RESOURCES & TRANSPORTATION:

31st Annual

HIGHWAY GEOLOGY SYMPOSIUM PROCEEDINGS

Joe C. Thompson Conference Center
The University of Texas at Austin
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ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

PROCEEDINGS

Contents

KARST PROBLEMS ALONG TENNESSEE HIGHWAYS: AN OVERVIEW

by

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INTRODUCTION

Through the years Tennessee highways have been plagued with numerous sinkhole-type collapses and repeated flooding in areas of karst terrain. Some of these collapses have been catastrophic and have presented challenging geotechnical engineering problems which, in some cases, have resulted in unique solutions. These problems have become more acute owing to continued rapid, unplanned commercial and residential expansion into less favorable geologic sites such as active karst areas.

The numerous construction and maintenance problems along Tennessee highways involving collapse and/or flooding of the roadway and adjacent areas are directly attributable to alterations in ground-water flow and surface runoff.

The hydrology of the geologic setting is most instrumental in affecting the surface stability in karst terrain. It is also easily and constantly manipulated by mechanical means. Typically, ground-water flows from one minute fracture to another and eventually to open cavities. A network of solution-enlarged joints and cracks results and it eventually coalesces into caverns of varying dimensions. Surface water enters the ground-water regime by percolating through residual clay soils down into fractures of the underlying bedrock.

Problems that usually occur as a result of changes in the ground-water regime involve (1) development or enlargement of sinkholes, collapse features, and cave entrances; (2) flooding resulting from alteration of both the surface and subsurface drainage systems; and (3) alteration of ground-water levels, spring and well discharge rates, and other problems that are combinations of all the above. In addition, flooding and other drainage problems must be recognized as significant both in litigation that may occur as a result of the flooding damage and in the formulation of remedial concepts.

GEOGRAPHIC DISTRIBUTION OF KARST PROBLEMS

Sinkholes, depressions, sinking creeks, and cave entrances are the surface reflection of rigorous subsurface solution activity. Figure 1 illustrates the geographic distribution of numerous zones of karst-related characteristics across Tennessee.

Karst-related geotechnical engineering problems in Tennessee are confined to three major physiographic provinces: the Valley and Ridge, the Highland Rim, and the Nashville Basin (fig. 2). Within these provinces are disbribution zones that reflect areas of intense solution activity.

In the Valley and Ridge province, karst-related characteristics are zoned into bands reflecting strike belts of carbonate strata such as the Sequatchie Valley.

The next zone that can be delineated is a belt along and parallel to the western Cumberland Plateau escarpment. It is a few miles wide and runs longitudinally from the northern to the southern Tennessee border.

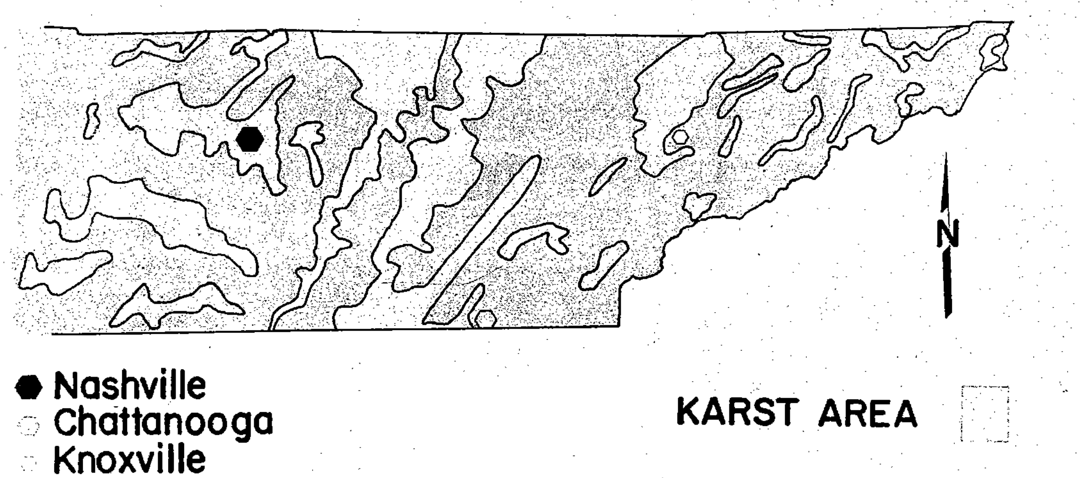


Figure 1. The geographic distribution of karst areas in Tennessee.

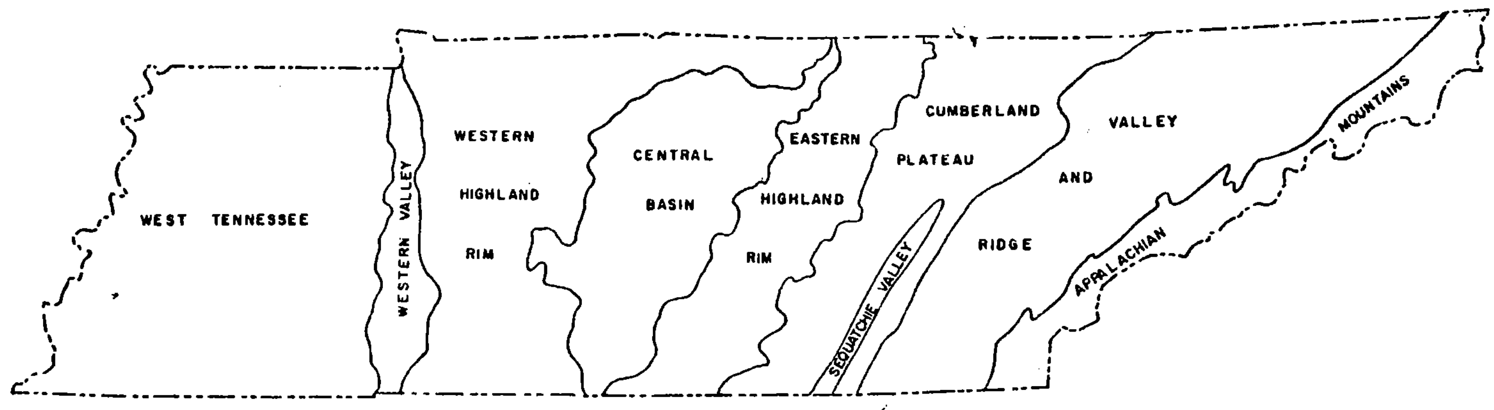


Figure 2. Illustrated above are the physiographic provinces of Tennessee. The major karst areas of Tennessee extend from the Western Highland Rim through the Ridge and Valley Province. (After Hershey and Maher, 1963).

Another zone of intense karst activity characterized by numerous caves is along the eastern Highland Rim escarpment where it joins the Nashville Basin. This area is parallel to the karst zone along the western Cumberland Plateau escarpment.

The karst area to the west, which encompasses the Nashville Basin and western Highland Rim, is arranged in belts coinciding with the drainage areas of the Cumberland, Duck, and Elk Rivers. The location of these karst belts may be related directly to the general joint system of the Nashville Dome.

Generally, there is a geographic distribution of karst-type problems and characteristics that correspond to specific topographic features across Tennessee.

GEOLOGIC DISTRIBUTION OF KARST PROBLEMS

In addition to geographic distribution, karst problems also have a geologic distribution across Tennessee. Certain geologic formations contain more caves, sinkholes, depressions, and other karst characteristics than do other geologic formations.

In areas of karst development, cave systems are the focus of intense solution activity and are reliable indicators of areas prone to karst-type geotechnical engineering problems. Using cave location data from Barr (1961), Matthews (1971), and Moore (1973), a calculation can be made of the geologic distribution of caves. Given a total of 1002 cave locations, the Monteagle Formation contains 23.2 percent of the known caves in Tennessee (fig. 3). Following the Monteagle Formation are the Knox Group (8.9 percent), Bigby-Cannon Formation (8.2 percent), Catheys Formation (7.7 percent), Bangor Formation (7.1 percent), Warsaw Formation (6.3 percent), and the Saint Louis Formation (5.3 percent). Small percentages are found in a number of other formations.

GEOLOGIC AGE	FORMATION	VALLEY & RIDGE	HIGHLAND RIM	NASHVILLE BASIN	% OF KNOWN CAVES
Mississippian	Bangor		X		7.1
	Monteagle		X		23.2
	St. Louis		X		5.3
	Warsaw		X		6.3
	Ft. Payne		X		3.7
	Lepiers			X	2.0
	Catheys			X	7.7
Ordovician	Bigby-Cannon			X	8.2
	Ridley			X	2.3
	Knox GP.				8.9
Cambrian	Copper Ridge	X			2.9
Pre-Cambrian Through Mississippian	(32 other formations)	X	X	X	22.4

Figure 3. General geologic distribution of caves in Tennessee. Karst problems involving highways and other types of construction can be expected when working in the strata of these formations.

Karst-related geotechnical engineering problems in the Valley and Ridge of East Tennessee are principally found in either the Knox Group (and equivalent formations) or the Holston Formation. Along the Highland Rim, most karst problems are found in the Bangor, Monteagle, and Warsaw Formations. In the Nashville Basin the Bigby-Cannon, Catheys, Ridley, and Lebanon Formations contain most of the carbonate solution problems experienced in geotechncial engineering.

TYPES OF KARST PROBLEMS

Introduction

Karst-related difficulties documented by the Tennessee Department of Transportation occur in both Middle and East Tennessee. Sinkholes, caves, depressions, and other karst features can greatly affect highways and are recognized as areas of potential instability. Most of the karst-type problems in Tennessee are grouped into collapse-type failures or drainage-related problems. Additionally, there is always the potential for litigation, which often accompanies these karst-related problems.

Collapse

Although collapse failures are usually the result of certain karst drainage problems, they are unique in their immediate effects.

Collapse failures along transportation routes in Tennessee can be any of the following types: collapse of roadway surfaces and cutslopes, collapse in ditchlines and relocated creek channels, collapse failures involving structure foundations, and combinations of the above.

In most cases collapses of roadway surfaces (dropouts) occur in cut sections of the highway, especially where excavation has approached the soil-rock interface (fig. 4). These dropouts are usually circular and of varying size with a limestone pinnacle visable in the lower parts of the dropout.

Collapse failures in the roadway surface are second only to dropouts in ditchlines and relocated channels (fig. 5). These are by far the most common of the collapse problems reported to the Tennessee Department of Transportation.

Two factors seem to govern the actual incidence of karst-related dropouts in ditchlines and relocated channels: first, surface drainage being allowed to stand or to drain slowly from unpaved ditches, and second, the exposure of suddenly large quantities of water to freshly excavated ditches and relocated channels. In addition to seepage and erosion, a third, less common, cause of collapse is surface water piped directly into caves adjacent to or in ditchlines.

In most cases these dropouts are circular to irregularly shaped and involve solution cavities that intercept and drain the ditch or channel. Solution cavities exposed in ditchline dropouts are generally less than 66 to 95 cm (24 to 36 inches) in diameter whereas the actual surface depression may be 3 to 4 m (10 to 12 ft.).

A less frequent occurrence of collapse features involves structure foundations. Detailed local drilling investigations prior to construction usually delineate such potential problems.

In Tennessee the settlement of residual soil around piling is the most common karst-type failure associated with structures (fig. 6). Pilings, with point bearing on cavernous strata, act

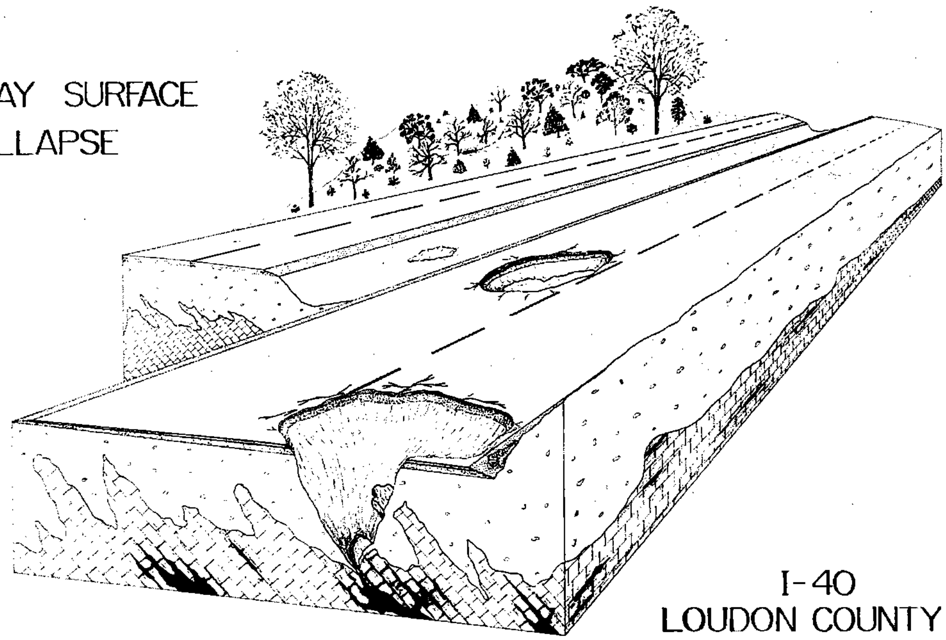


Figure 4. This schematic drawing illustrates a typical roadway surface collapse in karst terrain. These types of collapses have plagued a section of I-40 in Loudon County.

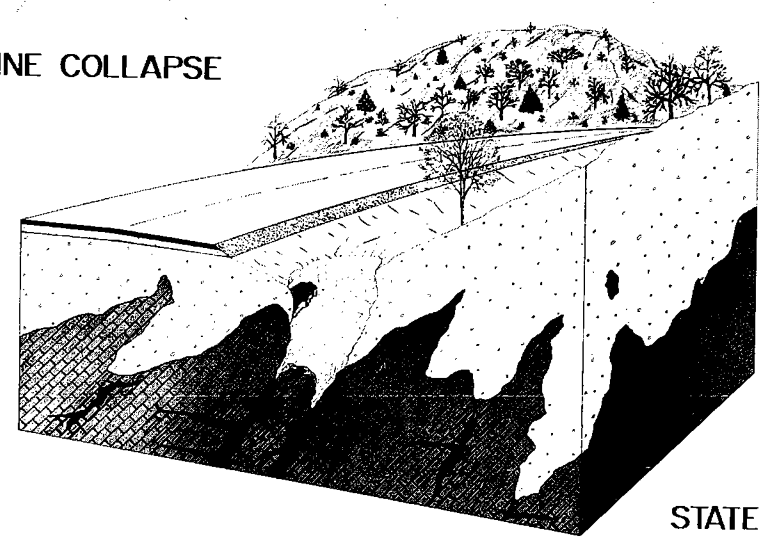


Figure 5. The occurrence of collapse failures in the ditchlines of roadways and drainage channels in karst areas are usually the result of increased seepage pressures which tend to weaken soil strength parameters.

