is chosen.

The Indiana Department of Highways has a similar design for a sinkhole used for drainage, as illustrated in Figure 24. However, a precast reinforced concrete chimney is

Figure 24. Simplified Cross-Section of a Sinkhole Used For Drainage.
(From Strate, 1972)

installed in place of the perforated casings. A concrete mat is often required if the bedrock is unstable or openings are large. Granular fill is placed around the chimney, but native material is used to fill the rest of the excavation. Care must be taken to avoid piping along the outside of the chimney drain. Installation of collars at intervals of several feet along the structure should prevent this.

Location of Wells in Karst Regions

Often, problems of subsidence, collapse, or flooding are not anticipated or are of secondary importance compared to the problem of locating wells which will provide a sufficient water supply. Primary permeability (interparticle void spaces) is usually very low compared to secondary permeability (along structural defects such as joints and bedding planes) in carbonate rock. Therefore, wells must be located near fracture zones where groundwater flow is concentrated. Intersecting vertical solution zones is difficult. Bedding planes are more often encountered, and are sometimes better sources of groundwater supply (Figure 25a). Fault zones and fracture intersections often produce the highest yields (Figure 25b). Fractures created by tectonic forces and subsequently enlarged by solutioning, such as near the crest of an anticline, also provide good locations for wells (Figure 25c). Careful consideration of exploration data and geologic and hydrologic conditions, should be combined with pumping test data to assess the expected well

(a) Secondary permeability zones.

(b) Fracture intersections and fault zones.

(c) Fractures in crest of anticline.

Figure 25. Occurrence of High Permeability Zones for Well Location in Carbonate Rock. (After Freeze and Cherry, 1979)

yield, extent of the resulting cone of depression, and subsequent land subsidence and collapse.

Conclusion

The hydrologic and geologic factors which must be considered when evaluating potential or already occurring problems in karst regions are listed in Table 10. A suggested procedure for assessing the cause of the problem and choosing an appropriate technique is included. The techniques presented here are merely examples which were successful for a particular geologic and hydrologic setting. Ingenuity is required to properly design a remedial or preventive system based on the factors presented above. These examples should be used only as a guide, since solutions must be tailored to the particular problem.

Table 10. Procedure for Evaluating Hydrologic Aspects of Problems Encountered in Karst Regions.

<p>1. <u>Identify problem:</u></p> <ul style="list-style-type: none"> (a) Land subsidence (b) Sinkhole formation (c) Flooding <p>2. <u>Collect Geologic and Hydrologic Data on the Region:</u></p> <ul style="list-style-type: none"> (a) Geologic structure (joint orientation, etc.) (b) Depth, direction of flow and slope of gw table (c) Thickness and composition of the regolith (d) Degree of solution development and present activity (e) Relief and topographic expression (f) Size of area being drained by sinkhole (g) Location of known or assumed points of discharge (h) Precipitation and flood levels (i) Well locations and level fluctuations (j) Location and size of swallets (k) Potential for further land development <p>3. <u>Check for Problem Indicators:</u></p> <ul style="list-style-type: none"> (a) Local ponding of water and sinkhole flooding after periods of heavy precipitation (b) Turbidity (presence of fines) in well water (c) Correlations between pumping and precipitation data and problem occurrence <p>4. <u>Assess Cause(s) of Problem(s):</u></p> <ul style="list-style-type: none"> (a) Groundwater withdrawal (b) Alteration of surface drainage characteristics from: <ul style="list-style-type: none"> - Land development - filling of old sinkholes - a broken utility line - opening of old swallets during grading or excavation 	<p>5. <u>Design Preventive or Remedial Treatment:</u></p> <ul style="list-style-type: none"> (a) Maintain satisfactory surface (and sometimes sub-surface) drainage within the area (b) Provide structural stability adequate for the intended use of the area (c) Minimize the long-term risks of future ground subsidence or collapse (d) Groundwater withdrawal problems: <ul style="list-style-type: none"> - cease pumping operations - locate wells away from developed areas - recharge groundwater regime - grout underground solution channels in vicinity of pumping (e) Control of surface drainage: <ul style="list-style-type: none"> - design other appropriate cutoff - design impermeable drainage facilities - bridge over existing sinkholes while maintaining their drainage capacity - design drainage system to discharge downstream on groundwater table with respect to facility - provide drainage wells to carry surface runoff directly to underlying bedrock - improve drainage characteristics of sinkhole <p>6. <u>Monitor Performance of Chosen Solution:</u></p> <ul style="list-style-type: none"> (a) Establish telescopic benchmarks on sensitive structures (b) Install settlement indicators at points on surface and subsurface (c) Continue monitoring piezometers to assess fluctuations or new changes in the groundwater regime
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ENGINEERING DESIGN AND CONSTRUCTION IN KARST REGIONS

The previous section presented methods for stabilizing collapse features that develop over cavities in carbonate bedrock as a result of hydrologically-induced stresses. The importance of obtaining sufficient geologic and hydrologic data was emphasized. However, general design of structures such as highway embankments requires a more detailed assessment of many engineering parameters. The following sections provide an outline and discussion of these parameters as related to design and construction in karst regions. Geotechnical properties of carbonate rock and the residual soil derived from it are included. A classification system relating index properties to general field behavior is provided. Some simple analysis techniques for evaluating strength and deformation characteristics are developed. Finally, state-of-the-art construction methods applied specifically to sinkhole repair and highway embankments are described. Routine design procedures are not included; only those techniques applicable to special problems in karst terrain are considered.

Engineering Classification of Carbonate Materials

The first step in any geotechnical engineering design is to classify the constituent materials. Various classification systems for carbonate rock, the residual soil, and the weathered subsurface profile have been proposed. Dearman (1981) recommends that the following characteristics be included in any rock description:

1. Rock Name
 - (a) Supplementary petrographic properties
2. Rock Material Properties
 - (a) Color
 - (b) Texture
 - (c) State of weathering
 - (d) Strength
3. Rock Mass Properties
 - (a) Structure
 - (b) Discontinuities
 - (c) Weathering profile

Figure 26 presents a simplified engineering geological classification system for carbonate rock based on chemical composition and grain size. Such a scheme enables the engineer to define the rock under consideration on a consistent basis. Certain physical qualities and general behavior are implied from the rock name.

A more meaningful engineering index takes into account the strength and deformation characteristics of the material. In rock mechanics, these properties are represented by the unconfined compressive strength, q_u , and the elastic modulus, E , for strength and deformation, respectively. Such a system was proposed by Deere and Miller (1966), and is presented in Figure 27. Limestones

Percentage Carbonate		0	10	50	90	100
		Limestone				
Predominant Grain Size (mm)	2	Conglomerate	Calcareous Conglomerate	Gravelly Limestone	Calcirudite	Limestone
		Sandstone	Calcareous Sandstone	Sandy Limestone	Calcar-enite	
	0.06	Mudstone	Calcareous Siltstone	Silty Limestone	Calci-siltite	
	0.002		Claystone	Calcareous Claystone	Clayey Limestone	Calcilutite
		Marlstone				

Note: Non-carbonate constituents are rock fragments or quartz, micas, clay minerals; predominant grain size implies over 50% by weight.

Calcite		0	10	50	90	100%
		Dolomite	Calcitic Dolomite	Dolomitic Limestone	Limestone	
Dolomite		100	90	50	10	0%

Figure 26. Classification of Limestones, Limestone-Dolomite Mixtures, and Calcareous Sedimentary Rocks. (From Dearman, 1981)

LEGEND

- ES Extremely strong
- VS Very strong
- S Strong
- MS Moderately strong
- W Weak

*From Peck, et. al. (1974)

Figure 27. A General Engineering Classification of Carbonate Rocks Based on Elastic Modulus and Unconfined Compressive Strength. (After Dearman, 1981)

are generally in the strong to very strong range. A value for the Bedford Limestone, as reported in Peck, et. al. (1974) is plotted for reference.

A qualitative classification of the degree of weathering of the subsurface profile is possible after assessing subsurface data obtained from the site investigation. Such a scheme is presented in Figure 28. The classes range from Grade I for intact rock to Grade VI for completely weathered residual soil. The classification gives an indication of the present solution activity, condition of the rock and soil, and the general degree of weathering of the profile. An exact profile is difficult to obtain, so the index is meant only as a guide for identifying zones of potential instability. It is most applicable for distinguishing weathering zones identified from road cuts and test pits.

General Foundation and Stability Requirements

Before analysis or design of the structure is undertaken, all possible failure mechanisms must be considered. Only then can the appropriate model and analysis technique be chosen. Common foundation failure modes in carbonate rock (Dearman, 1981; and Sowers, 1976) are described, and a brief discussion of allowable bearing pressure is presented below.

Figure 28. Typical Weathering Profile and Classification Scheme for Carbonate Rocks. (From Dearman, 1981)

Foundation Failure Modes

Several modes of foundation failure in carbonate rock are possible, depending on the degree of weathering, and some of the common types are illustrated in Figure 29. For weak, relatively homogeneous rock (Figure 29a), failure occurs in general shear, and bearing capacity is calculated the same as for soil. Local shear failure occurs in soft, compressible rocks such as chalk (Figure 29b), and recommended values of bearing capacity are two-thirds of those for general shear. If the beds are horizontal and the top layer is a thin, relatively rigid rock overlying a plastic stratum, the foundation may fail in tension as in a beam (Figure 29c). Cases of this failure mode have been documented in southern Florida in the Miami Limestone (Kaderabek and Reynolds, 1981). A punching shear failure may result in a thin rigid rock stratum overlying a compressible stratum of porous or highly weathered material (Figure 29d).

Bearing capacity failures in strong carbonate rocks are greatly influenced by the discontinuities and solution features present. Theoretical shear failure in a rock with closely spaced, tight joints (Figure 29e) is conceivable, but does not seem plausible from a physical standpoint. Failure of individual blocks in unconfined compression between closely spaced open joints is illustrated in Figure 29f; while tensile splitting between widely spaced joints is shown in Figure 29g. Where solutioning has reduced the

Figure 29. Foundation Failure Modes on Carbonate Rocks.
(From Dearman, 1981; after Sowers, 1976)

bedrock to a highly pinnacled surface (Figure 29h), the bearing capacity is governed by the strength of the individual pinnacles under eccentric loading.