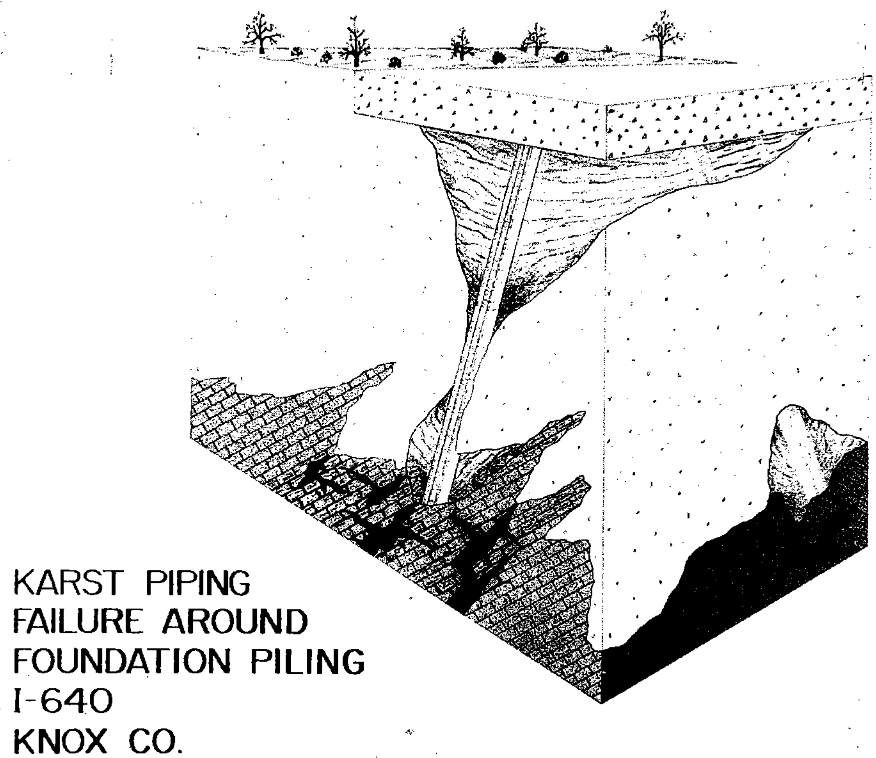


Figure 6. The collapse and/or settlement of bridge abutments and pier footings into underlying voids in the soil created by the internal erosion of soil around foundation piling can be a major problem in karst areas.

as focus areas for infiltrating water. Extending to a "bottom outlet" such as a cave or fracture in the rock, the erosion of the residual soil occurs by way of the infiltrating water around the piling. The collapse and/or settlement of bridge abutments and pier footings into underlying voids in the soil is the result (fig. 7).

Most collapse-type karst problems along Tennessee highways involve a combination of both ditchline and/or channel collapses and roadway collapses. It has been the Department's experience that usually a dropout will occur in the ditchline first. Afterward, if the ditchline dropout is not corrected, a collapse in the roadway adjacent to the initial collapse will occur. Experience has shown that most of the collapse dropouts result after moderate to heavy periods of precipitation, which may occur at any time during the year but are more frequent in the spring.

Drainage

Understanding the surface and subsurface drainage patterns and characteristics in karst areas is mandatory in developing applicable remedial concepts for damaged highway sections.

Drainage problems experienced in karst areas along Tennessee highways primarily involve flooding of depressions and sinkholes (fig. 8). Secondarily, the drainage problems involve surface drainage being directed into and away from sinkholes and depressions. Flooding of the highway is the main result of karst drainage problems, but collapses or dropouts can be precipitated from the flood waters.

In recent years, commercial and residential development in outlying rural areas has had a dramatic impact on the highways constructed through depressions and sinkholes. One of the

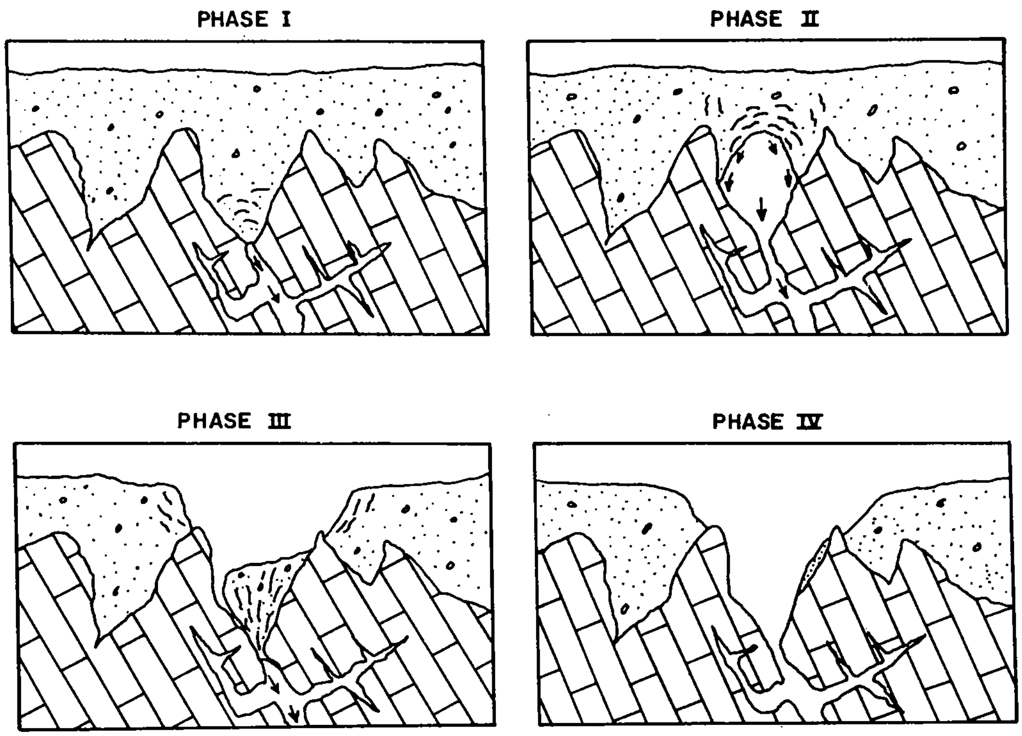


Figure 7. This series of schematic drawings illustrates a hypothetical karst type collapse associated with structure foundations (Note the internal erosion of soil around the piling).

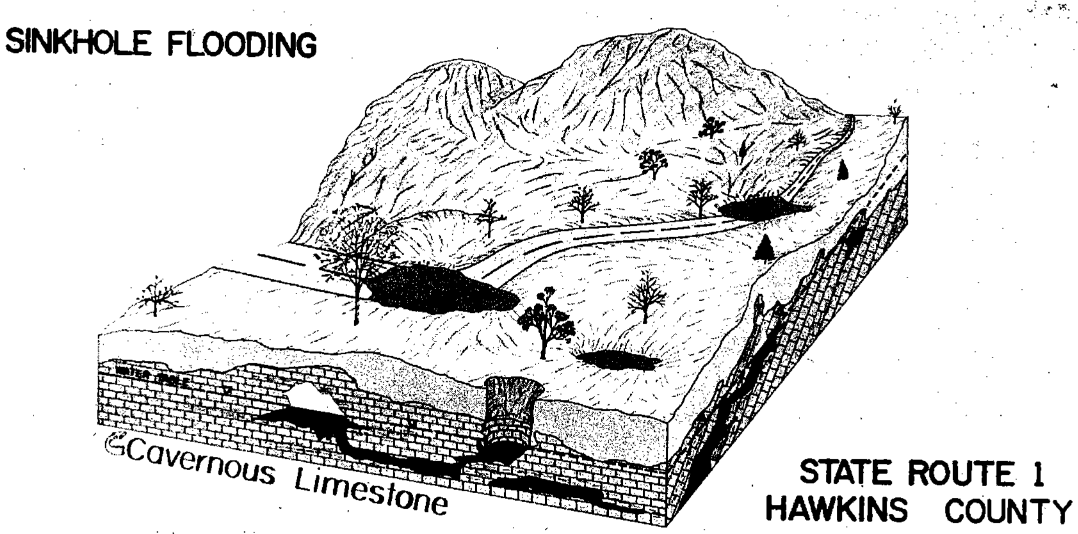


Figure 8. This schematic block diagram illustrates how a section of Tennessee State Route 1 in Hawkins County is periodically flooded by elevated water table conditions in sinkholes and depressions.

greatest potential problems is flooding. Clogged cave entrances or sinkholes and an elevated water table are usually the secondary cause of flooding, the primary one being precipitation.

The sinkholes and depressions that affect highways are variable in size and number. Most depressions and sinkholes are usually 5 acres or less in size but a few have drainage areas of over 50 acres.

Directing surrounding drainage into a nearby cave or sinkhole adjacent to a roadway can lead to serious consequences. If the cave entrance becomes obstructed, surface runoff can quickly back up and flood the adjacent highway. Continued discharging of surface runoff into sinkholes and caves can create extensive subsurface voids, some located beneath highway surfaces, resulting in collapses or dropouts in the roadway.

In summary, there are two main types of karst problems experienced along Tennessee highways. They include first, collapses or dropouts, and second, drainage involving both flooding and collapses. Collapse-type problems include roadway surface collapses, ditchline or relocated channel collapses and structure foundation collapses. Drainage problems usually include flooding of highways constructed through depressions and sinkholes, and directing surface drainage into and away from adjacent depressions and sinkholes.

REMEDIAL MEASURES

Correcting karst-related geotechnical problems often involves the use of innovative ideas that are modified to fit the site conditions. Each karst problem will have a unique cause and solution.

Identifying the cause of a karst problem is as fundamental as correcting the problem itself. To develop proper remedial concepts for karst-related problems, certain key information must be acquired. This might include identifying: (1) source of surface water; (2) direction of subsurface waterflow; (3) attitude of rock strata; (4) water table elevation; and (5) drainage area.

Remedial measures used in correcting karst-related problems may be divided into three areas: bridging, drainage, and relocation. The techniques used in bridging may include rock pads and fills, rock backfill, concrete structures, and grouting. Remedial measures used in drainage might include alteration of the existing drainage by the use of paved ditches, special ditches, plastic overlays, pumps, horizontal drains, and the maintenance of obstructed sinkhole entrances. Although not widely used, relocation of the highway facility around a karst area is a viable corrective measure.

Bridging

The most common type of remedial measure used in Tennessee for karst problems is bridging. A bridging technique, commonly recommended, consists of a limestone rock pad used to span an area of intense solution activity. Such was the case on a section of Tennessee State Route 1 (U. S. 11-W) in Hawkins County where a new four-lane section of State Route 1 was constructed over an area of active sinkholes (fig. 9). A 5 ft. thick rock pad and a special drainage ditch were designed and constructed to "bridge" this karst area (fig. 10). This rock pad provided stability for the highway by allowing ground water to flow freely from the sinkholes to a special channel and box culvert.

The rock pad constructed over this sinkhole area has a

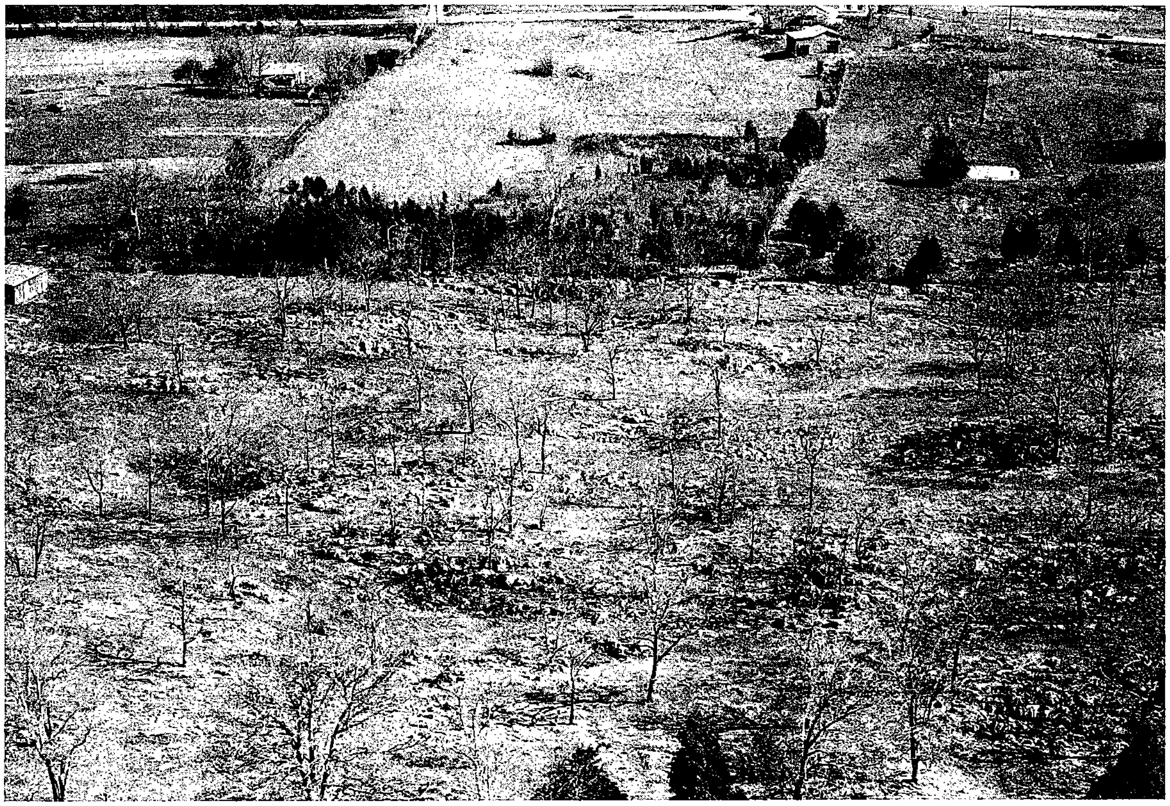


Figure 9. Limestone outcrops and sinkholes characterize portions of the topography through which Tennessee S. R. 1 was constructed in Hawkins County.

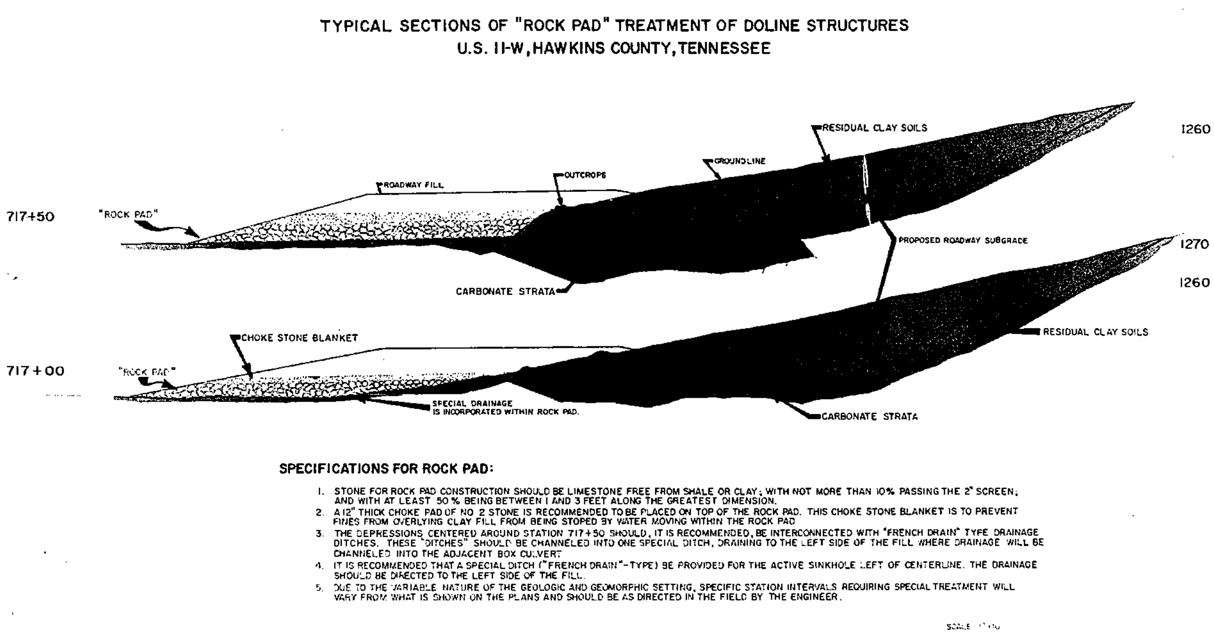


Figure 10. These schematic drawings illustrate the use of free draining rock pads for embankments constructed through karst terrain along a section of Tennessee State Route 1 (US 11-W), Hawkins County.

minimum thickness of 5 ft. and meets the following rock specifications:

> Rock pad material shall consist of sound, non-degradable limestone with a maximum size of 3 ft. and be free of shale and/or clay; at least 50 percent (by volume) of the rock shall be uniformly distributed between 1 foot and 3 ft. in diameter, and no greater than 10 percent (by volume) shall be less than 2 inches in diameter; the rock material shall be roughly equi-dimensional; thin slabby material will not be accepted.

Another type of bridging technique that is widely used for correcting collapses involves the use of a chunk rock backfill. The rock backfill differs from the rock pad material in that no specifications are involved in the rock backfill. Small stone to chunk rock may be used.

The rock backfill concept involves excavation of the failed material. After the collapse is cleaned out, chunk limestone is then backfilled into the excavation. Finally, the chunk backfill is capped with a choker pad of smaller stone to prevent overlying fines from filtering down into the chunk rock. Occasionally, a concrete slurry is used to seal the backfill material.

Rock backfill is widely used in most of Tennessee where limestone is readily available. Although this concept is not a cure-all for collapse type karst problems, rock backfill has proven to be the most cost-effective procedure to date.

Concrete grouting with high-slump slurry-concrete of karst-type collapse as a remedial concept is often considered and sometimes employed. A common type of failure in which cement grouting is used involves the removal of soil fines from around piling due to piping into an underlying solution cavity. One such case involved a bridge pier footing on a section of I-24 at the S. R. 76 interchange in Montgomery County (fig. 11). This section of I-24 was constructed over an extreme karst area of upper Middle Tennessee. On October 1, 1975, after a period of heavy precipitation a collapse structure developed beneath the east pier of the I-24 structure over S. R. 76. Upon investigation, it was discovered that the foundation piles (some on bedrock) provided channels that facilitated percolation of surface water entering the collapse area around the pile cap.

It was decided that a series of deep, cement-grouted holes would help seal off the solution cavities along the soil-rock interface. Several holes were drilled into the soil beneath the pile cap and then pressure grouted with a cement slurry. The cement grout should prevent subsurface erosion above the water table due to water table fluctuation. After the grouting was complete, a rock backfill plug was installed at the base of the pier. Finally, soil was backfilled around the exposed piling and compacted.

In some cases where field conditions warrant, a fourth type of bridging, one involving the use of concrete structures, may be used. This concept may entail the construction of (1) a free spanning highway concrete bridge; (2) a concrete bridge with the deck constructed right on the ground surface (commonly called a "Muck Trestle"); (3) drilled cast-in-place concrete piers; or (4) a concrete slab over the subsurface opening.

An often recommended but as yet untried concept in relation to structure foundations in cavernous strata is the use of drilled cast-in-place piers (caisson). Where soluble limestone strata occur beneath a river channel, numerous cavities exist, some clay filled and some void. In some instances a series of solution

Figure 11. Karst collapse failures around the piling of foundations. The failure pictured above involves a bridge pier footing on a section of I-24 at the S. R. 76 interchange in Montgomery County.

cavities may extend downward to depths of over 100 ft. Excessively deep cavernous strata make excavation prohibitive.

Another technique that may be used in bridging a severe karst problem entails the construction of a ground-level bridge called a "Muck Trestle" (fig. 12). The "Muck Trestle" bridge is currently being considered for a severe karst-related flooding and dropout problem in Hamblen County along State Route 34 near Morristown.

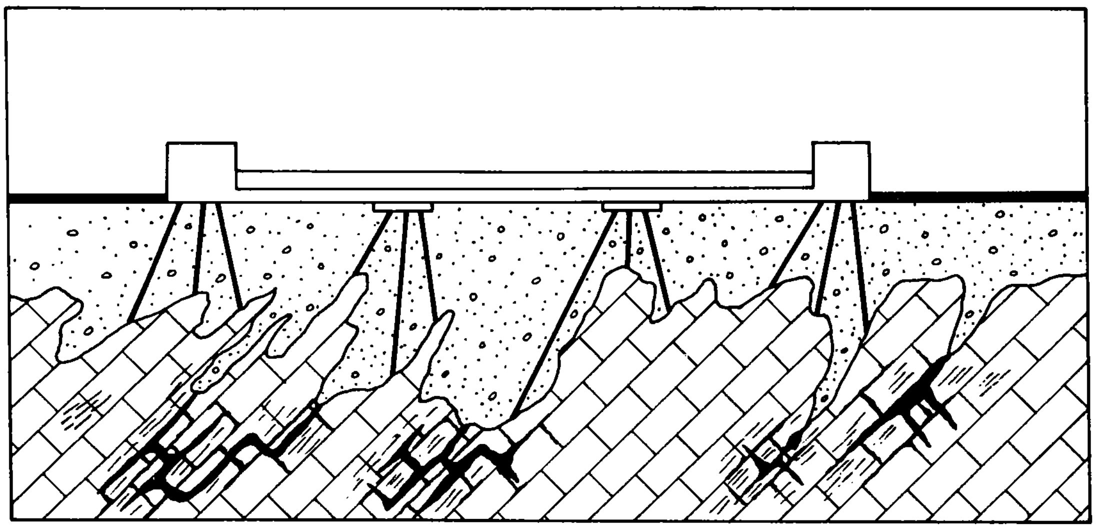
It is envisioned that the "Muck Trestle" would span a 200-foot section of highway plagued with collapse features. It is theorized that when dropouts occur beneath the "Muck Trestle" the concrete deck will span the collapse opening.

Another remedial concept developed for this karst problem in Hamblen County entails the construction of an at-grade aquaduct system spanning a 1 1/2-mile section of the karst area. This would provide positive drainage from the overtaxed karst drainage area to nearby Panther Creek. Either concept is considered costly; estimates range to over one million dollars.

Another bridging technique that has been employed in Tennessee karst areas involves the use of lime stabilization. One such case involved the development of a collapse structure on the shoulder of the westbound lane of I-24 in Montgomery County near Station 363+00 during the spring of 1976. It was found that surface seepage pressures were responsible for this collapse. The soil thickness, the apparent position of the water table, and the topographic expression relative to the roadway geometry led to this situation.

The actual remedy consisted of undercutting 1.2 to 2 m (4 to 6 ft.) of soil through the sink area, treating the base of

"MUCK TRESTLE" BRIDGING CONCEPT



THIS SCHEMATIC DIAGRAM DEPICTS THE USE OF A STEEL REINFORCED CONCRETE STRUCTURE ("MUCK TRESTLE" CONCEPT) TO SPAN AN AREA OF INTENSE SOLUTION ACTIVITY AND COLLAPSE STRUCTURES.

Figure 12. The "Muck Trestle" bridging concept considered for use in spanning a severe collapse problem on S. R. 34 in Hamblen County.

the excavation with 5 percent hydrated lime, backfilling and lime treating two additional 0.6 to 1 m (2 to 3 ft.) lifts of clay soil, crowning the entire sink area with 0.6 to 1.2 m (2 to 4 ft.) of clay soil, re-routing the drainage around the sink, and finally heavily seeding and mulching the entire area. Hydrated lime was used to seal the sink area from surface seepage pressures. This procedure, which used 48 tons of lime and treated three 13,000 $ft.^2$ lifts, was completed in one day. To date the treated area remains stable.

Drainage

The development of proper drainage systems for highways constructed across karst terrain is fundamental in effecting highway stability.

The most common technique used in drainage involves the alteration of an existing drainage system. This may include directing surface runoff into a sinkhole, re-routing runoff away from a sinkhole, or constructing special channels or ditches to do either.

Another common technique of drainage control involves the use of paved ditches. It is estimated that over 90 percent of the collapse type karst problems occur in sodded ditchlines (with a gradient of less than 3 percent). Unpaved ditches provide for increased seepage pressures, which may result in subsurface erosion and finally collapse. During the past few years, geotechnical design data provided to road designers have included such items as paved ditches in karst areas or in areas underlain by geologic formations known to be prone to cavity development. This procedure of treating the problem before-the-fact should help eliminate such karst problems.

Some drainage problems entail the use of plastic polyethylene overlays. This provides an impermeable barrier preventing seepage pressures from developing and sealing the subsurface in that area from direct moisture.

Although the use of self-actuating centrifugal pumps has not been employed in Tennessee, the concept has merit. The main drawback to using pumps to discharge collected runoff along roadways located in depressions involves their maintenance. The possibility of the pumps being neglected or overlooked due to other maintenance projects has been the main issue in not utilizing them.

In several karst areas across the State of Tennessee situations require the continued maintenance of sinkhole cave entrances to prevent flooding. Although this may seem to be troublesome or to indicate poor engineering, there may simply be no other alternative.

Sinkholes clogged with debris such as garbage, car bodies, discarded home appliances, and animal carcasses, are a hazard to the public because they flood the highways and they also pollute the ground-water regime from which drinking water is taken.

A serious situation involving highway embankments, if left untreated, is the water damage generated by the activity of springs. The purpose of any remedial concept dealing with springs is to confine the water and provide an outlet. It has been the Department's experience that using a crushed stone French drain, reinforced with perforated under-drain pipe, is the most cost-effective method of treatment of springs.

One such project involves a part of I-640 in Knox County where a large six-lane embankment, some 19 to 22 m (60-80 ft.) high, was constructed over an area underlain by numerous active springs. These springs yielded flows varying from about 1 to over 10 gallons per minute (gpm). During the initial construction, 11 springs were located within the limits of the embankment, which was 272 m (1,000 ft.) long. A series of French drains was installed from the spring sources to areas that would be outside of the embankment. These trenches were backfilled up to 1.3 to 1.8 m (4 to 5 ft.) in thickness with 5-cm (2-inch) stone (common septic drainfield stone). The trenches were again excavated, but only through a part of the backfill stone so that perforated underdrain pipe could be installed (fig. 13). After the backfill stone was placed over the perforated pipe,embankment construction proceeded. These perforated pipe-reinforced French drains are effectively providing drainage for these springs (fig. 14).

Using drainage as a remedial measure for karst-related problems can be most effective. For some conditions, it is the only concept applicable.

Relocation

A number of highway problems involve karst situations in which the best remedy would be relocation of the roadway facility. However, economy must be considered, and as a result most established highways constructed in karst areas cannot be economically relocated. This is not to rule out the relocation concept. In Tennessee, highway location programs are effectively using geotechnical data for evaluating highway alignments situated in karst areas. As a result, several proposed highway alignments that were initially located in severe karst terrain have been relocated.

As stated earlier, most remedial work on karst problems

Figure 13. Perforated pipe reinforced French drains have been used with great success in draining springs along a part of I-640 in east Knox County.

Figure 14. French drains reinforced with perforated underdrain pipe are used effectively to drain karst areas with numerous springs. Illustrated above is the installation of a French drain which will provide drainage for one of eleven springs located beneath a fill section of I-640 in Knox County.

usually involves the combination of one or more of the techniques or concepts discussed. Bridging, drainage and relocation have been the three methods used by the Tennessee Department of Transportation to correct or prevent karst problems.

SUMMARY

In Tennessee, karst features such as sinkholes, depressions, ponors, uvalas, caves, and fluted-pinnacle-type limestone outcrops are generally found in three physiographic provinces: the Valley and Ridge, Highland Rim, and Nashville Basin. Karst-related geotechnical engineering problems have been experienced in all of the provinces.

Most of the highway-related karst problems experienced in Tennessee have a specific geologic distribution pattern that coincides with the incidence of known caves and sinkholes and/or depressions. Geologic formations that are prone to intense solution activity include the Knox Group and the Monteagle, Bigby-Cannon, Catheys, Bangor, Warsaw, Saint Louis, and Holston Formations.

Karst-related geotechnical engineering problems in the Valley and Ridge of East Tennessee are principally found in either the Knox Group (and equivalent formations) or the Holston Formation. Along the Highland Rim, most karst problems are found in the Bangor, Monteagle, and Warsaw Formations. In the Nashville Basin the Bigby-Cannon, Catheys, Ridley, and Lebanon Formations contain most of the carbonate solution problems experienced in geotechnical engineering.