Allowable Bearing Pressure

Foundation shear failures in competent rock are rare, and bearing capacity is usually not the limiting factor in design. Allowable bearing pressure on rock is governed by the deformability of the rock mass rather than the actual strength of the material, unless the strength of the intact rock is equal to or less than that of normal concrete. Rock quality designation (RQD) is an index used to relate the number and spacing of discontinuities to compressibility. Table 11 presents correlations of RQD with allowable bearing pressure, q_a , on unweathered rock. If these recommended values are not exceeded, expected settlement should be less than 0.5 in., even for large loaded areas (Peck, et. al., 1974). Many other values of q_a are specified in various building codes (D'Appolonia, et. al., 1975; and Peck, et. al., 1974), and are presented in Table 12.

Slopes and Underground Excavations

Design of slopes and underground excavations in carbonate rock must take into account two types of failure mechanisms: (a) failure of a rigid rock mass along structural defects such as joints, faults, and bedding planes; and (b) shear failure within an intact rock mass or rock

Table 11. Allowable Contact Pressure on Carbonate Rock
Based on RQD. (From Peck, et. al., 1974)

Rock Quality Designation (RQD)*	Allowable Contact Pressure, q_a ** tsf (psi)
100	300 (4170)
90	200 (2780)
75	120 (1660)
50	65 (970)
25	30 (410)
0	10 (140)

$$* \text{RQD} = \frac{\text{Length of intact rock pieces greater than 4 in.}}{\text{Total length of rock core}} \times 100 \%$$

** If q_a exceeds the unconfined compressive strength of the intact rock, q_u , then $q_a = q_u$.

Table 12. Allowable Contact Pressure on Carbonate Rock from Various Building Codes. (After D'Appolonia, et. al., 1975; and Peck, et. al., 1974)

Bedrock Type	q_a (tsf)				
	Building Code *				
	A	B	C	D	E
Massive Crystalline Bedrock	100	100	$0.2q_u$	$0.2q_u$	100
Sound Sedimentary Rock	25	15	$0.2q_u$	$0.2q_u$	15
Soft or Broken Sedimentary Rock	10	--	$0.2q_u$	$0.2q_u$	10

* Building Codes: A = BOCA (1970)
 B = National Building Code (1967)
 C = Uniform Building Code (1970)
 D = Indiana Building Code (1967)
 E = Ohio Building Code (1970)

mass with numerous small discontinuities. In addition, the irregularity of the soil-rock contact must be considered, especially in rock cuts for highways. The following sections identify some of these problems and recommend procedures for analyzing slopes and underground excavations. Some general analysis techniques are provided as a design guide, but the final choice of the procedure is left up to the experience and judgment of the engineer. Every project is unique and no one procedure can be used in all cases.

Qualitative Assessment

Highway cuts and excavations in carbonate rock present difficult problems because of the erratic nature of the subsurface. Figure 30 illustrates a hypothetical road cut through weathered limestone bedrock with several developed karst features. Point (a) represents an old filled sinkhole with a perched water table which, upon excavation, may collapse because of the low shear strength of the soil and the unstable nature of the feature. At point (b) is an old collapse sink, which usually does not present an instability problem since its presence is detectable during excavation, and the rock fragments give additional strength to the clay matrix. Point (c) illustrates an incipient sinkhole, which is the most unstable of the three features. The soft clay residuum has a low shear strength and localized slope failures are common. Changes from rock to soil are frequent and often unpredictable, and present a difficult challenge

Figure 30. Weathering Features Influencing the Stability of Road Cuts in Carbonate Rocks. (From Dearman, 1981)

because excavation techniques, as well as slope design, are different for each material (Dearman, 1981).

The stability of subsurface cavities in carbonate rock depends upon many factors, including the degree of weathering, joint thickness, spacing, and orientation in the rock roof, and the stress imposed by any additional load, such as from an embankment. If an underground cavern detected during the subsurface investigation is large and accessible at the surface, the stability can be ascertained by inspection. Jingmin and Zhizhong (1982) have developed a qualitative stability index for inspection of karst caverns, which is presented in Table 13. Their work concerns the use of caverns for storage of petroleum and gas, treatment of waste material, and for other purposes in the People's Republic of China. While karst caverns in southern Indiana are not employed in this way at present, the possibility is not ruled out in the future. In addition, where large cavities are known to underlie foundation sites and are accessible at the surface, underground inspection which considers the factors outlined in Table 13 is recommended for a preliminary stability assessment of the area.

Design Guidelines

Design of slopes and underground openings in jointed carbonate rock requires analysis of several failure modes. The first is the block or wedge analysis, in which a rock

Table 13. Qualitative Stability Index of Karst Caverns. (From Jingmin and Zhizhong, 1982)

CONDITIONS	STABLE	BASICALLY STABLE	LESS STABLE	UNSTABLE
Geologic Structure	No fault and fold, only slightly fissured; filled and cemented very well; no unstable block exposed.	With a few faults and/or folds; but more fissures, filled and cemented well; a few unstable blocks exposed.	With more faults and/or folds and fissures; a few fissures are open; filled though cemented poorly; some filled by water; many unstable blocks exposed.	With major faults and/or heavily folded; highly fissured and open; unfilled and cemented poorly; some filled by water; a great many unstable blocks exposed.
Rock Properties	Very thick or thick inclined strata or beds; the continuity of rock masses is perfect; strikes intersect with tunnel axis.	Thick or medium inclined strata or beds; fairly continuous; strikes intersect with tunnel axis.	Medium and thin, broken, gently inclined strata or beds strike parallel to tunnel axis.	Thin or medium, nearly horizontal strata or beds with intercalated shale layer; highly broken; strikes parallel to tunnel axis.
Groundwater, Karst and Underground River	No water dripping from roofs; no karst water on bases and no underground river nearby; only a few karst caves.	Little water dripping from roofs; little karst water appears on bases; many karst caves; no underground river.	Fairly more water dripping from roofs; karst water appears on bases, many karst caves and modern or ancient underground rivers, but easy to treat.	Water heavily dripping from roofs, a good many karst caves; much more karst water and modern underground river, but difficult to treat.
State of the Roof	Covered with calcium shells; no fresh mark of collapse; rock mass interlocks with good arching action in the roof.	Covered with calcium shells over two-thirds of the roof; no fresh mark of collapse; rock mass interlocks with good arching action in the roof.	Covered with calcium shells about one half of the roofs, and some fresh mark of collapse observed in rock mass isolated points; interlocks with poor arching action.	Covered with calcium shells only on partial roof; more fresh surfaces of collapse; no arching action in the rock mass.

Table 13. Continued

CONDITIONS	STABLE	BASICALLY STABLE	LESS STABLE	UNSTABLE
Rockfall	Only a few rockfalls observed, but no unstable rockfalls at present.	A number of rockfalls observed, but no unstable rockfalls at present.	A good number of rockfalls observed and rockfalls may occur at isolated points.	A great number of rockfalls observed and large rockfalls may occur.
Treatment Before Use	May be used after a slight treatment.	May be used after a certain treatment.	May be used after a greater treatment.	Treatment is very difficult, must not be used.

mass is assumed to slide along a discontinuity such as a joint or bedding plane. The factor of safety, FS, is expressed as the ratio of the sum of the resisting forces, f_r , over the sum of the sliding forces, f_s , as determined from a free body diagram of the rock mass. The value of f_r is a function of the normal force, N , the angle of friction, ϕ , and the cohesion, c , along the sliding plane; while f_s is dependent upon the weight of the block, W , and the angle of the sliding surface, θ . FS is expressed in equation form:

$$FS = \frac{f(N, \phi, c)}{g(W, \theta)} \quad (3)$$

Various failure modes and shapes of the block or wedge should be investigated using this simplified friction block approach, as illustrated in Hoek and Bray (1977).

For rocks that are relatively free of discontinuities or have many small fractures, a more sophisticated approach is needed. Finite element analyses are most often used to determine the in-situ stresses in the rock slope or around the excavation, tunnel, or natural cavity (Naylor, et. al, 1981; Goodman, 1977; and Wilson, 1977). Stress contours are plotted to determine zones of potential instability. A simplified straight line Mohr-Coulomb failure criterion is often used as shown in Figure 31. Where the critical failure plane occurs in the intact portion of the rock, the following relationship is used (Figure 31a):

$$\sin \phi > \frac{\sigma_1 - \sigma_3}{2c \cot \phi + \sigma_1 + \sigma_3} \quad (4)$$

(a) Strength criterion for $\phi_1 > \beta$.

(b) Strength criterion for $\phi_1 < \beta < \pi/2$.

Figure 31. Mohr-Coulomb Failure Envelopes for Failure (a) in Intact Rock and (b) along Discontinuity.

where

σ_1 = major principal stress

σ_3 = minor principal stress

ϕ = angle of internal friction of rock

c = cohesion intercept of rock

When failure occurs along discontinuities in the rock, the following criterion applies (Figure 31b):

$$\tan \phi_1 > \frac{(\sigma_1 - \sigma_3) \sin 2\beta}{2c_1 \cot \phi_1 + (\sigma_1 - \sigma_3) \cos 2\beta + \sigma_1 + \sigma_3} \quad (5)$$

where

β = angle between discontinuity and σ_3

ϕ_1 = angle of friction along discontinuity

c_1 = cohesion along discontinuity

Failure occurs if the left hand side of either Equation (4) or Equation (5) is exceeded. Whether the critical failure plane occurs within the rock or along a discontinuity is determined by

$$\phi_1 < \beta < \pi/2 \quad (6)$$

For $\beta > \phi_1$, Equation (5) is used; otherwise Equation (4) applies (Jingmin and Zhizhong, 1982).