Most of the karst-type problems experienced along Tennessee highways are grouped into collapse-type failures or drainage-related problems. Collapse problems may involve the roadway surface, ditchlines, structure foundations, or a combination of the above. Drainage problems may include (1) the flooding of sinkholes, depressions, and adjacent highways and residential and commercial areas; (2) the manipulation of drainage into and away from sinkholes and depressions resulting in flooding, collapse features, and the effect on springs and wells; and (3) a combination of drainage and collapse problems.

Remedial measures used by the Tennessee Department of Transportation in correcting these karst problems can be classed into one of the following categories: bridging, drainage, or relocation. Bridging techniques may include the use of rock pads and fills, rock backfill, concrete grouting, concrete structures, and chemical stabilization. Drainage may include the alteration of existing drainage features, the use of paved ditches, plastic overlays, pumps, horizontal drains, maintenance of obstructed sinkhole entrances, and the appropriate treatment of springs. Relocation entails moving a segment of roadway to bypass a karst area. In many cases, combinations of the above techniques are required for satisfactory treatment of karst-related problems.

In Tennessee the Department of Transportation is effectively using geotechnical personnel to develop innovative remedies to karst problems. In the past, geotechnical input into karst-related highway problems was on an after-the-fact basis. At present, geotechnical expertise is being utilized at the programming, location, and design phases as well as in the construction and maintenance phases. This before-the-fact involvement in identifying and controlling karst-related problems has heightened the effectiveness of highway engineering in Tennessee.

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IMPACT OF EVAPORITE DISSOLUTION AND COLLAPSE ON HIGHWAYS AND OTHER CULTURAL FEATURES IN THE TEXAS PANHANDLE AND EASTERN NEW MEXICO[1]

by

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INTRODUCTION

Thick sequences of Permian evaporites that are both highly soluble (halite) and moderately soluble (anhydrite and gypsum) compose a significant part of the sedimentary fill in the Permian Basin of the southwestern United States. The Bureau of Economic Geology, The University of Texas at Austin, has been studying evaporite dissolution in the Texas Panhandle since 1977 as part of a comprehensive analysis of Permian salt as a potential site for isolation of nuclear wastes. As part of this study, Gustavson and others (1980b) have determined that pre-Holocene and Holocene dissolution of Permian bedded salts of the Palo Duro, Dalhart, and Anadarko Basins is a major factor in the development of the Texas Panhandle landscape. Sinkholes, collapse depressions, fractures, and faults are common surface manifestations of subsurface dissolution.

This process of dissolution and collapse directly affects highways that traverse parts of the Texas Panhandle and eastern New Mexico. This first became

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evident to us when investigating the faults along FM 2639 in Hall County, Texas. Further investigations employing questionnaires and field work confirm highway damage due to subsidence and faulting over the zone of salt dissolution. Large reservoirs and stock tanks are also affected to a lesser degree.

Highways occupy only a small part of the total land surface in the Texas Panhandle and eastern New Mexico. The number of collapse features and faults observed near and on highway right-of-ways suggests that they are widespread in fenced, privately owned land not traversed by highways. Thus, the phenomena described in this paper are only a small fraction of the total number in the region.

Major physiographic features in the study area include the Northern and Southern High Plains surfaces (Llano Estacado), the Caprock Escarpment, and the Rolling Plains and Pecos Plains to the east and west, respectively (fig. 1). The High Plains is divided into north and south sections by the valley of the Canadian River, termed the Canadian Breaks. The High Plains surface is developed on the late Tertiary Ogallala Formation, the remnants of a large alluvial plain that originated in the Sangre de Cristo Mountains and Pedernal Hills in New Mexico and spread eastward across the Texas Panhandle (Seni, 1980). The Ogallala Formation is now overlain in most areas by eolian cover sand. The High Plains surface and underlying Ogallala Formation and Dockum Group are truncated by the Caprock Escarpment, an erosional scarp which in Texas has retreated westward to its present position since Tertiary time. A similar escarpment, the southern part of which is known as the Mescalero Ridge, forms the western rim of the High Plains in eastern New Mexico. The Caprock Escarpment is supported by massive caliche zones within the uppermost part of the Ogallala Formation and to some extent by indurated sandstones in the upper part of the Triassic Dockum Group. Vertical relief on the escarpment locally exceeds 300 m (1,000 ft). East of the

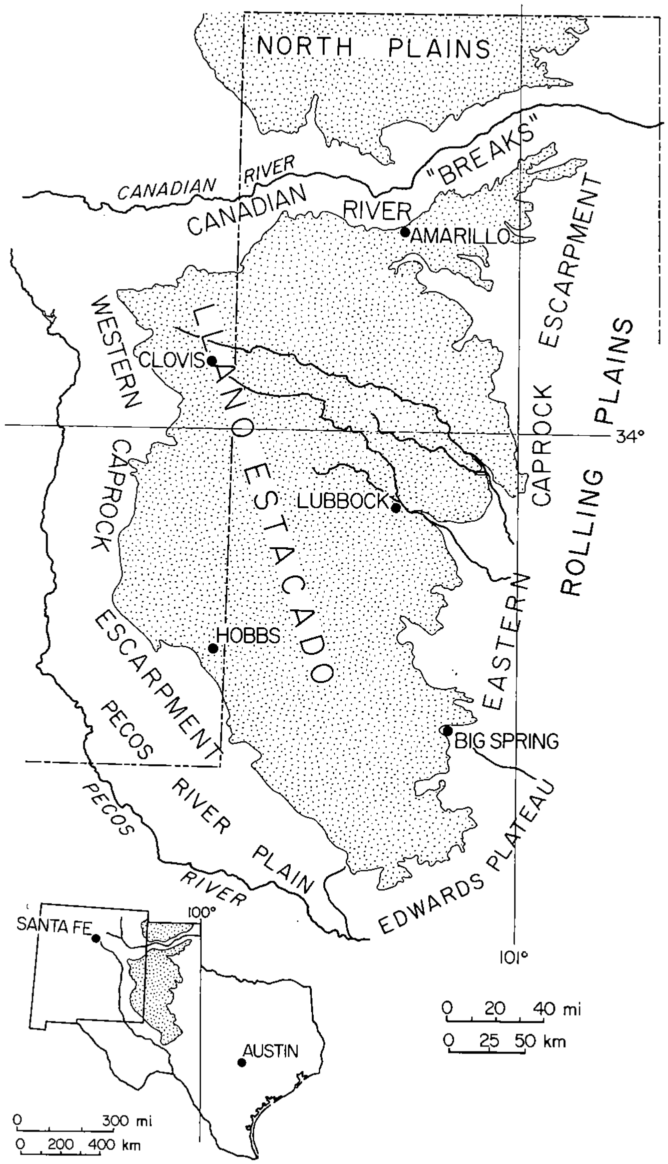

Figure 1. Physiographic units of the Texas Panhandle and eastern New Mexico (from Gustavson and others, 1980b).

Caprock Escarpment lies the Rolling Plains, a gently rolling surface developed on structurally disturbed Permian red beds. The Pecos River Plains lies west of the Caprock Escarpment in New Mexico, and the Edwards Plateau merges with the High Plains surface to the south.

GEOLOGIC SETTING

Major tectonic elements of the Texas Panhandle, eastern New Mexico, and western Oklahoma (fig. 2) have been discussed elsewhere by Nicholson (1960) and Johnson (1976). These elements, the Palo Duro, Dalhart, and Anadarko Basins, the Amarillo-Wichita uplifts, Matador Arch, and Oldham Nose, were tectonically active from late Mississippian to mid-Permian time. Fault displacements since late Permian have been identified, but the total extent of post-Permian displacements is unknown (A. Goldstein, personal communication, 1980).

Salt, gypsum, anhydrite, mudstone, sandstone, dolomite, and limestone make up the Permian strata in the Palo Duro, Anadarko, and Dalhart Basins (Dutton and others, 1979). Apart from areas of dissolution, evaporites compose 50 to 75 percent of the Upper Permian section in the basins (M. Presley, personal communication, 1980). This percentage of evaporites decreases toward basin margins because of increased terrigenous sediment and extensive evaporite dissolution. In the Panhandle, evaporite and associated carbonate rocks display facies that grade from supratidal environments in the north to subtidal environments in the south. From north to south lithofacies include upper sabkha salts, lower sabkha anhydrites, supratidal to subtidal dolomites, and subtidal carbonates. Red beds occur as sheets of basin-edge mudstones and fine-grained sandstones that intertongue basinward with dolomite-evaporites (Dutton and others, 1979).

There are seven salt-bearing units within the Anadarko, Dalhart, and Palo Duro Basins. These are the Salado, Seven Rivers, upper and lower San Andres,

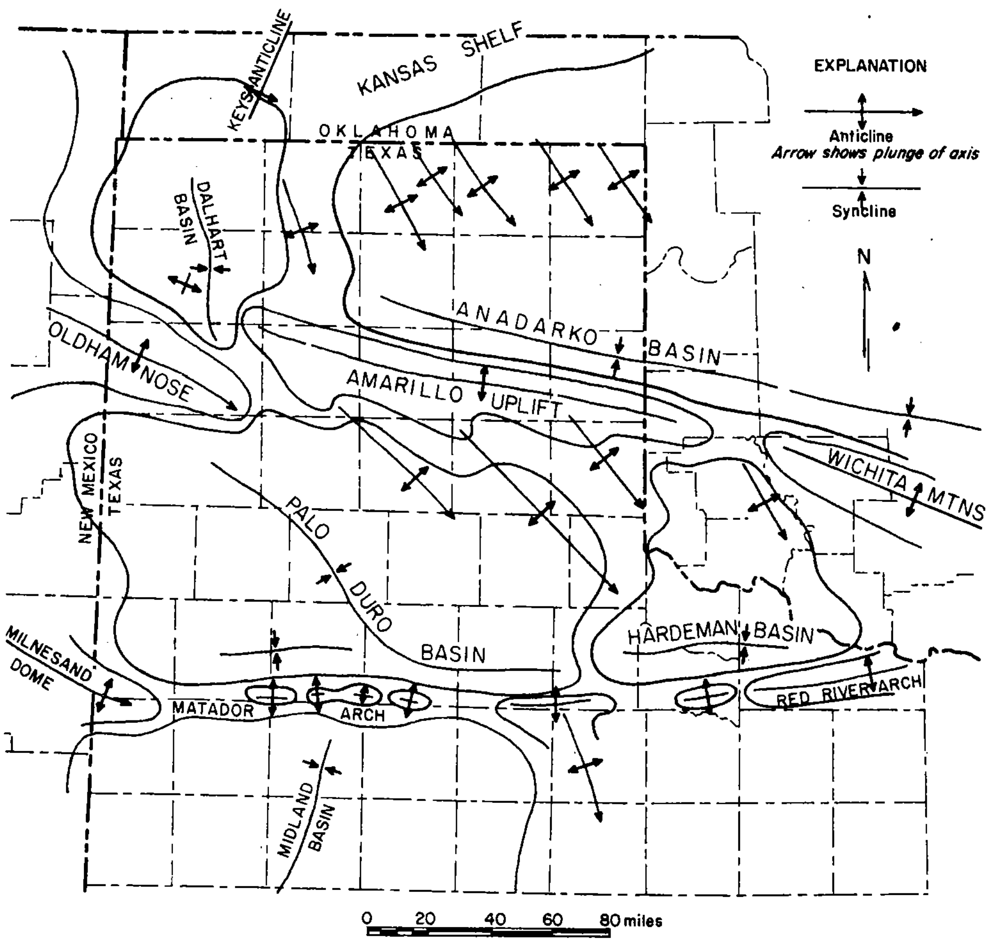

Figure 2. Major structural elements of the Texas Panhandle (after Nicholson, 1960).

Glorieta, and upper and lower Clear Fork Formations, which lie at depths between 150 and 757 m (500 and 2,500 ft). Stratigraphic names of some of these units change from basin to basin (fig. 3). Except for the lower Clear Fork Formation, all of the salt units are undergoing regional dissolution. Stratigraphically higher (shallower) salt beds are dissolved progressively farther toward the center of the Palo Duro Basin (fig. 4). Relatively fresh, undersaturated ground water migrating down to the salt beds is the agent of dissolution; thus, the shallowest salt beds have been the most affected by the dissolution process. Anhydrite, hydrated to gypsum near the land surface, is also undergoing dissolution, but apparently gypsum dissolution occurs only at or near the surface (Gustavson and others, 1980b).

PREVIOUS WORK

Subsurface dissolution of Permian bedded salt has been recognized since Johnson (1901) first suggested that this process accounts for the structure of the Meade Basin in southwestern Kansas. Lee (1923) and Morgan (1941) suggested that the present course of the Pecos River in eastern New Mexico is a result of the interconnection of solution troughs along that course. Fiedler and Nye (1933) described solution phenomena in the Roswell Artesian Basin. Adams (1963), Jordan and Vosberg (1963), Brown (1967), Hills (1968), Bachman and Johnson (1973), and Johnson (1976) have all attributed the thinning of subsurface evaporite sequences to the dissolution process.

Surface manifestations of subsurface dissolution have been described in eastern New Mexico and the Texas Panhandle. Kelley (1972) and Sweeting (1972) discussed the geologic setting of Santa Rosa, New Mexico, which occurs in a large collapse sink. Jones (1973) reported several depressions in the Los Medanos area of southeast New Mexico, including the San Simon Swale, Nash Draw, and Clayton

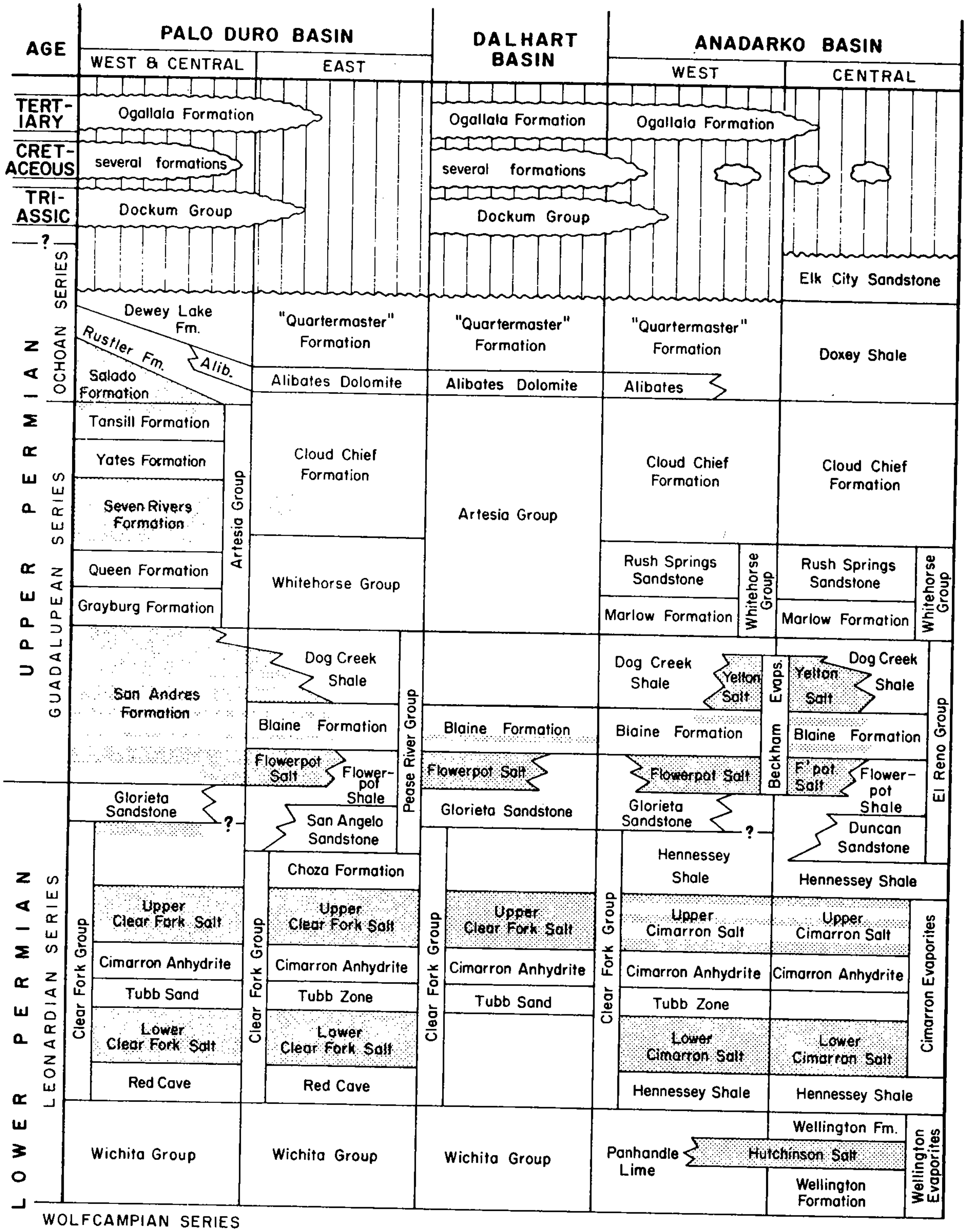

Figure 3. Stratigraphic nomenclature of Permian and younger strata in the Texas Panhandle and western Oklahoma. Principal salt units are shown in gray. Modified from data in Fay (1965), Jordan and Vosburg (1963), McKee and others (1967a, b), and Tait and others (1962) (from Johnson, 1976).

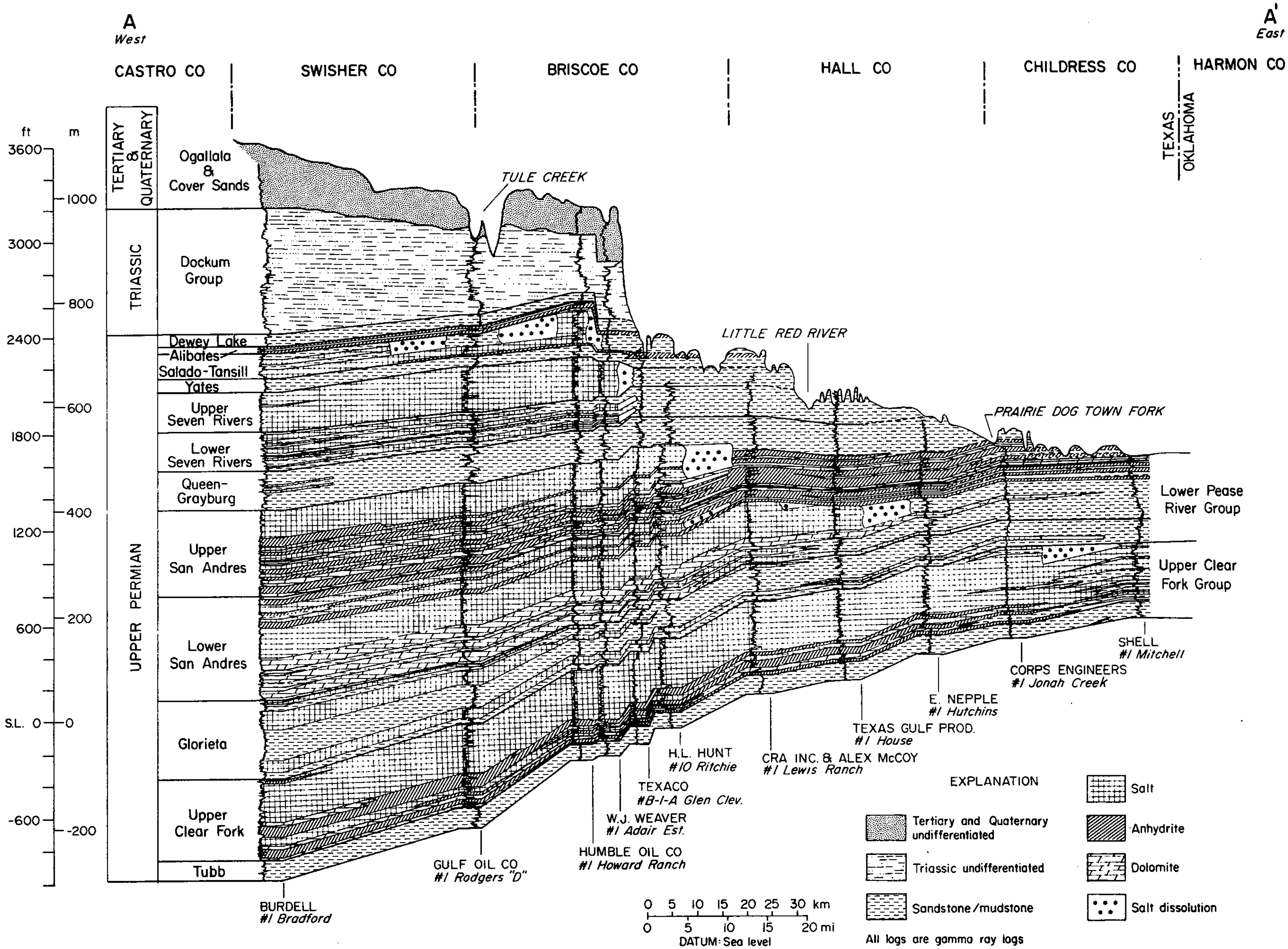

Figure 4. Zones of salt dissolution and stratigraphic units as interpreted from gamma-ray logs. Cross section extends from Castro County in Texas eastward into Oklahoma. Line of section is shown in figure 6 (from Gustavson and others, 1980a).

Basin. He attributed these depressions to the dissolution of salt, gypsum, and anhydrite in the subsurface. Vine (1976) described breccia pipes ascribed to salt dissolution in the same region. Anderson (1978) documented geomorphic features such as domes, sinkholes, and depressions in southeastern New Mexico, and attributed them to dissolution of salt and other soluble minerals within the (Upper Permian) Ochoan Series.

In the Texas Panhandle, dissolution of Permian salt beds within the Palo Duro, Dalhart, and Anadarko Basins has been discussed by Johnson (1976), Dutton and others (1979), Gustavson and Finley (1979), and Gustavson and others (1978, 1979, 1980a, 1980b, 1980c). Karst features attributed mainly to gypsum dissolution have been studied by Miotke (1969), Smith (1969), and Baker (1977).

SURFACE DISSOLUTION FEATURES AND EFFECTS

Extensive Holocene karstification related to evaporite dissolution has been documented in Hall County, Texas, by Gustavson and others (1980a), who used several vintages of aerial photography to identify over 400 total sinkholes and collapse depressions and 11 faults or fractures. Thirty-six sinkholes and two depressions formed between 1940 and 1979 within a 307 km^2 (120 mi^2) test area defined by the availability of a sequence of aerial photography during that time period. These sinkholes are generally circular to oval in plan view, and may be 100 m (330 ft) in diameter and 15 m (50 ft) deep. Sinkholes have vertical walls when formed (fig. 5), but by mass wasting and slope processes the vertical walls degrade to form a more stable, gentler slope. The stabilizing process is somewhat accelerated if the sinkhole is filled with water. Collapse depressions are broad, shallow, internally drained depressions which, in contrast to sinkholes, have no steep vertical sides and are generally larger and more oval in plan view. Lengths up to 2.4 km (1.5 mi) have been observed in the study area. Fractures

Figure 5. Sinkhole located near U.S. Highway 287, 6.4 km (4 mi) northwest of Estelline in Hall County, Texas. The feature is approximately 10.6 m (35 ft) wide and 7.5 m (25 ft) deep.

and faults resulting from dissolution are recognized from aerial photographs as offsets in highways (faults) and open fractures in cultivated fields. All fractures and faults identified in Hall County trend between N 25° E and N 50° E.

To determine the extent of collapse features, fractures, and faults in other areas of the Panhandle region, a questionnaire was sent to each county Soil Conservation Service (SCS) and Agricultural Stabilization and Conservation Service (ASCS) office in the Texas Panhandle and eastern New Mexico. Conversations with representatives of these agencies in Hall County, Texas, revealed their thorough knowledge of sinkholes, collapse depressions, fractures, and faults in that county. We hoped that in other counties these agencies would also be familiar with such features. Of 54 counties in the study region, there was at least one response by representatives of 98 percent of the counties and 2 responses were returned by representatives of 70 percent of the counties. In 37 percent of the reporting counties, SCS and ASCS agents knew of sinkholes, collapse depressions, fractures, and/or faults.

Effect on Highways

For those counties in which collapse features, fractures, and faults were recognized by SCS and ASCS personnel (37 percent of total counties), additional questionnaires were sent to representatives of the Texas Department of Highways and Public Transportation and the New Mexico State Highway Department. The purpose of the second questionnaire was to determine if the development of collapse features, fractures, and faults was a recognized problem among those who construct and maintain highways. Results of that questionnaire, shown diagrammatically in figure 6, demonstrate that such features are recognized by highway personnel in 38 percent of those counties which responded positively to the first questionnaire sent to SCS and ASCS personnel. The responses from highway person-

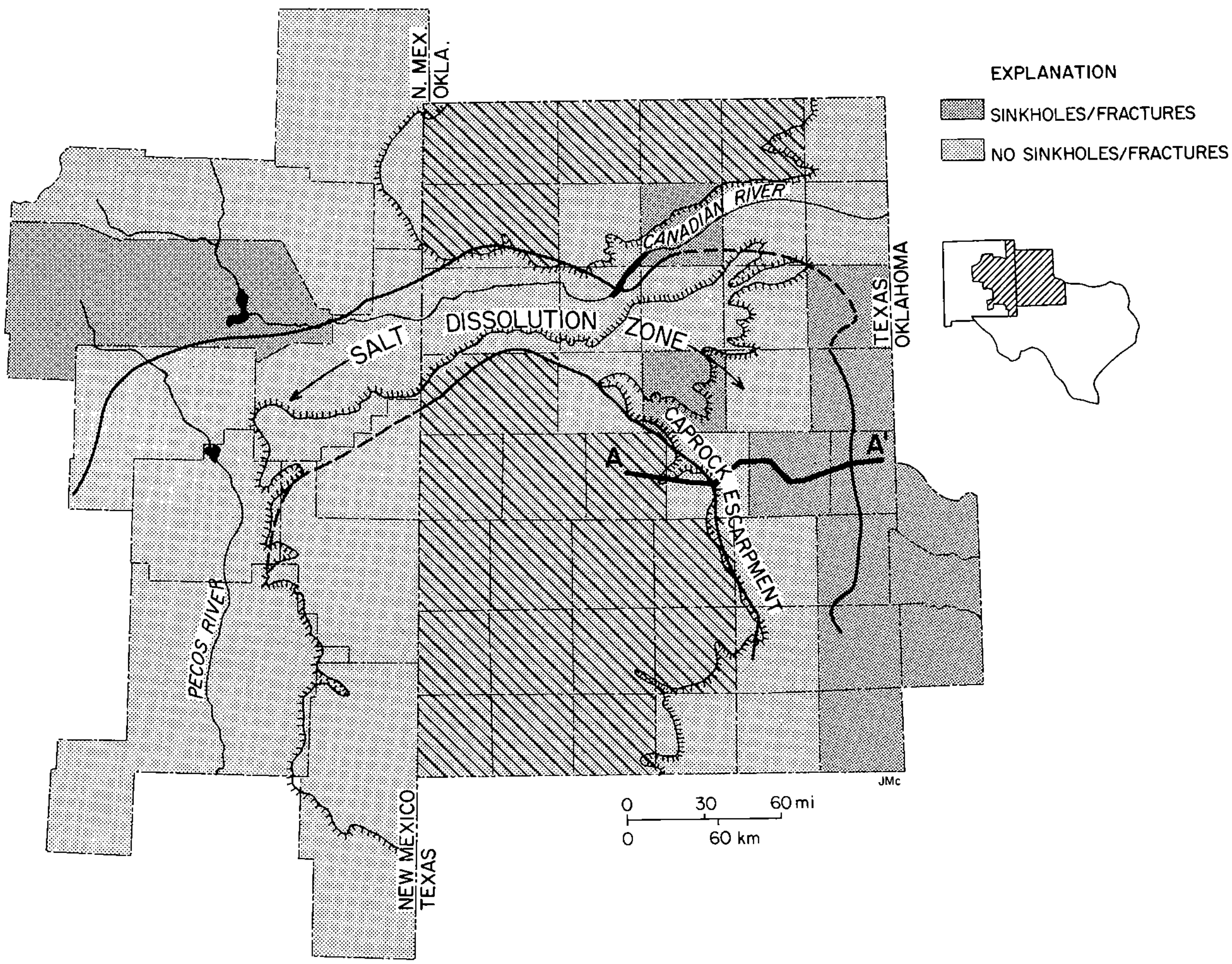

Figure 6. Distribution of collapse features, fractures, and faults that affect highways in the Texas Panhandle and eastern New Mexico, as recognized by highway personnel responding to the second questionnaire. Hachured areas are those counties whose SCS and ASCS representatives did not recognize collapse features, fractures, and faults in the first questionnaire. Correspondingly, these counties were not sent the second questionnaire dealing solely with impacts of collapse features on highways. Salt dissolution lines in Texas after Gustavson and others (1980a) and salt limits in New Mexico after Foster and others (1972). Line of section A-A' refers to figure 4.

nel were particularly significant because highways cover only a small part of the total area, but collapse features, fractures, and faults commonly were recognized and conceived of as a significant problem in maintaining the structural integrity of the highways. Counties in which highway personnel reported such features all lie within the zone of active salt dissolution (fig. 6) and within outcrop belts of gypsum beds of the Permian Blaine Formation (see Barnes, 1967; 1968).