Recent advances in strength criteria evaluation use semi-empirical non-linear relationships for the failure envelope. Hoek and Brown (1980a) have developed a relationship between the major and minor principal stresses at failure using two parameters which are dependent on the structural condition of the rock mass:

$$\frac{\sigma_1}{\sigma_c} = \frac{\sigma_3}{\sigma_c} + m \frac{\sigma_3}{\sigma_c} + s \quad (7)$$

where

σ_1 = major principal stress at failure

σ_3 = minor principal stress

σ_c = uniaxial compressive strength of the intact
rock

m, s = empirical constants that depend on rock
properties and discontinuities

Replacing σ_1/σ_c and σ_3/σ_c with σ_{1n} and σ_{3n} , respectively,
Equation (7) becomes

$$\sigma_{1n} = \sigma_{3n} + m \sigma_{3n} + s \quad (8)$$

The value of s ranges from 1.0 for intact rock to zero for a
completely granular material. The parameter m is dependent
upon rock type, interparticle or interblock friction, and
the degree of particle interlocking within the rock mass.
The failure envelope can also be represented by the shear
stress, τ_n , and the normalized applied stress, $\sigma_n = \sigma/\sigma_c$:

$$\tau_n = A(\sigma_n - \sigma_{tn})^B \quad (9)$$

where σ_{tn} is the normalized tensile strength of the rock
given as

$$\sigma_{tn} = \frac{\sigma_t}{\sigma_c} = \frac{1}{2} m - m^2 + 4s \quad (10)$$

and A and B are empirical constants dependent upon m . The
relationship of Equation (9) is illustrated in Figure 32a.

(a) Shear strength envelope.

(b) Variation of strength parameters ϕ_i and c_i with normal stress σ_n .

Figure 32. Shear Strength Envelope and Variation of Strength Parameters with Normal Stress for Hoek and Brown (1980a) Strength Criteria.

Hoek and Brown (1980a) collected data from many sources for various rock types with different degrees of weathering. Rock types were quantified by the rock mass quality schemes developed by the Norwegian Geotechnical Institute (NGI) (Barton, et. al., 1974) and the South African Council for Scientific and Industrial Research (CSIR) (Bieniawski, 1976). These rating systems and examples of their use are presented in Appendix C. Regression analyses were performed on available triaxial compression test data for oven and air-dried samples, and the parameters m , s , A , and B of Equations (8) and (9) were evaluated. The relationships obtained for carbonate rock of various degrees of rock quality are listed in Table 14.

These empirical relationships can be used in standard slope stability analyses for circular and non-circular slips such as Janbu's or Bishop's methods. It is more convenient for the analysis to be performed using the shear strength parameters ϕ and c , and instantaneous values are obtained for a given σ_n using the assumed constants m , s , A , and B from Table 14 by the relationships:

$$\tan \phi_i = AB \sigma_n^{B-1} - \sigma_{tn} \quad (11)$$

and

$$c_i = \tau - \sigma \tan \phi_i \quad (12)$$

where

ϕ_i = instantaneous value of angle of internal friction

Table 14. Approximate Strength Criteria for Intact and Jointed Carbonate Rock. (From Hoek and Brown, 1980a)

ROCK QUALITY	STRENGTH CRITERION RELATIONSHIP
<p>INTACT ROCK SAMPLES</p> <p>Laboratory size rock specimens free from structural defects</p> <p>CSIR rating 100+, NGI rating 500</p>	$\sigma_{1n} = \sigma_{3n} + \sqrt{7\sigma_{3n} + 1.0}$ $\tau_n = 0.816 (\sigma_n + 0.140)^{0.658}$
<p>VERY GOOD QUALITY ROCK MASS</p> <p>Tightly interlocking undisturbed rock with unweathered joints spaced at >3 metres</p> <p>CSIR rating 85, NGI rating 100</p>	$\sigma_{1n} = \sigma_{3n} + \sqrt{3.5\sigma_{3n} + 0.1}$ $\tau_n = 0.651 (\sigma_n + 0.028)^{0.679}$
<p>GOOD QUALITY ROCK MASS</p> <p>Fresh to slightly weathered rock, slightly disturbed with joints spaced 1 to 3 metres</p> <p>CSIR rating 65, NGI rating 10</p>	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.7\sigma_{3n} + 0.004}$ $\tau_n = 0.369 (\sigma_n + 0.006)^{0.669}$
<p>FAIR QUALITY ROCK MASS</p> <p>Several sets of moderately weath- ered joints spaced at 0.3 to 1 metre.</p> <p>CSIR rating 44, NGI rating 1.0</p>	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.14\sigma_{3n} + 0.0001}$ $\tau_n = 0.198 (\sigma_n + 0.0007)^{0.662}$
<p>POOR QUALITY ROCK MASS</p> <p>Numerous weathered joints spaced at 30 to 500 mm with some gouge filling/clean waste rock</p> <p>CSIR rating 23, NGI rating 0.1</p>	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.04\sigma_{3n} + 0.00001}$ $\tau_n = 0.115 (\sigma_n + 0.0002)^{0.646}$
<p>VERY POOR QUALITY ROCK MASS</p> <p>Numerous heavily weathered joints spaced less than 50 mm with gouge filling/waste rock with fines</p> <p>CSIR rating 3, NGI rating 0.01</p>	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.007\sigma_{3n} + 0}$ $\tau_n = 0.042 (\sigma_n)^{0.534}$

c_i = instantaneous value of cohesion

σ = imposed value of stress

τ and σ_{tn} are evaluated from Equations (9) and (10), respectively. A plot of ϕ_i and c_i versus σ_n is shown in Figure 32b. This analysis requires only the unconfined compressive strength of the intact rock, σ_c , used as a scaling factor, and an assessment of the quality of the rock mass for selection of the appropriate relationship from Table 14.

Stress analysis of underground excavations can be performed using finite element or other techniques (Hoek and Brown, 1980b), and zones of overstepping are determined using the principal stress equations of Table 14. The method outlined here should be used only for preliminary design calculations since it is based on a limited amount of data. It should not be used where failure occurs by slip along one or two discontinuities, as in a block or wedge slope failure. However, the technique is extremely useful for assessing overall rock strength and stability in the early stages of design before more extensive lab tests, field tests, and trial excavations are made (Hoek and Brown, 1980a). Additional laboratory techniques for rock mechanics are discussed in American Society for Testing and Materials (1966).

In-situ measurements are preferred for any large design in carbonate rock because of the non-homogeneity of the

material. Many types of in-situ tests are currently employed in practice, and a description of them is beyond the scope of this report. Tests are very site specific and are designed accordingly. Good discussions of in-situ testing are given by Hoek and Bray (1977) and Goodman (1981).

Construction Methods

Solution features require treatment with unique construction techniques which are both costly and time consuming. Each problem must be treated with a method tailored to its unique characteristics. Design is basically an art and often relies on qualitative, rather than quantitative factors. However, guidelines are necessary and the development of rational design methods based on quantitative parameters is warranted. The following section describes current foundation schemes for karst regions. Two relatively new techniques for bridging over sinkholes and solution cavities are described, and a design procedure for one of them is developed in detail. In addition, a general outline for sinkhole repair is provided.

General Foundation Schemes

The design of foundations in karst regions is very site specific. Couch (1984) lists several points which must be included in any foundation design program: (a) a geologic assessment; (b) a site specific subsurface investigation; (c) a siting study; (d) an evaluation of the sensitivity of

the structure to deformation; (e) a choice of several technically feasible alternatives; and (f) a study to evaluate the risk versus cost for each alternative. Points (a) to (c) have already been discussed; point (d) is left up to the judgment of the structural and geotechnical engineer; and point (f) is usually left up to the owner to decide, based on the information provided by the engineer. Some techniques to be considered under point (e) are discussed below.

Figure 33 presents four foundation design alternatives for karst regions (Partridge, et. al., 1981). Method I consists of a concrete mat foundation which is supported directly on bedrock pinnacles. The bedrock highs must be sound, numerous, and at about the same level. It is only economical if the excavation depth is less than 10 ft.

Method II employs a concrete raft supported on piers that are socketed into bedrock. Couch (1984) notes several cases in which this type of foundation was used in Tennessee, Alabama, and Virginia. The location of pinnacles must be firmly established before construction. Preboring at each pier location provides data on depth, helps in estimating cost, and aids in controlling construction. Shafts about 3 ft in diameter are drilled to the top of rock. Proof drilling 5 or 10 ft into the rock is required to assure that it is sound and no voids exist below founding level. The pier should be firmly socketed into bedrock and located away from highly sloping rock. Other problems of

Figure 33. Alternative Methods of Founding on Carbonate Rock.
(From Partridge, et. al., 1981)

pier construction include blow-in of the soil bottom and subsidence triggered by dewatering; resistance from small rock pinnacles above founding level; and difficulty in excavating rock underwater in a confined area. Driven piles present additional problems since estimation of length and sufficient end bearing is very uncertain in irregularly weathered rock (Sowers, 1976). For this reason, driven piles are not recommended for use in carbonate residual soils unless the bedrock surface is level and solution activity is low. Method II is recommended only for tall heavy structures sensitive to differential settlement (Partridge, et. al., 1981). It is most applicable where the residuum is 10 to 20 ft thick.

Methods III and IV rely on a soil arch in the overburden to bridge over existing or future cavities. Method III is used where the residual soil is shallow (10 to 20 ft). Soil strengthening is required because the overburden is not thick enough to provide a sufficient span in case of cavity development. Method IV relies on a "stable" soil cover of at least 50 ft to provide a natural raft or to bridge over voids in the bedrock, and is based on empirical observation. It was commonly used in South Africa until the 1962 collapse of the crusher plant at the West Driefontein Mine, which was constructed according to this principle (Partridge, et. al., 1981). Method IV is not recommended unless the soil cover is very hard, approaching the consistency of rock, and then

only if the underlying bedrock is sound. The structure should be lightweight, runoff must be carefully controlled, and lowering of the groundwater table should be prevented.