Areas in Texas were subsequently field checked with representatives of the Texas Department of Highways and Public Transportation. Areas in New Mexico were not field checked because of limited positive responses to the questionnaire. Field checking verified several sites where sinkholes, collapse depressions, and faults have affected highways (fig. 7).

In July 1979, in the northern part of the study area, a sinkhole formed beneath part of U.S. Highway 83 just north of the Interstate 40 overpass in Shamrock, Texas (fig. 7; site 2). The sinkhole opened to a depth of 2.4 m (8 ft) and a diameter of 3.0 to 3.6 m (10 to 12 ft). City water and sewer lines were damaged and part of the northbound line of U.S. Highway 83 was closed. Gypsum beds of the Blaine Formation are exposed in roadcuts throughout the area, but whether gypsum dissolution is the sole cause of this sinkhole is not known. A possible collapse depression occurs approximately 4 km (2.5 mi) west of Shamrock on Interstate 40 (fig. 7; site 3). At the point where Interstate 40 passes over Finley Creek, the pavement is depressed about 5 cm (2 inches) over a distance of approximately 9.1 m (30 ft), and the swale has developed since the highway was built in June 1968. Evidence of previous subsidence at this site is shown by the badly faulted old road (formerly U.S. Route 66), which now parallels Interstate 40 on the south. The old faulted road and the depressed sections of Interstate 40 are generally aligned in a northeasterly direction. A sinkhole

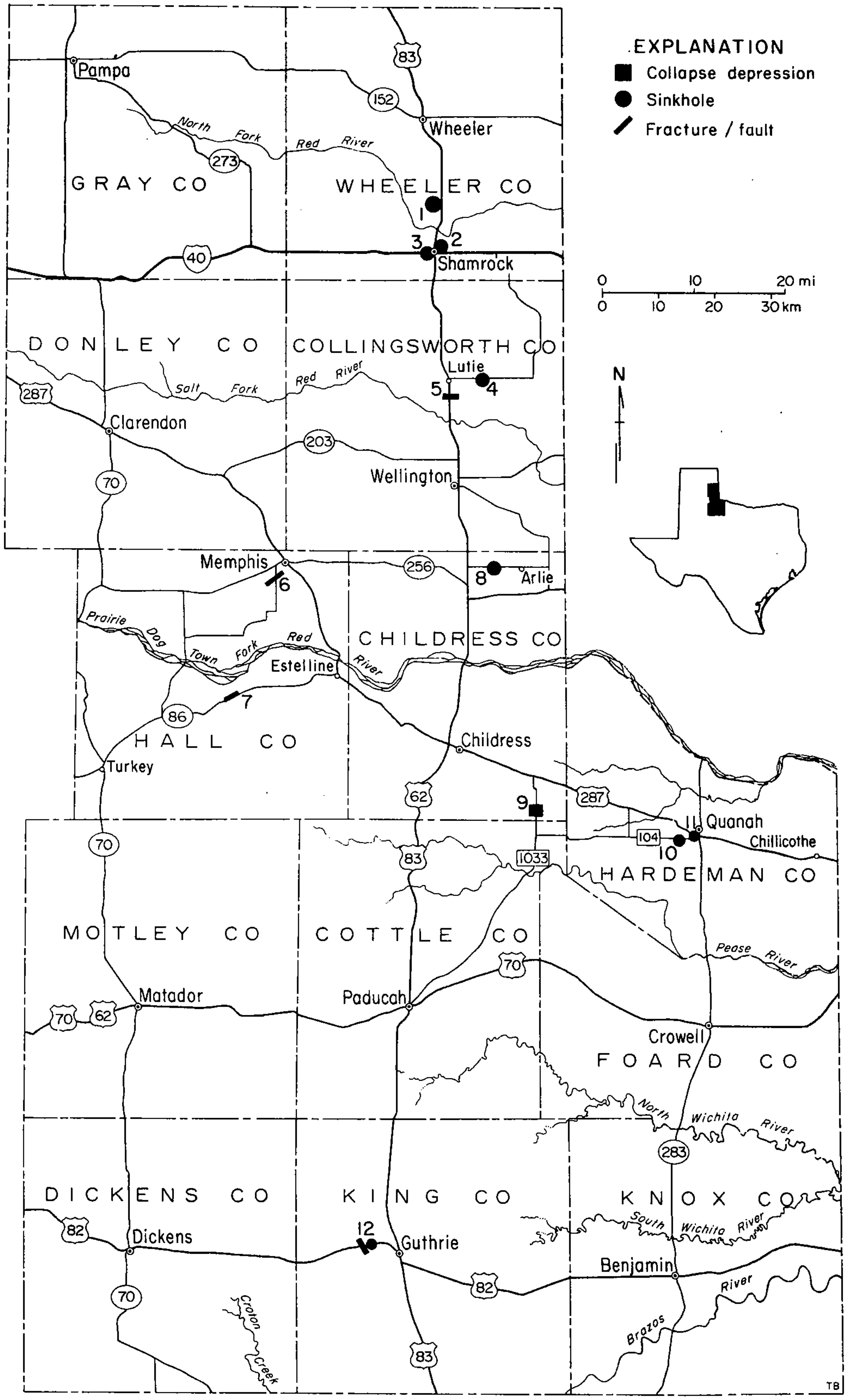

Figure 7. Locations of collapse features, fractures, and faults that affect highways within the study area.

measuring 3.3 m (10 ft) in diameter occurs in a roadcut immediately west of this site, which further confirms that the processes of dissolution and collapse are active in this area.

In the central part of the study area, a small collapse depression was identified on FM 1033 south of Kirkland, Texas, 1.6 km (1.0 mi) north of the Childress-Cottle county line (fig. 7; site 9). Diameter of the feature is about 6 m (20 ft); about half of the depression affects the pavement surface. Several generations of road patches attest to the fact that collapse has occurred over several years. Repairs to this section of the highway have been made regularly since 1966. Within the collapse depression, a smaller sinkhole in the highway borrow ditch collapses periodically and must be filled (fig. 8). The effect on the highway itself, however, is mainly from the collapse depression.

A small-scale drilling operation was initiated at this site near Kirkland to determine the depth of subsidence and whether this feature was due to shallow gypsum dissolution. Drilling indicated that the center of the sinkhole contained moist, silty clay (highway fill) to a depth of 4.5 m (15 ft), at which point Permian bedrock was encountered. The second hole about 6 m (20 ft) away contained about 2.7 m (9 ft) of dry, sandy alluvial sediment overlying Permian bedrock. From the depth of fill in the center of the sinkhole, it appears that a minimum of 1.8 m (6 ft) and maximum of 4.5 m (15 ft) of total subsidence has occurred. More significant is the fact that no gypsum was encountered in either hole. This suggests that shallow gypsum dissolution is not responsible for this feature, at least not in the vicinity of the boreholes. The collapse depression occurs in strata that are stratigraphically below a major gypsum unit (McQueen Gypsum Bed) of the Blaine Formation (Barnes, 1968). Dissolution of deeper gypsum or salt is probably the cause of this karst feature.

Figure 8. View south along FM 1033 in Childress County showing collapse depression. The depression affects most of the northbound lane and extends into the borrow ditch. Different shades of road patches at this site attest to the history of subsidence.

Numerous smaller sinkholes, commonly called "gyp sinks" by highway personnel, are recognized in borrow ditches throughout the study area (fig. 7; sites 1, 4, 8, 10, 11, and 12). Many sinkholes exist in outcropping gypsum bedrock (fig. 9). Somewhat larger sinkholes in gypsum bedrock occur along Texas Highway 114/U.S. Highway 82, 4.8 km (3 mi) west of Guthrie, Texas (fig. 7; site 12). Two holes with depths between 5.2 and 6.4 m (17 and 21 ft) have developed there within the last ten years. A third sinkhole began as a very small hole in the borrow ditch and eventually expanded under the highway along a N 30° W trend. Subsidence was sufficiently rapid to require repairs at least twice a week for the first year and a half after collapse began. A linear trend like that displayed by this sinkhole is unusual. In this case the trend may actually comprise a much larger sinkhole that collapsed first where the surface was not supported by highway pavement.

Another significant impact of dissolution and collapse on highways is faulting. The best example of faulting occurs along FM 2639, 19 km (12 mi) west of Estelline in Hall County, Texas (fig. 7; site 7). Here, six faults trending N 25° E to N 50° E were recognized by displacement along the faults measured from 1 to 4 cm (0.4 to 1.6 inches). A prominent patched fault is shown in figure 10. Although only a few inches wide when formed, the fault plane was visible to a depth of 1.5 m (5 ft). Repairs have been necessary on the road about three times per year since the fault appeared in 1979. Similar faults have been recognized near Memphis, Texas (fig. 7; site 6), and 9.6 km (6 mi) south of Lutie, Texas (fig. 7; site 5). The Lutie fault plane was visible to a depth of nearly 0.9 m (3 ft) and the fault trace trended almost east to west across the northbound lane of U.S. Highway 83. It has since been filled and has not recurred.

Although natural evaporite dissolution contributes most to collapse and faulting, failure may be aggravated by highway construction practices and high-

Figure 9. Sinkhole in the borrow ditch adjacent to FM 1034 west of Arlie, Texas. The sinkhole is approximately 0.9 m (3 ft) deep and 1.2 m (4 ft) wide. Sinkholes of this type are common along highways in the Texas Panhandle.

Figure 10. Fault trending northeast across FM 2639 in Hall County. Left side of fault is depressed approximately 3 to 4 cm (1.5 to 1.8 inches) relative to the right side. Fault extends into the cultivated field.

way traffic. Pre-road weaknesses caused by salt and anhydrite/gypsum dissolution may be aggravated by use of heavy construction machinery and construction of borrow ditch systems. Borrow ditches may concentrate water along preexisting fractures in near-surface gypsum and promote further evaporite dissolution. Subsequent automobile and truck traffic may promote further fracturing of gypsum and compaction of the unconsolidated roadbed fill and alluvium which may fill irregularities on the gypsum surface. All these processes contribute to continuing subsidence in the immediate vicinity of the highway. However, in investigating individual cases of collapse, human impact is difficult to separate from natural processes of evaporite dissolution and collapse.

Engineering Problems

In addition to highway construction and maintenance problems, other engineering problems resulting from evaporite dissolution were noted during construction projects in the Texas Panhandle and eastern New Mexico. Eck and Redfield (1963) and Bock and Crane (1963) identified 27 collapse chimneys during excavations for Sanford Dam on Lake Meredith, 64 km (40 mi) northeast of Amarillo, Texas. Collapse chimneys generally are circular to elliptical in cross section and are typically filled with slumped and brecciated sediments from the overlying Triassic Dockum Group, Ogallala Formation, or Canadian River terraces. The largest collapse chimney exposed at the Sanford Dam site is approximately 305 m (1,000 ft) in diameter. Origin of chimneys is attributed to collapse due to regional dissolution of Permian bedded salts (Gustavson and others, 1980b).

Eck and Redfield (1965) considered the problem of reservoir leakage through collapse chimneys to be a serious geologic problem. Pressure tests within the chimneys and adjacent formations indicated low permeability, and examination of drill cores also indicated that most fractures were filled with silty clay and gypsum. No mention was made of the possible impacts of future evaporite dissolution and surface collapse at the dam site. McDowell (1972) and Spiegel (1972) investigated structurally disturbed Permian strata at the Los Esteros Dam site, north of Santa Rosa, New Mexico, which is within the zone of active salt dissolution.

It would be prudent to assess the potential impacts of evaporite dissolution and subsequent collapse of overlying strata during the planning of large reservoirs and other projects in the two-state region. Clearly, development of large sinkholes similar to the collapse chimneys at Sanford Dam could have a catastrophic effect on a dam structure. Placement of a large reservoir over zones of active dissolution might initiate further dissolution, possibly by rerouting ground-water flow across the soluble beds or by adding more fresh, undersaturated ground water to the system.

Although no problems associated with salt dissolution have been reported for large reservoirs, gypsum dissolution is a problem where gypsum rock crops out on the floor of the reservoir. A classic example of this is McMillan Reservoir on the Pecos River north of Carlsbad, New Mexico (Esmiol, 1957). Since its construction in 1893, the dam has continually cracked, subsided, and lost water along underground channels (Brune, 1965). To alleviate these conditions, as well as to solve the problem of sedimentation which has occurred behind the dam since construction, the Brantley Reservoir has been proposed to replace the original reservoir (Redfield, 1967). Although the sedimentation problem may be solved by the new structure, recent preliminary hydrogeochemical studies by the U.S. Geological Survey indicate that the bed of the reservoir will still contain a high percentage of gypsum and that dissolution of gypsum will add significantly to the downstream solute load of the Pecos River (H. Claassen, personal communication, 1979).

Brune (1965) has noted that along the outcrop of the Blaine gypsum in Childress, Cottle, and King Counties (see Barnes, 1967; 1968, for outcrop location) in the Texas Panhandle an unusually large number of stock tanks (farm ponds) lose large amounts of water because of fractures and sinkholes in surface and near-surface gypsum. Stock tank failure also occurs in Collingsworth and Stonewall Counties, according to ASCS and SCS representatives. The Soil Conservation Service now requires preliminary coring at proposed stock tank sites to determine local extent of gypsum, and subsequent construction of either a positive cutoff to bedrock or a mud blanket on the bottom of the reservoir to help prevent water loss (Brune, 1965).

Evaporite dissolution related to human activity (such as solution mining) is not well documented in Texas or New Mexico but it does occur. On July 25, 1978, a sinkhole opened near an abandoned brine well owned by Phillips Petroleum Corporation in the vicinity of Borger, Hutchinson County, Texas (Borger News-Herald, 1978). The hole grew to 33 m (100 ft) in diameter directly adjacent to a Phillips Petroleum Corporation tankfarm. The site is about 45.5 m (150 ft) from U.S. Highway 270. Collapse of a large sinkhole near Wink in Winkler County, Texas, in June 1980 probably resulted from salt dissolution caused by natural and human activities in the region (Baumgardner and others, 1980).