It must be kept in mind that not all solution features are detected prior to construction, no matter how thorough the site investigation. The design must be flexible enough so that unanticipated problems uncovered during construction can be handled with a minimum of delay and additional cost. Couch (1984) cites a project in Nashville, Tennessee in which a multi-story building was constructed over highly solutioned Ordovician limestones. Excavation for the rock-supported spread footings revealed a large sinkhole which had developed at the intersection of two joints under one corner of the building. The structure was redesigned so that the corner was cantilevered out over the sink, and costly repair of the feature itself was avoided. Ingenuity is often a prerequisite for a successful foundation design in karst regions.

Compacted Soil Fill

Design and construction of a compacted fill over weathered carbonate rock requires estimation of the minimum thickness necessary to span a cavity in bedrock. A brief overview of theories of arching in soil is needed to develop an adequate design analysis.

Arching Theory

Terzaghi (1943) develops a theory of arching in ideal granular soil based on a transfer of pressure from a yielding mass of soil to the adjacent stationary soil through shearing resistance. It is no less permanent than the shear resistance beneath a footing or in a soil slope.

A simplified theory similar to Terzaghi's concept can be developed for cohesive soils. Figure 34 illustrates a subsurface profile of a void developing in the residual soil above a solution-enlarged cavity in the bedrock. A soil block above the arched zone is supported by the vertical shearing resistance of the adjacent soil. The actual sliding surface is curved so that the block is somewhat larger than $2B$ at the surface. However, vertical sides are assumed for simplicity. The horizontal stresses are assumed to be of the active mode, since erosion of the arch has resulted in a release of lateral pressure. A simple Mohr-Coulomb strength criteria is chosen:

$$s = c' + \sigma_h' \tan \phi' \quad (13)$$

where

s = soil shear strength in terms of effective stress

c' = cohesion intercept of soil in terms of effective stress

σ_h' = effective horizontal stress

ϕ' = effective angle of internal friction of soil

Figure 34. Simplified Analysis of Arching Effect in Soil above a Developing Cavity in Limestone Bedrock.

Assuming fully mobilized active earth pressure along the sides of the soil block, the strength, S , is

$$S = 2[c'(h+h_w) + (\frac{1}{2}K_a \gamma_m h^2 + K_a \gamma_m h h_w + \frac{1}{2}(K_a \gamma_m - \gamma_w) h_w^2) \tan \phi'] \quad (14)$$

where

γ_m = moist density of soil

γ_w = unit weight of water

h, h_w, B = defined in Figure 34

K_a = coefficient of active earth pressure

The weight of the block (assumed rectangular for simplicity) is

$$W = 2B[h\gamma_m + h_w(\gamma_m - \gamma_w)] \quad (15)$$

Equating S and W , and rearranging to solve for B yields

$$B = \frac{c'(h+h_w) + (\frac{1}{2}K_a \gamma_m h^2 + K_a \gamma_m h h_w + \frac{1}{2}(K_a \gamma_m - \gamma_w) h_w^2) \tan \phi'}{h\gamma_m + h_w(\gamma_m - \gamma_w)} \quad (16)$$

If the maximum cavity or joint opening in the bedrock is known from the subsurface investigation, this simple analysis technique can be used to roughly assess the potential for collapse under the present subsurface geologic and hydrologic conditions. However, it is difficult to evaluate the ϕ' and c' parameters in terms of effective stresses and h and h_w may vary considerably in the profile. The worst conditions should be assumed and a conservative design chosen.

The theory is not meant to be rigorous because of the many simplifying assumptions, and especially since it is

two-dimensional. However, it does provide the basis for development of a design method for constructing a soil fill to bridge solution cavities. Such a design method is virtually non-existent. If the assumptions and limitations are kept in mind, this approach can be a useful tool for designing a compacted soil fill for embankment foundations in the limestone residual soil of southern Indiana.

Design Method

A technique for analyzing a compacted clay fill constructed over solutioned carbonate bedrock has been developed by Partridge, et. al. (1981). It uses the same basic theory as outlined above, except an additional residual stress is assumed to exist in the soil as a result of compaction, and the presence of a water table is neglected. The principle is similar to that of Method III in Figure 33, but the soil-cement mix is replaced with a highly compacted clay fill. Design is based on an analysis of the dimensions of the maximum span that the fill can maintain over a cavity in the soil. The critical parameters are the height of the arch at the crown, H_c , and the span of the arch, $2B$, as defined in Figure 35.

The diagram in Figure 35a illustrates a hypothetical soil arch which has developed in the soil between two bedrock pinnacles. Equating the horizontal forces as analyzed from a free body diagram of a half arch (Figure

Figure 35. Simplified Theory of Two-Dimensional Compacted Soil Arch. (After Partridge, et. al., 1981)

35b) yields the following equation:

$$N_c H_c + \frac{1}{2} K \gamma H_c^2 = N_a H_a + \frac{1}{2} K \gamma H_a^2 \quad (17)$$

where

N_c = lateral stress above crown of arch

N_a = residual lateral stress due to compaction at
abutments

K = lateral earth pressure coefficient of soil

γ = density of soil

H_c, H_a = defined in Figure 35a

The soil at point O is in unconfined compression since the vertical stress, σ_v , is zero. The horizontal stress, q_h , at point O is

$$q_h = N_c + K \gamma H_c \quad (18)$$

From the Mohr's circle (Figure 35c), the shear stress, τ , at point O is

$$\tau = \frac{N_c + K \gamma H_c}{2} = \frac{q_u}{2} \quad (19)$$

where

q_u = unconfined compressive strength of the soil

Rearranging Equation (17) and solving for N_c yields

$$N_c = \frac{N_a H_a + \frac{1}{2} K \gamma (H_a^2 - H_c^2)}{H_c} \quad (20)$$

Replacing N_c in Equation (19) with Equation (20) results in the following expression for the shear stress at point O:

$$\tau = \frac{1}{2} \frac{N_a H_a + \frac{1}{2} K \gamma (H_a^2 - H_c^2)}{H_c} + K \gamma H_c \quad (21)$$

Rearranging Equation (21) produces a quadratic in terms of H_c . Solving for H_c yields

$$H_c = \frac{4\tau - 16\tau^2 - 4K\gamma(2N_a H_a + K\gamma H_a^2)}{2K\gamma} \quad (22)$$

To solve for B, the arch is assumed to make an angle $(\pi/4 - \phi/2)$ with the horizontal, since it is in a passive state induced by compaction. The weight of the arch, $2W$, is, from geometry

$$2W = \gamma 2BH_a - \frac{B^2(\pi/4 - \phi/2)}{\sin^2(\pi/4 - \phi/2)} + B^2 \cot(\pi/4 - \phi/2) \quad (23)$$

where

ϕ = angle of internal friction of compacted soil
in radians

c = cohesion intercept of compacted soil

This weight is resisted by the vertical shear force, T_v , at the abutments given by

$$T_v = 2cH_a + (N_a H_a + \frac{1}{2}K\gamma H_a^2) \tan \phi \quad (24)$$

Equating T_v and $2W$ and rearranging terms produces a quadratic in terms of B. Solving for B yields the following:

$$B = 2\gamma H_a - 4\gamma H_a^2 - 8 \frac{(\pi/4 - \phi/2)}{\sin^2(\pi/4 - \phi/2)} - \cot(\pi/4 - \phi/2)$$

$$\times \left(cH_a + (N_a H_a + \frac{1}{2}K\gamma H_a^2) \tan \phi \right)^{\frac{1}{2}} /$$

$$2 \frac{(\pi/4 - \phi/2)}{\sin^2(\pi/4 - \phi/2)} - \cot(\pi/4 - \phi/2) \quad (25)$$

The parameters H_c and B can be used as guides for estimating the maximum dimensions of a void that can be bridged by a soil arch. This theory is not meant to be rigorous because of the simplistic assumptions and the high uncertainty of many of the parameters. For example, the value of the residual lateral stress, N_a , is difficult to estimate since it is dependent upon the type of compaction equipment, thickness of lift, fill properties, etc. Partridge, et. al. (1981) use the following relationship for N_a :

$$N_a = \frac{\frac{2P}{\pi z}(1-\sin\phi) - 2c \cos\phi}{1-\sin\phi} \quad (26)$$

where

P = line impact load of roller

z = lift thickness

Values for P range from about 100 psf for very light rollers to over 2000 psf for heavy vibrating or impact rollers.

Partridge, et. al. (1981) programmed Equations (22) and (25) and performed parametric studies of H_c and B by varying γ , K , N_a , and H_a . The following trends were observed: (a) H_c increased significantly with increasing N_a ; however, instability of the arch was found to occur at very high values of N_a , where complex roots were obtained from Equation (22); (b) H_c increases marginally with increasing K ; this effect is enhanced with higher values of N_a ; (c) H_c increases marginally with higher γ ; once again the effect is magnified

with higher N_a ; (d) there is a minimum value of τ below which an arch will not form; and (e) B increases with higher N_a , c , K , and H_a . The value of c has the most influence on B , all other factors being equal.

There are a number of limitations to this theory, the most obvious of which is that it is two-dimensional. However, this is often not too bad an assumption since cavities usually develop over solution-enlarged joints and other lineaments. It also does not account for the presence of a water table, but this can be easily handled by the computer, even though the resulting equations are more complex. The arch must be able to withstand an embankment or foundation load, which further complicates the analysis. This can only be handled by using the computer. Estimation of the soil and compaction parameters is highly uncertain. For example, the coefficient of earth pressure may be higher than the at-rest condition due to compaction. Another point is that spalling on the underside of the arch may increase the angle from $(\pi/4 - \phi/2)$ to $(\pi/4 + \phi/2)$ because of relaxation of lateral pressures and creep (Partridge, et. al., 1981). Further development of this theory and parametric studies using soil parameters from the limestone residuum of southern Indiana is left as a future exercise.

Foundation Stabilization

When heavy loads are to be applied over cavernous bedrock, or where a compacted soil fill designed as in the previous section is not thick enough to support the increased stress, soil reinforcement is required. This may entail filling the cavities with a grout or lean concrete to prevent collapse, or it may involve pressure injecting grout into the soil above the bedrock surface, as illustrated by Method III in Figure 33. Steiner (1975) describes a grouting program consisting of primary, secondary, and tertiary holes designed to stabilize a soil layer above cavernous dolomite in southeastern Pennsylvania. Couch (1984), Ruth and Degner (1984), Simmons (1982), Kaderabek and Reynolds (1981), and Sowers (1975) describe cases of grouting and backfilling with concrete for foundation stabilization.

Couch (1984) describes a foundation treatment at a site on the Florida gulf coast north of Tampa in which deep dynamic compaction was used to break up a sand and clay-filled, badly solutioned, weak limestone system. A 225 ton steel weight was dropped from a height of 60 ft for 19 to 28 repetitions at each point on a 15 ft square grid pattern across the site. This method is most successful in loose granular materials, and therefore is not very applicable to the karst region of southern Indiana.

Reinforced Earth Slab

A novel technique is described by Steiner (1975) in which a reinforced earth slab was constructed to bridge over cavernous dolomite bedrock at a highway construction site in southeastern Pennsylvania. A 75 ft diameter sinkhole with no visible bottom developed as a result of erosion of clay seams in the bedrock during a period of heavy rainfall. Subsequent exploration revealed that competent rock existed at depths ranging from 20 to 200 ft, with some rock ledges inclined at 45° . Many voids were detected. One proposed solution was to construct a 3-ft thick two-way concrete slab over an area 150 ft by 1100 ft to support a highway embankment over the cavernous bedrock. The slab was designed to span a 50 ft diameter void with no appreciable deflection. A more economical alternative was to replace the concrete with a reinforced earth slab of the same design load capacity. The earth slab obtains its strength from the friction developed from the normal force exerted by the overburden along specially designed and patented steel strips. This particular design required a 15 ft embankment to provide sufficient overburden pressure. The reinforced earth slab was chosen because of its much lower cost. A schematic of its design is shown in Figure 36.

Construction procedures were rapid and involved placement of steel strips 5 in. on center, which were bolted end-to-end until the width of the slab was covered. Each

Figure 36. Reinforced Earth Slab Constructed over Cavities to Support a Highway Embankment. (From Steiner, 1975)

strip was anchored to elliptical-shaped facing elements at the ends of the slab. A 12 in. granular fill was compacted over the strips, and then another layer of strips was placed in the transverse direction. Two longitudinal and two transverse layers of strips were required. Construction of the reinforced earth slab resulted in a \$500,000 savings over the concrete alternative, and also lead to an unexpected 60 % savings in construction time.

Because reinforced earth had never been used in this capacity before, special precautions were taken. Before construction of the slab, a program of primary, secondary, and tertiary grouting of the soil above the bedrock and below the embankment was completed to prevent future sinkhole development. Although the slab was designed to span a 50 ft cavity, soil grouting was done to prevent such a condition from ever developing. In addition, vertical settlement plates were installed in the embankment to monitor deformation, and strain gauges were attached to some of the steel strips to monitor stress in the slab. The data were used to assess the performance of the entire structure.

Sinkhole Repair

Sinkhole formation is a dynamic process as emphasized by the previous discussions. Simply backfilling the depression with soil is dangerous if structures are to be located on or near the fill. Special design is required to prevent

continued underground erosion and future collapse. Several techniques are currently used, but they all follow the same basic procedure, with some variation in their detail and the materials used.

As noted earlier, three principal design factors must be considered in remedial treatment of sinkholes (Weber and Darnell, 1984): (a) maintain satisfactory drainage; (b) provide structural stability; and (c) minimize long-term risk of future subsidence or collapse. Hydrologic aspects have already been considered and will not be repeated here. Stability is often a problem because surface runoff accumulates in the depression, where soft, unstable, and sometimes organic material is deposited. Mass excavation is necessary not only to remove the unstable soil, but to locate the throat of the sinkhole.

Figure 37a illustrates a collapse triggered by construction. In shallow bedrock areas (5 to 10 ft below the surface), excavation of the soil down to competent rock, and identification of the solution cavity is usually not difficult. Where the overburden thickness is greater (10 to 20 ft), the throat is less conspicuous, but can often be found at the lowest part of the depression. Excavation is performed with scrapers, backhoes, or draglines. When bedrock is located below a depth of 15 to 20 ft, the throat is usually not visible at the surface, and the sinkhole appears as a broad, concave depression. Treatment of such a large

(a) Sinkhole collapses during site grading.

(b) Repair of sinkhole with concrete plug and compacted soil.

Figure 37. Repair of Sinkhole Using Excavation – Concrete Plug Method. (From Weber and Darnell, 1984)

feature is difficult. The Indiana Department of Highways (Strate, 1972) specifies that excavation should not exceed 15 ft unless soft, unstable, or organic material is present. Only those sinkholes located within the limits of a highway embankment should be treated at all because of the high cost of repair.

Williams and Darnell (1984) recommend the following procedure. Where the bedrock can be reached, the soil is excavated to expose any opening in the rock. The excavation is sloped back away from the opening as illustrated in Figure 37b. Dental excavation is then performed to remove all compressible material from around the throat. A few cubic yards of lean concrete with a high slump are allowed to infiltrate the bedrock cavities. If concrete continues to flow into the bedrock, other material such as crushed stone or shot rock can be used to choke off the openings. Concrete is then poured into the bottom of the excavation to form a plug. The cap is allowed to cure for two or three days, then the excavation is backfilled with a compacted clay. A moderate to highly plastic material, although more difficult to compact, is recommended to seal openings around the cap. The surface is graded to slope away from the center so that ponding of water does not occur.

Kemmerly (1984) describes a filter system that is used in place of the concrete. Granular material of progressively finer gradation is constructed over the bedrock

opening, and a clay backfill is compacted to fill the rest of the excavation. The filter system cannot be constructed in horizontal layers unless confined to a large bedrock opening, since horizontal flow would erode soil fines into the coarse material of the lower layers, leading to potential subsidence and collapse of adjacent areas. Each layer would have to be constructed in a radial pattern concentric with the bedrock opening, and intersecting the rock to form a seal. Such a system is difficult, if not impossible, to construct. Therefore, the concrete plug method is recommended for sinkholes located within the influence of a foundation or embankment load.

If continued excavation is unsuccessful in locating the throat of the sinkhole, flooding of the excavation is recommended. The increased weight and seepage force exerted by percolating water act to erode and expose the throat leading to the bedrock cavity. Once the opening is located, repair continues as outlined above. If the throat is still not obvious after flooding, excavation should continue until stable soil is encountered or a specified depth is reached. This depth is determined from a design analysis of the height of fill required to bridge over the suspected cavity (H_c from the previous section). Once this depth is reached, the natural soil is scarified and recompactd to seal any relict features. If the natural soil is only marginally stable, placement of an engineering fabric provides a stable

subgrade on which to construct the fill. Specification of 90 or 95 % of the maximum Modified Proctor density is recommended for compaction.

Weber and Darnell (1984) note that collapse is more likely to occur during construction than afterward. Altered surface drainage from site grading, vibration from heavy equipment, and blasting greatly increase the risk of sinkhole formation. This is an important consideration when planning construction activities in any karst region.

An outline of the procedure of sinkhole repair followed by the Pennsylvania Department of Transportation (Skean, et. al., 1963) is presented in Appendix D. Conditions in Pennsylvania are similar to those of southern Indiana, so techniques of sinkhole repair are similar for the two regions.

Remedial treatment of sinkholes is expensive and time consuming, and the method chosen should provide flexibility if unexpected subsurface conditions are encountered, which is often the rule, rather than the exception in karst areas. No one procedure can be followed in every case, and all geologic and hydrologic factors must be considered before design or construction is initiated.

CONCLUSION

Summary

The Mitchell Plain in southern Indiana is underlain by a series of intensely weathered limestone formations which exhibit classic karst topography. Lawrence County, located in the center of this karst area, was chosen for a study of the geotechnical engineering problems associated with the solution features that develop on carbonate bedrock. Several of these problems are reviewed for each landform-parent material association in the county. A rating of the engineering problems related to design and construction of transportation routes is also included.

Karst topography is defined as an irregular landscape characterized by streamless valleys, sinkholes, and streams that disappear underground, all developed by the action of surface and underground water in soluble rocks such as limestone and dolomite. Limestone is composed primarily of the mineral calcite; while dolomite rock has a significant amount of the mineral dolomite, as well as calcite. These rocks weather when carbonic acid in the groundwater reacts with calcite to form calcium bicarbonate, which is very soluble in water. The prerequisites for karst development

include the presence of a soluble rock at or near the surface, which is dense, highly jointed, and thinly bedded; entrenched major valleys below uplands to provide a hydraulic gradient; and at least a moderate amount of rainfall. Unique landforms associated with karst include terra rosa, lapies, sinkholes, dolines, collapse sinks, uvalas, sinking creeks, karst valleys, and subterranean cutoffs.

Underground cavities are formed as flowing groundwater dissolves carbonate bedrock along joints, bedding planes, and fault zones. Sinkholes form as soil erodes or collapses into solution-enlarged bedrock cavities. Subsidence or collapse is triggered by changes in the effective stress around the void which are either natural or man-induced. An irregular soil-bedrock contact is a direct result of the solutioning process. The residual soil derived from weathered carbonate rock is often highly plastic and contains the relict structure of the parent material. All of these factors present difficult problems for construction of transportation routes or other structures in karst regions.

A site evaluation consisting of preliminary studies, reconnaissance surveys, and appropriate field investigation techniques is required to thoroughly assess the potential for developing these problems. Preliminary studies of all available information and engineering data should precede any field survey. Reconnaissance surveys using remote sensing techniques reveal clues for locating areas of potential

instability. Geophysical methods are extremely valuable for providing subsurface data on a large areal extent. Seismic refraction is the most efficient technique for delineating the soil-bedrock profile; while electrical resistivity is usually the most efficient method for detecting subsurface cavities. Routine borings should be spaced in such a manner as to have the highest probability of locating anomalous areas. Exploratory borings using fast and efficient drilling techniques, such as air-track drilling, are required to confirm anomalies identified by the remote sensing and geophysical surveys.