RESPONSE TO THE PROBLEM

Dissolution and collapse pose difficulties for geologists, highway engineers, and maintenance crews. Areas of active subsurface evaporite dissolution have been identified, but development of collapse features and faults at the surface in those areas generally follows no predictable pattern. The trend of faults caused by dissolution along FM 2639 in Hall County is, however, consistent with the lineament trend analyses in the region (Dutton and others, 1979).

In any case, attempts to control or prevent damage to highways and other structures over the long term have been ineffective. Sinkholes, collapse depressions, and faults are dealt with on a regular basis with short-term remedial measures. Sinkholes in borrow ditches are generally filled with sand or similar fill material. Depressions in an asphalt pavement are filled with asphalt mix, bladed with a road grader, and rolled out. On rigid concrete pavements such as Interstate 40, holes are drilled through the pavement surface and grout (normally soil and cement mixed into a slurry) is forced through tubes under the depressed part of the highway until the pavement is forced upward to its original level. This process is commonly known as "mud-jacking." Faults in the pavement are generally treated by filling with sand or other fill and patching with asphalt mix. Smaller tension cracks or faults may be mitigated to some degree by the use of a more expensive rubberized asphalt, which would allow the pavement to stretch before cracking.

SUMMARY

Dissolution of Permian evaporites and collapse of overlying strata are major processes that alter the landscape in parts of the Texas Panhandle and eastern New Mexico. Sinkholes, collapse depressions, fractures, and faults are the common surface expressions of these phenomena. These features were first recognized in Hall County, Texas. Responses to questionnaires sent to representatives of the Soil Conservation Service and Agricultural Stabilization and Conservation Service in the region indicate that at least 37 percent of the counties in the Texas Panhandle and eastern New Mexico contain similar features.

Evaporite dissolution has a significant effect on highways, reservoirs, and stock tanks. Highways have sustained the most reported damage to date, with several sections of roads having been patched or filled on a regular basis

because of faulting or subsidence. Collapse chimneys, which were identified during excavations for the Sanford Dam, are evidence of past evaporite dissolution. Small stock tanks frequently lose water to fractures and solution cavities within surface and near-surface gypsum beds.

Long-term methods to predict and thus mitigate the effects of collapse have been unsuccessful. The problem, however, is of large scope and potential impact. We suggest that in future construction of highways, reservoirs, stock tanks, and other structures, care should be taken to investigate the history of, and potential for, evaporite dissolution beneath the construction site. Areas with high densities of collapse features, fractures, and faults should be avoided when possible.

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MICROEARTHQUAKE STUDIES IN TEXAS

by

J. W. Sansom, Jr., and D. H. Shurbet

Texas has generally been considered aseismic, but in 1969 Shurbet (1969) reported that 20 earthquakes had been located in Texas from 1882 to 1969. Nine of those earthquakes occurred between 1957 and 1969. Two additional earthquakes have occurred since 1969. Therefore, a total of 22 earthquakes have been located in the past 98 years, and 11 of these have occurred since 1957. These later observations probably do not represent increased seismic activity, but rather, an increase in instrumentation capable of detecting seismic events.

The above numbers include only earthquakes large enough to be felt by local people and/or recorded at several permanent seismograph stations. It is well known that hundreds of small earthquakes have occurred in Texas within the last few years and that most, if not all, of these were man-made. There is an increasing awareness of earthquakes and their potential hazard to man-made structures such as bridges, buildings, and large reservoir impoundments. Texas is more seismically active than it had previously been thought to be, and designers of large structures have a particular need to know the probable risks associated with seismic activity in their areas of concern.

In recent years there have been reports of both small and large earthquakes following the filling of new reservoirs in regions where there had been no previous earthquakes documented. For example, in 1962 more than eight earthquakes followed the filling of Kogna Reservoir in India, an area not known to be

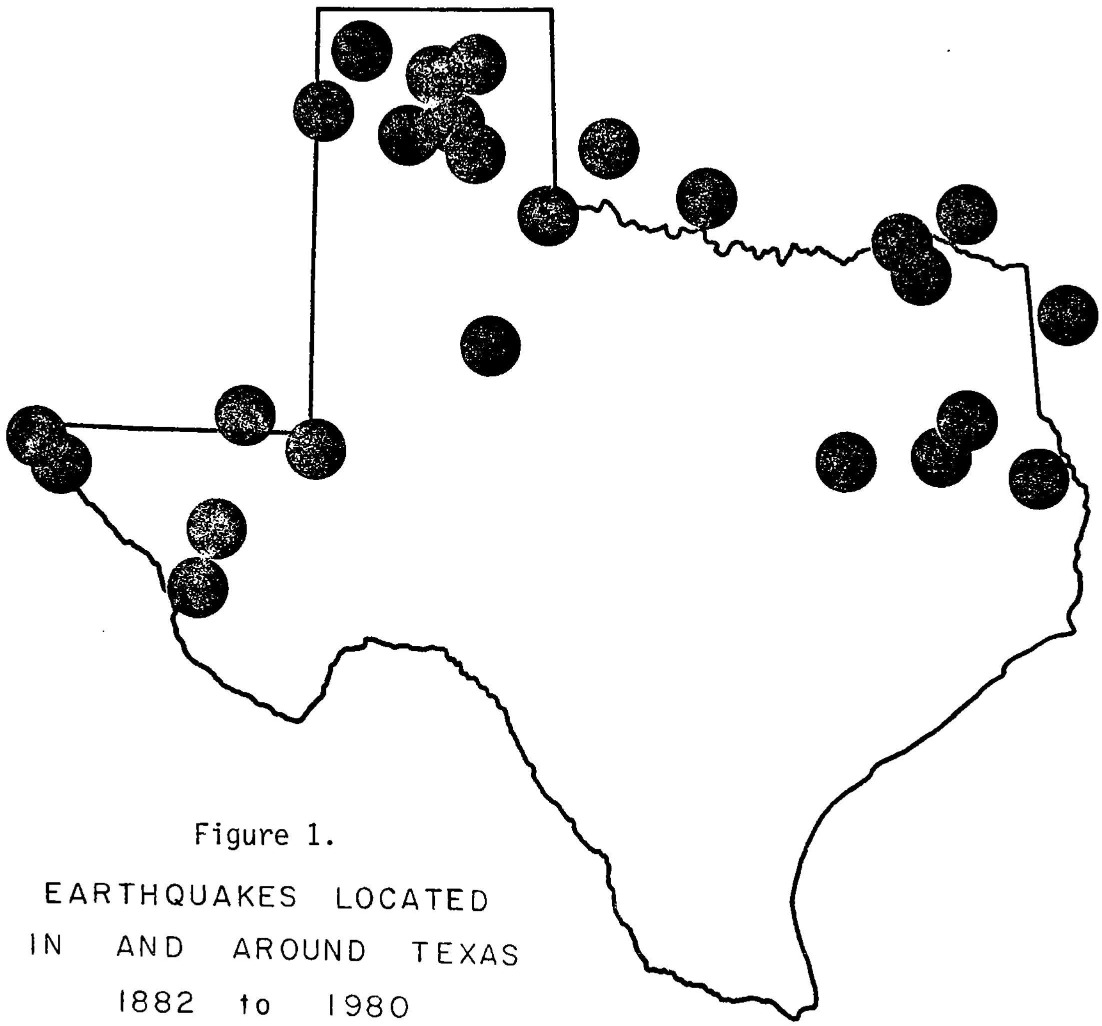

Figure 1.

EARTHQUAKES LOCATED IN AND AROUND TEXAS 1882 to 1980

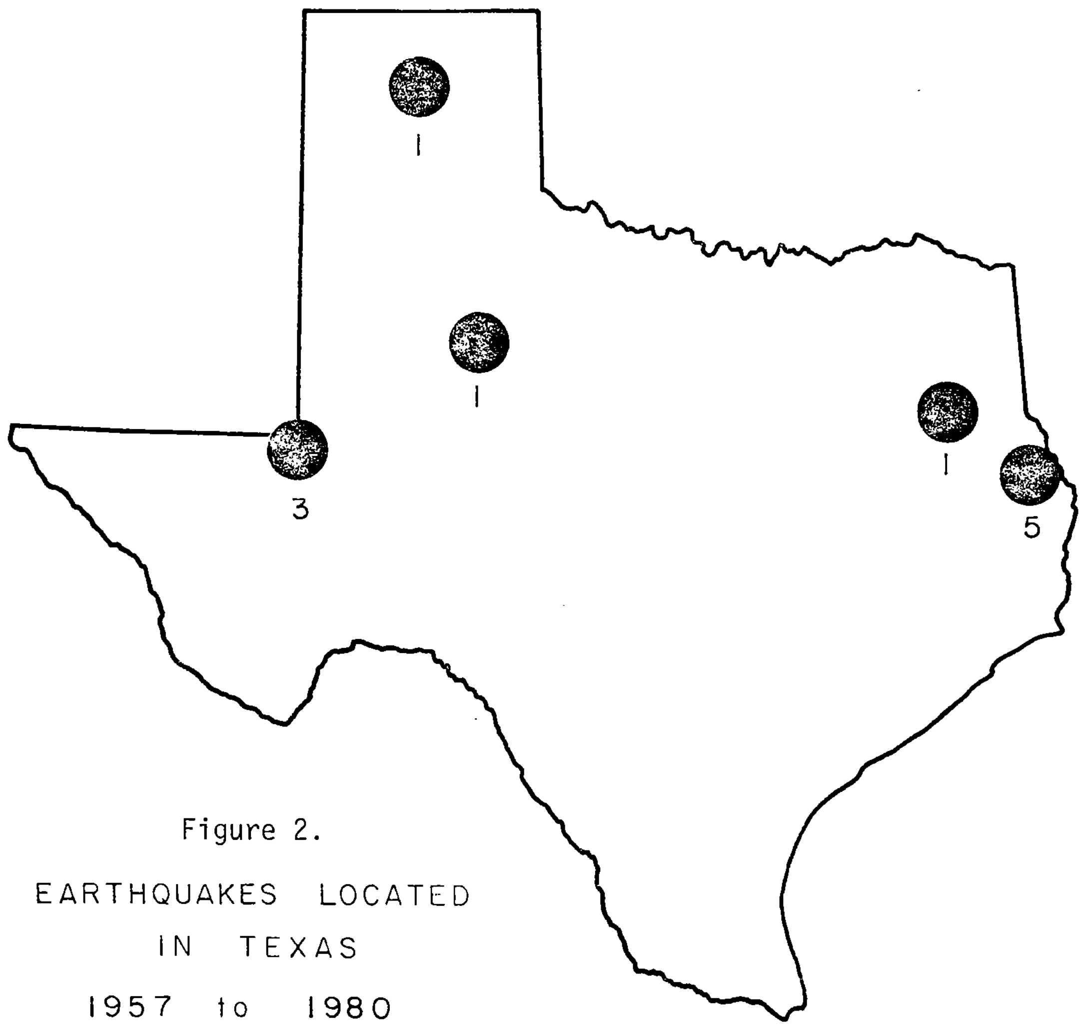

Figure 2.

EARTHQUAKES LOCATED IN TEXAS 1957 to 1980 active before construction of the dam and impoundment of water (Carder, 1968). There were similar occurrences soon after completion of Lake Mead in Arizona (Carder, 1968), Lake Kariba on the Rhodesia-Zambia border (Carder, 1968), and Lake Meredith in Texas (Shurbet, 1969). Widespread damage and fatalities resulted from some of these occurrences; however, no damage and fatalities were reported at Lake Meredith. Shurbet reported a large number of microearthquakes in the vicinity of Lake Meredith, but only one or two were felt by local people.

Since 1973 the Texas Department of Water Resources (TDWR) has been conducting microseismicity studies with Professor D. H. Shurbet of Texas Tech University Seismological Observatory in areas of the State where new storage and conveyance systems have been proposed and where existing facilities are concentrated. Shurbet's studies, performed through an interagency contract, have utilized TDWR's seismograph, which is a portable microearthquake seismograph system built by Spregnether Company. The purpose of these studies has been threefold: (1) to determine the natural seismicity of those areas of the State in which water facilities are planned; (2) to provide seismic data for developing sound engineering design criteria for the construction of dams and reservoirs; and (3) to collect data for the determination of possible man-made earthquakes. Thus far the study areas have been in the south-central and northeast sections of the State. Data have been collected at Mount Pleasant, Victoria, San Marcos, Riviera, Silverton, Austin, Three Rivers, Huntsville, Commerce, Conroe Dam, and Livingston Dam.

To date, TDWR's seismograph has not detected natural microearthquake activity at any of the locations where it has been placed. A microearthquake was recorded when the seismograph was located near Commerce, Texas, but the event was believed to have occurred in Oklahoma. Data collected at the Livingston Dam site indicated that a microseismic event possibly occurred in the area, but the

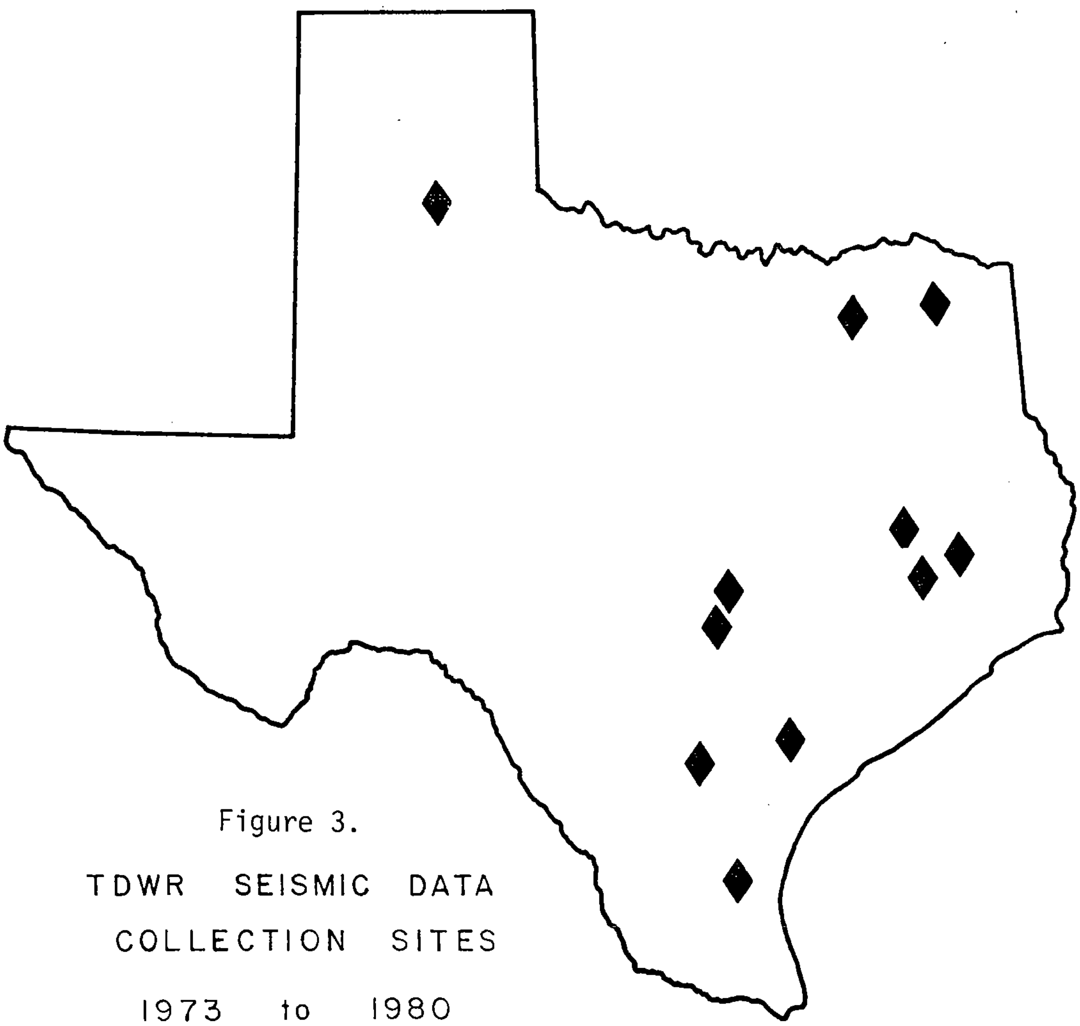

Figure 3.