Water is the key factor in the development of karst features. Subsurface erosion of soil is induced by a decline in the groundwater level or by alteration of surface drainage characteristics. Land subsidence, sinkhole collapse, and flooding are three major problems caused by hydrologic changes. Corrective or preventive measures include construction of adequate surface drainage facilities to carry runoff away from areas of potential instability; limiting groundwater withdrawal, or grouting to confine the area of its influence; or improving the drainage capacity and stability of existing sinkholes.

Engineering design in karst regions begins with proper identification and classification of the constituent materials. Foundation design requires an assessment of the possible failure modes so that appropriate analysis techniques

are chosen. Allowable bearing pressure on rock depends on the deformation characteristics of the entire rock mass. Design of large slopes and underground excavations in rock must consider failure of a rigid block along a discontinuity and failure within the intact rock mass. Block and wedge analyses and finite element studies are used to assess rock stability. A straight line Mohr-Coulomb or semi-empirical relationship may be chosen for the strength criteria in preliminary design, but in-situ testing is required for final assessment of rock stability for large projects.

A simplified arching theory is developed to evaluate the maximum cavity dimensions that a compacted clay fill can span. A case study of a reinforced earth slab to support a highway embankment over cavernous bedrock is reviewed. A procedure for remedial treatment of sinkholes is discussed in detail.

Conclusions

Several conclusions are drawn from this research and are summarized by the following points.

(a) A combination of aerial photographic interpretation and studies of existing geologic, hydrologic, and geotechnical information can be used to efficiently map carbonate residual soil areas based on landform-parent material associations on a regional basis. The engineering soil units on the map identify areas where particular geotechnical

problems can be expected.

(b) An efficient methodology is required for a thorough site evaluation in a karst region. It must include sufficient preliminary studies, reconnaissance surveys, and field investigations using geophysical techniques, sample borings, and soundings.

(c) A sinkhole density map of a large area can be easily prepared from existing drainage maps or airphotos, and is useful for planning transportation routes.

(d) Identification of lineaments from airphotos based on vegetative and tonal patterns is difficult, at least for small scale photography (1:24,000). However, linear trends in sinkhole alignment were observed and correlated with the regional joint pattern of the underlying limestone formations in Lawrence County.

(e) An understanding of the hydrologic aspects of subsidence, collapse, and flooding is required before any remedial or preventive technique can be applied in a karst region.

(f) A compacted clay fill can be used to bridge over solution-enlarged bedrock cavities. An elementary theory

has been developed for analysis, but further research is needed.

(g) More specific guidelines for sinkhole repair based on accumulated performance data from actual case histories are required.

(h) Literature describing analysis, design, and construction techniques applied to geotechnical engineering problems of karst regions is scarce. Much additional research is necessary to develop more analytical models for design and to transform the state-of-the-art to a more quantitative model.

Recommendations for Future Study

Recommendations for future study related to this research include the following points.

(a) Specific design guidelines for construction of foundations and embankments over carbonate rock should be developed.

(b) The arching theory developed for analyzing a compacted fill over cavernous bedrock should be extended to include the effects of both the groundwater table and external foundation loads.

(c) The method of (b) should be programmed into the computer, and parametric studies performed with data for the limestone residual soil of southern Indiana. The influence of soil properties on the design parameters can then be assessed.

(d) Further evaluation of techniques for analyzing stress and deformation within an embankment and its foundation is necessary. Guidelines for acceptable limits of stress and deformation should be developed for karst regions.

(e) Field monitoring or inspection for assessing the performance of recently constructed transportation routes through karst regions, such as S.R. 37 in Monroe, Lawrence, and Orange Counties, is recommended.

(f) A further review of case histories of karst problems in southern Indiana, along with an evaluation of the success of the solutions applied to them, is warranted.

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APPENDICES

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Appendix AAdditional Ratings of Agricultural Soils of Lawrence County

Table A1. Frost Potential, Corrosivity Ratings and Spring Water Table Depths of Agricultural Soils Associated with Landforms of Lawrence County.

LANDFORM	AGRICULTURAL SOIL	FROST POTENTIAL	CORROSIVITY		DEPTH TO GWL IN SPRING (ft)
			STEEL	CONCR.	
Flood Plain along East Fork White River	Nolin Newark Petrolia Chagrin	--	High	Mod.	---
		High	High	Low	+1.0-1.0
		High	High	Low	0.0-3.0
		Mod.	Low	Mod.	4.0-6.0
Flood Plains along Tributaries of East Fork White River	Haymond Stendal Wilbur Burnside	High	Low	Low	>6.0
		High	High	High	1.0-3.0
		High	Mod.	Mod.	1.5-3.0
		Mod.	Low	High	3.0-5.0
Sandy Terraces along East Fork White River	Alvin Abscota Bloomfield Tyner	Mod.	Low	High	>6.0
		Low	Low	Low	2.5-5.0
		Low	Low	High	>6.0
		Low	Low	High	>6.0
Bedrock-Defended Terraces	Elkinsville Pekin Burnside	High	Mod.	High	>6.0
		High	Mod.	High	2.0-6.0
		Mod.	Low	High	3.0-5.0
Sand/Loess over LS and SS-Sh	Princeton Martinsville	Mod.	Mod.	Mod.	>6.0
		Mod.	Mod.	Mod.	>6.0
Lakebed or Slackwater Plain	Bartle Henshaw	High --	High --	High --	1.0-2.0P ---
Lacustrine Terrace	Markland McGary	Mod.	High	Mod.	3.0-6.0P
		Mod.	High	Low	1.0-3.0
Sandstone-Shale Uplands	Ebal Gilpin Hosmer Wellston	Mod.	High	High	3.0-6.0P
		Mod.	Low	High	>6.0
		High	Mod.	High	2.5-3.0P
		High	Mod.	High	>6.0
Sandstone-Shale Sideslopes	Berks Gilpin Weikert	Low	Low	High	>6.0
		Mod.	Low	High	>6.0
		Mod.	Mod.	Mod.	>6.0
Sandstone-Shale over Limestone	Ebal Gilpin Hosmer Wellston Caneyville	Mod.	High	High	3.0-6.0P
		Mod.	Low	High	>6.0
		High	Mod.	High	2.5-3.0P
		High	Mod.	High	>6.0
		--	High	Mod.	>6.0
Limestone Uplands	Bedford Crider Frederick Hoosierville Muren	High	High	High	1.5-3.5P
		--	Mod.	Mod.	>6.0
		Mod.	Mod.	High	>6.0
		High	High	Mod.	0.0-1.0
		High	High	Mod.	3.0-6.0
Limestone Sideslopes	Caneyville Frederick	--	High	Mod.	>6.0
		Mod.	Mod.	High	>6.0

P = Perched Water Table. (Data Compiled from "Soil Survey of Lawrence County, Indiana", 1984)

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Appendix BEvaluation of Accessible Explorations and Drilling Methods

Table B1. Evaluation of Types of Accessible Explorations.

<u>Technique</u>	<u>Principles of technique</u>	<u>Data Obtained</u>	<u>Application</u>	<u>Comment</u>
<u>TRIAL EXCAVATIONS</u>				
	A hole is opened in the ground through the ground by an auger or machine in situ ground conditions.	Position and outlines of excavations. Dimensions of excavations in three dimensions. Excavation technique parameters.	Evaluation of rock and soil characteristics. Access to field test and laboratory test sample locations.	Usually limited to shallow depth because of cost. The excavation may cause a problem in the subsequent construction, water inflow problems below water table.
Pits	Excavations opened by hand or machine. Approximately square in outline. Size < 2 m square $\times$ 2 m deep.	As above.	Access to field test and laboratory test sample locations. Examination of overburden conditions.	Need for support for personnel safety.
Trenches	Opened by hand or by machine. About 1 m wide $\times$ 6 m deep of indefinite length.	As above.	Used to trace lateral changes in rock and fill, rock and soil strata, detection of faults.	Mechanical excavation may produce unstable slides. Entry is hazardous.
Shafts	Sunk either as deep pits by hand, grab, and blasting excavation, by large diameter auger, or boring machine.	As above.	Access to field and laboratory test sample locations. Evaluation of rock and soil conditions with emphasis on distinguishing types. Entry to underground caverns.	Requirement for support may obscure observations. Water inflow problems below water table.
Adits	A tunnel (generally horizontal) from surface or the bottom of a shaft. Driven by conventional tunneling methods. Size usually < 2 m $\times$ 2 m.	As above.	As above.	As above.

(From Table 2.5.a of Price, et. al., 1981)

Table B2. Evaluation of Drilling Methods.

Technique	Principle of technique	Data Obtained	Application	Comment
<u>BOREHOLES</u>				
Percussion drilling (shell and auger)	A small diameter circular hole (30 mm-600 mm), usually vertical, is made by percussive or rotary methods to obtain samples of the ground penetrated. In loose ground the borehole may have to be supported by casing. Motive power is provided by motor except for hand augering.	Position of suburface units along the line of the borehole. Water table levels. Resistance to boring.	To obtain samples for classification and laboratory testing. Access to locations for in-situ testing (e.g. permeability, pressure etc.). observation (water levels, in-situ stress etc.) and in-situ instrumentation (attenuometer, reflectometer etc.).	Small size of samples in comparison with scale of mass and engineering project. Limited opportunity for three dimensional observations. Techniques often limited to either rock or soil boring. Special difficulties associated with rock head (at change from soil to rock boring technique).
	A hole is made by repeatedly dropping a cylindrical boring tool at the bottom of the hole to loosen, penetrate and recover disturbed samples of the ground. Holes always vertical. "Undisturbed" samples are obtained by driving tubes into the ground. Hard (rock) may be penetrated by chisel-ling.	As above. Samples obtained as "disturbed" (for identification only) and "undisturbed" (taken by tube samplers for laboratory tests).	As above, but in situ testing limited, because of disturbance of material and irregular form of hole.	Strata boundaries identified by drill crew. Samples not continuous, and "undisturbed" sample not truly undisturbed. Percussion drilling in rocks produces low quality samples of little value.
Rotary core drilling	A hole is made by pushing down and rotating a cylinder (the core barrel) armed with a cutting bit (tipped with diamond or hard steel cutting teeth) into the ground. A cylindrical core is obtained. Holes mostly vertical but may be inclined. Cuttings from the bit are brought to surface by water, mud or air-flush.	As for BOREHOLES. Samples are cores of varying sizes (see table 2.5.c).	Mostly for the recovery of continuous samples of rock although some special soil sampling core barrels are sometimes used.	The aim of rotary core drilling is to obtain 100% core recovery which depends upon the use of the correct combination of core barrel bit and drill and experience of local site geology. Site investigation and random extraction are necessary enough to achieve this.
Rotary "open hole" drilling	A hole is made by pushing down and rotating a drill rod tipped with a drilling bit. The bit is commonly of hard-rock type and the preparation of a simple geological record. Cuttings from the bit are brought to surface by water, mud or airflush. Mud supports the borehole.	No "undisturbed" samples allow identification of rock type and the preparation of a simple geological record.	For preliminary examination of site geology, amplification of geology derived from rotary core drilling or in-situ testing purposes.	Limited accuracy of identification because of small size of cuttings. Generally used only in rock. Difficulties in penetrating water bearing clayey overburden. Attendance of engineering geologist on drilling rig essential for preservation of geological records.

Table B2. Continued

<u>Technique</u>	<u>Principle of technique</u>	<u>Data Obtained</u>	<u>Application</u>	<u>Comment</u>
Rotary percussive drilling	As above, but a "down the hole" hammer provides a percussive action to the drilling bit.	As above.	As above.	As above, but cutting generally even smaller than those from open-hole drilling making identification yet more difficult. As drilling rises of this type can case boreholes.
Continuous flight augering	A hole is made by pushing and rotating into the ground a spiral auger around a metal rod. The rod can be solid or hollow and the bottom of the auger is tipped with a drilling bit.	As above. For augers with solid stems no undisturbed sampling is possible. Undisturbed samples may be obtained through hollow stem augers.	Shallow depth boreholes in overburden soils or very soft rock.	Limited to soil or very soft rock; no facility to penetrate hard rock or obstructions (boulders etc.). Often difficult, if not impossible, to install casing.
Wash boring (jetting)	A hole is made by pumping water at high pressure through a drilling tool and washing material from the ground. "Undisturbed" tube samples may be taken in the hole.	As above in BOREHOLES.	Shallow depth boreholes in overburden.	Limited accuracy of sample depth and type observations. Casing difficult.
Hand augering	A hole is made using small augers and samples attached to light drill rods. "Undisturbed" tube samples may be taken. Motive power is provided by one or two men.	Simple descriptions of overburden to shallow depths, usually 0-4 m.	Shallow depth holes in overburden, for preliminary investigation work, site classification. In aid of engineering geological mapping.	Limited depth capacity. No possibility of using casing.

(From Table 2.5.b of Price, et. al., 1981)

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Appendix CCSIR and NGI Classification Systems for Jointed Rock Masses

Table C1. CSIR Geomechanics Classification of Jointed Rock Masses.

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

PARAMETER		RANGES OF VALUES					
1	Strength of intact rock material	Point load strength index	> 8 MPa	4-8 MPa	2-4 MPa	1-2 MPa	For this low range - uniaxial compressive test is preferred
		Uniaxial compressive strength	> 200 MPa	100-200 MPa	50-100 MPa	25-50 MPa	10-25 MPa 3-10 MPa 1-3 MPa
	Rating		15	12	7	4	2 1 0
2	Drill core quality RQD		90%-100%	75%-90%	50%-75%	25%-50%	< 25%
	Rating		20	17	13	8	3
3	Spacing of joints		> 3m	1-3m	0.3-1m	50-300 mm	< 50 mm
	Rating		30	25	20	10	5
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation < 1 mm Hard joint wall rock	Slightly rough surfaces Separation < 1 mm Soft joint wall rock	Slackensided surfaces Gauge < 5 mm thick Joints open 1-5 mm Continuous joints	Soft gauge > 5 mm thick Joints open > 5 mm Continuous joints
	Rating		25	20	12	6	0
5	Ground water	Inflow per 10m tunnel length	None		< 25 litres/min	25-125 litres/min	> 125 litres/min
		Ratio $\frac{\text{joint water pressure}}{\text{major principal stress}}$	0		0.0-0.2	0.2-0.5	> 0.5
		General conditions	Completely dry		Moist only (interstitial water)	Water under moderate pressure	Severe water problems
	Rating		10		7	4	0

B. RATING ADJUSTMENT FOR JOINT ORIENTATIONS

Strike and dip orientations of joints		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100-81	80-61	60-41	40-21	< 20
Class No	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK MASS CLASSES

Class No	I	II	III	IV	V
Average stand-up time	10 years for 5m span	6 months for 4 m span	1 week for 3 m span	5 hours for 1.5 m span	10 min for 0.5 m span
Cohesion of the rock mass	> 300 kPa	200-300 kPa	150-200 kPa	100-150 kPa	< 100 kPa
Friction angle of the rock mass	> 45°	40°-45°	35°-40°	30°-35°	< 30°

Table C2. The Effect of Joint Strike and Dip Orientations in Tunnelling.

Strike perpendicular to tunnel axis				Strike parallel to tunnel axis		Dip 0°-20° irrespective of strike
Drive with dip		Drive against dip				
Dip 45°-90°	Dip 20°-45°	Dip 45°-90°	Dip 20°-45°	Dip 45°-90°	Dip 20°-45°	
Very favourable	Favourable	Fair	Unfavourable	Very unfavourable	Fair	Unfavourable

Practical Example Using CSIR Geomechanics Classification:

Consider the example of a granitic rock mass in which a tunnel is to be driven. The classification has been carried out as follows:

<i>Classification Parameter</i>	<i>Value or Description</i>	<i>Rating</i>
1. Strength of intact material	150 MPa	12
2. RQD	70%	13
3. Joint spacing	0.5m.	20
4. Condition of joints	Slightly rough surfaces Separation <1mm. Hard joint wall rock	20
5. Ground water	Water under moderate pressure	4
Total score		<u>69</u>

The tunnel has been oriented such that the dominant joint set strikes perpendicular to the tunnel axis with a dip of 30° against the drive direction. From Table C2 this situation is described as unfavourable for which a rating adjustment of -10 is obtained from Table C1. Thus the final rock mass rating becomes 59 which places the rock mass at the upper end of Class III with a description of fair.

Table C3. Classification of Individual Parameters Using the NGI Tunnelling Quality Index.

<i>Description</i>	<i>Value</i>	<i>Notes</i>
1. ROCK QUALITY DESIGNATION	RQD	
A. Very poor	0 - 25	1. Where RQD is reported or measured as ≤ 10 (including 0), a nominal value of 10 is used to evaluate Q. 2. RQD intervals of 5, i.e. 100, 95, 90 etc are sufficiently accurate.
B. Poor	25 - 50	
C. Fair	50 - 75	
D. Good	75 - 90	
E. Excellent	90 - 100	
2. JOINT SET NUMBER	J_n	
A. Massive, no or few joints	0.5 - 1.0	1. For intersections use $(3.0 \times J_n)$ 2. For portals use $(2.0 \times J_n)$
B. One joint set	2	
C. One joint set plus random	3	
D. Two joint sets	4	
E. Two joint sets plus random	6	
F. Three joint sets	9	
G. Three joint sets plus random	12	
H. Four or more joint sets, random, heavily jointed 'sugar cube', etc	15	
J. Crushed rock, earthlike	20	
3. JOINT ROUGHNESS NUMBER	J_r	
<i>a. Rock wall contact and</i>		1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3m. 2. $J_r = 0.5$ can be used for planar, slickensided joints having lineations, provided the lineations are orientated for minimum strength.
<i>b. Rock wall contact before 10 cms shear.</i>		
A. Discontinuous joints	4	
B. Rough or irregular, undulating	3	
C. Smooth, undulating	2	
D. Slickensided, undulating	1.5	
E. Rough or irregular, planar	1.5	
F. Smooth, planar	1.0	
G. Slickensided, planar	0.5	
<i>c. No rock wall contact when sheared.</i>		
H. Zone containing clay minerals thick enough to prevent rock wall contact.	1.0	
J. Sandy, gravelly or crushed zone thick enough to prevent rock wall contact.	1.0	
4. JOINT ALTERATION NUMBER	J_a	ϕ_r (approx.)
<i>a. Rock wall contact.</i>		
A. Tightly healed, hard, non-softening, impermeable filling	0.75	—

Table C3. Continued.

	J_a	ϕ_r (approx.)	
B. Unaltered joint walls, surface staining only	1.0	(25° - 35°)	
C. Slightly altered joint walls non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc	2.0	(25° - 30°)	1. Values of ϕ_r , the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.
D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	(20° - 25°)	
E. Softening or low friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1-2mm or less in thickness)	4.0	(8° - 16°)	
<i>b. Rock wall contact before 10 cms shear.</i>			
F. Sandy particles, clay-free disintegrated rock etc	4.0	(25° - 30°)	
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous, < 5mm thick)	6.0	(16° - 24°)	
H. Medium or low over-consolidation, softening, clay mineral fillings, (continuous, < 5mm thick)	8.0	(12° - 16°)	
J. Swelling clay fillings, i.e. montmorillonite (continuous, < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water	8.0 - 12.0	(6° - 12°)	
<i>c. No rock wall contact when sheared.</i>			
K. Zones or bands of disintegrated	6.0		
L. or crushed rock and clay (see	8.0		
M. G,H and J for clay conditions)	8.0 - 12.0	(6° - 24°)	
N. Zones or bands of silty- or sandy clay, small clay fraction, (non-softening)	5.0		
Q. Thick, continuous zones or	10.0 - 13.0		
P. bands of clay (see G, H and	13.0 - 20.0	(6° - 24°)	
R. J for clay conditions)			

5. JOINT WATER REDUCTION FACTOR	J_w	approx. water pressure (Kgf/cm ²)	
A. Dry excavations or minor inflow, i.e. < 5 lit/min. locally	1.0	< 1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0	1. Factors C to F are crude estimates. Increase J_w if drainage measures are installed.
D. Large inflow or high pressure, considerable outwash of fillings	0.33	2.5 - 10.0	
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10	2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure continuing without decay	0.1 - 0.05	> 10	

Table C3. Continued.

6. STRESS REDUCTION FACTOR				
<i>a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated.</i>				
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)			SRF	
			10.0	
B. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth < 50m)			5.0	1. Reduce these values of SRF by 25 - 50% if the relevant shear zones only influence but do not intersect the excavation.
C. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth > 50m)			2.5	
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)			7.5	
E. Single shear zones in competent rock (clay free), (depth of excavation < 50m)			5.0	
F. Single shear zones in competent rock (clay free), (depth of excavation > 50m)			2.5	2. For strongly anisotropic virgin stress field (if measured) : when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where σ_c = unconfined compressive strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.
G. Loose open joints, heavily jointed or 'sugar cube' (any depth)			5.0	
<i>b. Competent rock, rock stress problems</i>				
	σ_c/σ_1	σ_t/σ_1	SRF	
H. Low stress, near surface	>200	>13	2.5	
J. Medium stress	200-10	13-0.66	1.0	
K. High stress, very tight structure (usually favourable to stability, may be unfavourable for wall stability)	10-5	0.66-0.33	0.5-2	
L. Mild rock burst (massive rock)	5-2.5	0.33-0.16	5-10	
M. Heavy rock burst (massive rock)	<2.5	<0.16	10-20	3. Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).
<i>c. Squeezing rock, plastic flow of incompetent rock under the influence of high rock pressure</i>				
			SRF	
N. Mild squeezing rock pressure			5-10	
O. Heavy squeezing rock pressure			10-20	
<i>d. Swelling rock, chemical swelling activity depending upon presence of water</i>				
P. Mild swelling rock pressure			5-10	
R. Heavy swelling rock pressure			10-20	
ADDITIONAL NOTES ON THE USE OF THESE TABLES				
When making estimates of the rock mass quality (Q) the following guidelines should be followed, in addition to the notes listed in the tables:				
1. When borehole core is unavailable, RQD can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relation can be used to convert this number to RQD for the case of clay free rock masses : $RQD = 115 - 3.3J_v \text{ (approx.)} \quad \text{where } J_v = \text{total number of joints per m}^3$ $(RQD = 100 \text{ for } J_v < 4.5)$				
2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed these parallel "joints" should obviously be counted as a complete joint set. However, if there are few "joints" visible, or only occasional breaks in the core due to these features, then it will be more appropriate to count them as "random joints" when evaluating J_n .				
3. The parameters J_r and J_a (representing shear strength) should be relevant to the <i>weakest significant joint set or clay filled discontinuity</i> in the given zone. However, if the joint set or discontinuity with the minimum value of (J_r/J_a) is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_r/J_a should be used when evaluating Q. The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.				
4. When a rock mass contains clay, the factor SRF appropriate to loosening loads should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent the strength of the intact rock may				

Table C.3. Continued

- become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.
5. The compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the saturated condition if this is appropriate to present or future in situ conditions. A very conservative estimate of strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

Practical Example Using the NGI Tunnelling Quality Index:

Quality Index (Q) =

$$Q = \left(\frac{RQD}{J_n} \right) \times \left(\frac{J_r}{J_a} \right) \times \left(\frac{J_w}{SRF} \right)$$

where

RQD is Deere's Rock Quality Designation

J_n is the joint set number,

J_r is the joint roughness number,

J_a is the joint alteration number,

J_w is the joint water reduction factor, and

SRF is a stress reduction factor.

An underground crusher station is to be excavated in the limestone footwall of a lead-zinc ore body and it is required to find the span which can be left unsupported. The analysis is carried out as follows:

Item	Description	Value
1. Rock Quality	Good	RQD = 80%
2. Joint sets	Two sets	$J_n = 4$
3. Joint roughness	Rough	$J_r = 3$
4. Joint alteration	Clay gouge	$J_a = 4$
5. Joint water	Large inflow	$J_w = 0.33$
6. Stress reduction	Medium stress	SRF = 1.0

Hence
$$Q = \frac{80}{4} \times \frac{3}{4} \times \frac{0.33}{1} = 5$$

Tables C1 to C3 and examples are from Hoek and Brown (1980b).

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Appendix D

Suggested Special Provisions for Sinkhole Areas

Sinkholes that are either evident or are encountered during construction shall be remedied by application of appropriate remedial measures, as determined or approved by the Engineer. Treatment shall be in accordance with suggested general procedures, according to the type of sinkhole and zone of occurrence. In general, sinkholes or solution caverns are either evident on the existing ground surface in embankment foundation areas or are partially developed during excavation. The basic sinkhole types are shown in Figure D1. Suggested procedures for dealing with each sinkhole type are discussed as follows.

I. SINKHOLES OR SOLUTION CAVERNS IN EXCAVATION

A. Partially Developed Sinkhole or Solution Cavern in Rock Subgrade

The suggested procedure in this case is to clean out all debris in the sinkhole or solution cavern, seal any opening(s) in the rock with Class C concrete, and then backfill the cavity with rock of diminishing gradation to the approximate subgrade elevation. Once this has been done, the contractor should then seal the top of rock backfill at subgrade elevation with Class C concrete in order to prevent infiltration of subbase drainage through or around the zone of the backfill.

B. Partially Developed Sinkhole in Soil Subgrade

The suggested procedure in this case is to first excavate and clean out the opening either to rock or firm material. If the rock surface is exposed in the bottom of the cavity, then seal any opening(s) in the rock with Class C concrete, and complete the backfill to subgrade elevation with impervious soil. If the base of the cleaned surface is still in soil, then backfill the entire opening with impervious soil to subgrade elevation. In either case, infiltration of subbase drainage should be prevented.

II. SINKHOLES IN EMBANKMENT FOUNDATION

A. Collapse Sinkhole in Rock

The suggested procedure in this case is to clean out all loose soil and debris, seal all openings in the rock with Class C concrete, and backfill the void with rock of diminishing gradation to an elevation where there is contact between bedrock and overburden. Then seal the entire top of the rock backfill surface with a layer of Class C concrete. The final backfilling of the cavity should be to existing

ground level with suitably compacted embankment material.

B. Collapse Sinkhole in Soil

The suggested procedure in this case is to first excavate and clean out the opening either to rock or firm material. If the rock surface is exposed, seal any opening(s) in the rock with Class C concrete, and complete backfill with impervious soil to the original ground level. If the base of the cleaned surface is still in soil, then backfill the entire opening with impervious soil to the original ground level.

C. Funnel Sinkhole in Rock

The suggested procedure in this case is the same as that for a collapse sinkhole in rock. The contractor must clean out all loose soil and debris, seal all openings in the rock with Class C concrete, backfill the void with rock of diminishing gradation to an elevation where there is contact between bedrock and overburden, then seal the entire top of the rock backfill with a layer of Class C concrete. The final backfilling of the cavity should be continued to existing ground level with suitably compacted embankment materials.

D. Funnel Depression in Soil

The suggested procedure in this case is the same as that for a collapse sinkhole in soil. The Contractor must excavate and clean out the opening either to rock or firm material. If the rock surface is exposed, seal any opening(s) in the rock with Class C concrete, and complete backfill with impervious soil to the original ground level. If the base of the cleaned surface is still in soil, then backfill the entire opening with impervious soil to the original ground level.

Soil Material for Backfill

Soil material to be used for backfill of sinkholes shall be suitable impervious clayey soil (A-6 or A-7-6), as approved by the Engineer.

Construction Requirements

All backfilling of sinkholes shall be done in accordance with applicable sections of the Pennsylvania Department of Transportation Specifications, as well as pertinent special provisions to these specifications.

Exploratory Drilling

Prior to the excavation and filling of any sinkholes or solution caverns, exploratory air track drilling may be performed at the discretion of the Engineer, in order to determine or more accurately define the presence of solution caverns.

Surface Drainage

As noted in the previous section on construction procedures, surface drainage shall be diverted from areas of known sinkholes. Drainage structures, wherever needed, shall be constructed and ditches excavated as soon as practical. Should final ditching prove undesirable, the Contractor may, with the permission of the Engineer, provide and maintain temporary ditches.

Basis of Payment

Special sinkhole treatment shall be paid for under the following items:

- Class 1 Excavation (Sinkhole)
- Foreign borrow Excavation (Rock)
- Selected Borrow Excavation (Soil)
- Class C Concrete
- Exploratory Drilling

Any portion of, or all of, the quantities of the above items may be deleted at the direction of the Engineer. Any additional quantities necessary will be paid for at the respective contract unit prices.

* Skean, Bernstein, and Claypoole, 1963, "Sinkholes in Highway Engineering in Pennsylvania", Commonwealth of Pennsylvania, Department of Transportation, Bureau of Research and Testing, Harrisburg, Pennsylvania.

I. Partially Developed Sinkholes or Solution Caverns in Excavation

A. Rock in Subgrade

B. Soil in Subgrade

II. Sinkholes in Embankment. Foundation Areas

A. Collapse Sinkhole in Rock

B. Collapse Sinkhole in Soil

C. Funnel Sinkhole in Rock

D. Funnel Sinkhole in Soil

Figure D1. Typical Sinkhole Formations in Pennsylvania.
(From Skean, et. al., 1963)

COVER DESIGN BY ALDO GIORGINI