TDWR SEISMIC DATA COLLECTION SITES

1973 to 1980

information was insufficient to be conclusive. In other studies Shurbet has recorded microearthquakes along the Rio Grande in both Texas and New Mexico, in the northern Texas Panhandle, near Kermit, Texas (Shurbet, 1969), in the Big Bend region, and near Snyder, Texas (Shurbet, 1979).

Aubrey D. Henley (1966) reported that in 1964 more than 70 microearthquakes occurred near Hemphill, Texas. Five earthquakes occurred that were of sufficient magnitude to be recorded by permanent seismograph observatories at considerable distance. This area had no history of earthquakes prior to 1964. Henley, who happened to be working in the area, detected these 70 microearthquakes; most of these events might not have been detected if he had not been working there.

Numerous microearthquakes were recorded by Shurbet in the vicinity of Kermit prior to the earthquake that occurred there in July 1966. The same sequence of events occurred preceding an earthquake that occurred north of Snyder, Texas (Shurbet, 1979), in June 1978 that had a magnitude of 4.6 on the Richter scale.

Shurbet's study for TDWR is part of his project to determine total natural seismicity in the State of Texas. Basic seismic data are a pertinent consideration in the total geologic investigation of any proposed construction project. The growing practice of considering seismic data during the design of a project is a step forward in preventing catastrophic failures and mankind's activities from triggering earthquakes.

While there is no history of large earthquakes in Texas, there is a history of small earthquakes that indicate areas of instability. Human activities might cause large earthquakes in unstable areas by changing the crustal loading. Design engineers need to know which areas are seismically active and which are stable so that they can adequately design structures. Dams and other engineering structures constructed in stable areas normally have less stringent design requirements than do those in unstable areas. The seismic record may be a determining factor in selecting, for instance, a dam site from among alternative sites as well as the design criteria to be considered.

Projects other than building dams and reservoirs might also benefit from studies of natural seismicity. Earthquakes have also been triggered by injection wells through which fluids are pumped under high pressure into the subsurface rocks. These problems have been reported at the Denver Rocky Mountain Arsenal disposal well and Rangely, Colorado, where secondary petroleum recovery took place. Secondary petroleum recovery methods are suspected to have caused the earthquakes near Kermit and Snyder, Texas. Earthquakes related to injection wells are not known to have caused any major damage; however, the damage earthquakes do to mankind is a function of their proximity to populated areas.

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CHARACTERIZATION OF SHALES BY PLASTICITY LIMITS, POINT LOAD STRENGTH, AND SLAKE DURABILITY

by

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INTRODUCTION

During the last decade significant progress was made in developing individual tests and classification systems for the use of shales in highway embankments. Almost all the systems use some test to evaluate the durability of shale to cycles of wetting and drying. These tests provide an index that reflects the long-term durability of shale placed in an embankment or fill after it has been excavated, broken down, and recompacted. The tests are performed on pieces of shale not subject to any load; the index does not reflect the strength of the shale. Cementation bonds usually make the shale more durable but they may not contribute as much to the strength of the shale.

Characterizing the strength of shales is equally important as this property has a direct bearing on any difficulties that might be encountered in excavating the shale, breaking it down, and compacting it to a suitable density. The type of equipment required to perform various tasks, as well as the construction costs, would largely depend on the strength of the shale. Hence, a classification system that takes into consideration the strength of the shale, in addition to its durability, would have a significant advantage over those systems based solely on durability.

Some classification systems use the plasticity index of the material comprising the shale. When the test sample is obtained from shale that has naturally disintegrated in water, the use of the plasticity index in a classification system is meaningful because it reflects the character of the shale after it has degraded. On the other hand, the more durable shales do not disintegrate naturally in water, and material suitable for performing the Atterberg limits tests has to be artificially prepared by mechanically breaking down shale pieces to fine particles passing a No. 40 sieve. For such shales the plasticity index is not as meaningful, and its inclusion in a classification system would appear to be questionable.

This paper examines engineering classification systems shown in Table No. 1. The systems proposed by Chandra (1970), Deo (1972), and Hudec (1978) rely solely on slaking indexes. But the test for evaluating an index is different in each system, which is unfortunate. The system proposed by Gamble (1971) uses the plasticity index and the slake durability index for classifying all shales regardless of their durability and strength. The system proposed by Strohm and others (1978) uses the jar-slake test and the slake durability test with descriptive supplements for the retained material. A new system has been tentatively proposed by Franklin Trow Associates (1979), that rates low durability shales based on their plasticity index and their durability index, and rates high durability shales based on their point load strength index and their slake durability index. The essential features of the test conditions for evaluating the indexes are shown in table 1.

Some Midwestern shales have been classified using the Franklin system and the other systems of classification wherever such a comparison is possible. A non-linear equation correlating one-cycle and two-cycle slake durability indexes is developed in this paper using data from Chapman (1975) and Hale (1979).

Table 1. Comparison of tests and systems for classifying shales.

Description of Test	Source				
	Gamble (1971)	Deo (1972)	Strohm (1978)	Hudec (1978)	Franklin (1979)
Simple Slaking Test or Wet/Dry Deterioration Test	No	Yes	Yes	No	No
Slake Durability Test	Yes	Yes	Yes	Yes	Yes
Dry Samples 200 rev.	Yes	No	Yes	Yes	Yes
500 rev.	No	Yes	No	No	No
Soaked Samples 500 rev.	No	Yes	No	No	No
Number of Cycles/Test	2	1	2	3	2
Number of Pieces/Test	10	10	10	n.a.	10
Aggregate Mass (g) or	40-60	50-60	40-60	-	40-60
Size (in)	-	-	-	P3/4RI/2	-
Total Sample Mass/Test (g)	450-550	500-600	450-550	500	450-550
Description of Partially Slaked Material Retained	No	No	Yes	No	No
Point Load Strength Test	No	No	No	No	Yes
Plasticity Index	Yes	No	No	No	Yes

Table 2.
Durability Classification - 5 cycle slaking loss, +½ inch (13 mm) sample size (after Hudec, 1978)

Classification	Number of Samples	Limits (% Loss)	Nature of Loss
Rock-like	20	0-5	No discernible effect.
Low Loss	6	5-25	Minor spalling along bedding planes.
Intermediate Loss	3	25-60	Spalling and disintegration.
High Loss	14	60-100	Disintegration into mud-like consistency.

It is shown to be applicable to shales tested by Gamble (1971) as well, indicating that the relationship may be similar for many Midwestern shales. However, the relationship for Ontario shales tested by Hudec (1978) appears to be different, and the correlation equation proposed in the paper must be verified experimentally before use. The data presented by Hudec (1978) also indicate that the variation of the slake durability index with the number of cycles depends not only on the type of shale but also on the size of the pieces of shale used in the test.

In regard to the point load strength index, it is shown in this paper that the size correlation chart proposed by the International Society for Rock Mechanics (ISRM, 1972b) is not generally applicable for Midwestern shales. The variation of the point load strength index with the size of the specimen appears to be dependent on not only the type of shale but also the shape of the specimen and the orientation of the loading points with respect to any bedding planes. Data presented by Bieniawski (1975) for the variation of the point load strength index with the diameter for diametrically loaded specimens are also at variance with the size correlation chart (ISRM, 1972b). Data presented by Bailey (1976) and Hale (1979) on irregular pieces of shale loaded perpendicular to the bedding planes yield similar conclusions, although the data refer to relatively small sizes of specimens, some of which are outside the size limits shown on the (ISRM, 1972b) correlation chart.

The writers support the recommendations of the International Society of Rock Mechanics (outlined in ISRM, 1972a) in regard to the evaluation of the slake durability index for classification purposes. In addition to recording the slake durability index, it is absolutely essential to record the condition of the retained material as proposed by Strohm and others (1978). Evaluation of the point load strength index requires standardizing the shape and size of the specimen as well as the mode of loading with respect to any bedding planes in the shale, if this index is to be used to classify shales. Experimental data obtained from standardized specimens should be generated to determine whether the rate of loading should be standardized as well.

REVIEW OF SHALE CLASSIFICATION SYSTEMS

Shales have been distinguished from other argillaceous rocks by geologic classification systems such as those proposed by Mead (1936), Gamble (1971) and others. For example, by observing the breaking characteristics of mudrocks, those that are fissile are classified as shales whereas those that are massive are classified as mudstones. Further subdivisions of shale have been made depending on the predominance of silt (silty shale) and clay (clayey shale). Shales have been subdivided further into soil-like and rock-like depending on the cementation bonds between the primary particles of silt and clay. These cementation bonds usually make the shale more durable, but they depend greatly on the chemical composition of the cementing agents and of the water in the shale.

These classifications of shale are inadequate for engineering purposes, and several engineering classification tests and systems based on the physical and mechanical properties of shale have been developed. The nature of the tests selected to characterize the shale was influenced to a great degree by the type of engineering project or activity envisaged.

Underwood (1967) was mainly concerned with slope stability and tunnel support problems and assigned ranges of values for physical properties for favorable and unfavorable in-situ behavior with respect to pore pressure development, bearing capacity, tendency to rebound, slaking, erosion, slope stability, and tunnel support. With regard to slaking, shales that reduced to flakes under cycles of wetting and drying were considered favorable whereas those that reduced to grain sizes were considered unfavorable. This criterion may be acceptable for slope stability and tunnel support, but it is clearly unacceptable for use of broken up shale in embankment construction. Underwood's (1967) tests, though valuable from an engineering point of view, are too complex and costly for routine testing and classification of shales, especially for embankments.

Gamble (1971) also proposed an engineering classification system based on the plasticity index and the two-cycle slake durability index in the Franklin test. Figure 1 shows Gamble's limits for slake durability and plasticity, which are identical to those proposed earlier by Chandra (1970).

The classification system developed by Deo (1972) for use of shales in embankments is shown in figure 2, and it has been used by the Indiana State Highway Commission for a number of years. Although figure 2 indicates that four classification tests have to be performed, in practice only some of them are needed. For example, if a shale slakes completely in the simple slaking test in water, it is immediately classified as "soil-like," and the remaining tests are not necessary. If it does not slake completely, the 500-revolution, single-cycle slake durability test is performed on dry samples. If the value of the resulting index $(I_d)_d$ is less than 90, the shale is classified as "soil-like" and the remaining tests are omitted. Whenever $(I_d)_d$ is greater than 90, the 500-revolution, single-cycle slake durability test is performed on soaked samples. If the value of the resulting index $(I_d)_s$ is less than 75 the shale is classified as "soil-like," and if it lies between 75 and 90 the shale is classified as "intermediate-2." The last test, namely the 5-cycle modified soundness test is performed only when $(I_d)_s$ is greater than 90. If the modified soundness index (I_s) is less than 70, the shale is classified

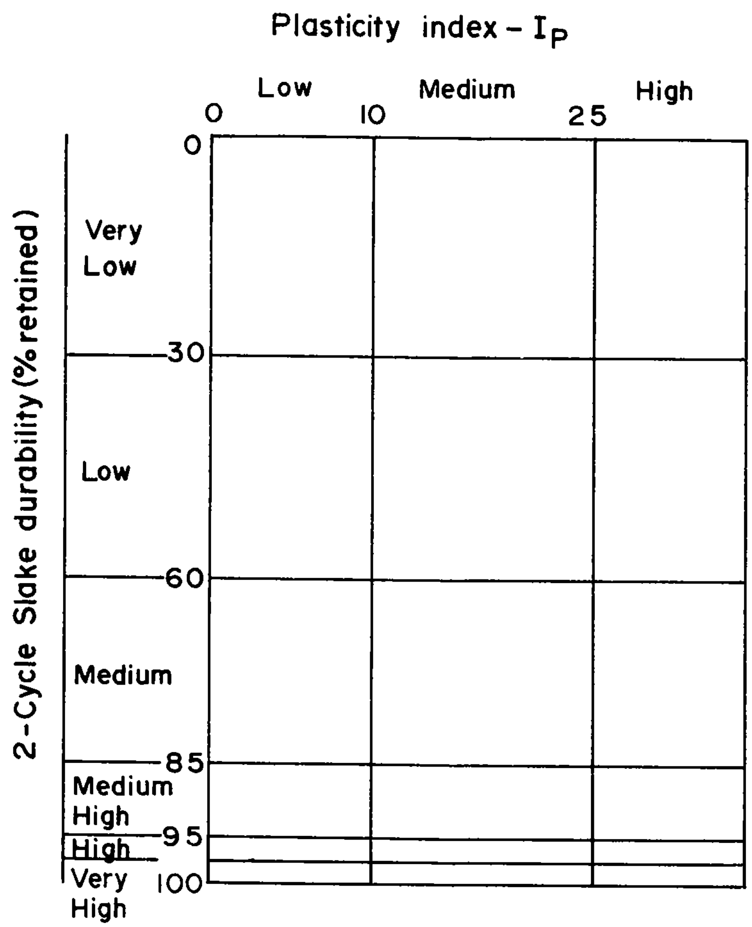


Figure 1. Gamble's durability-plasticity classification for shales and other argillaceous rocks (from Gamble, 1971)

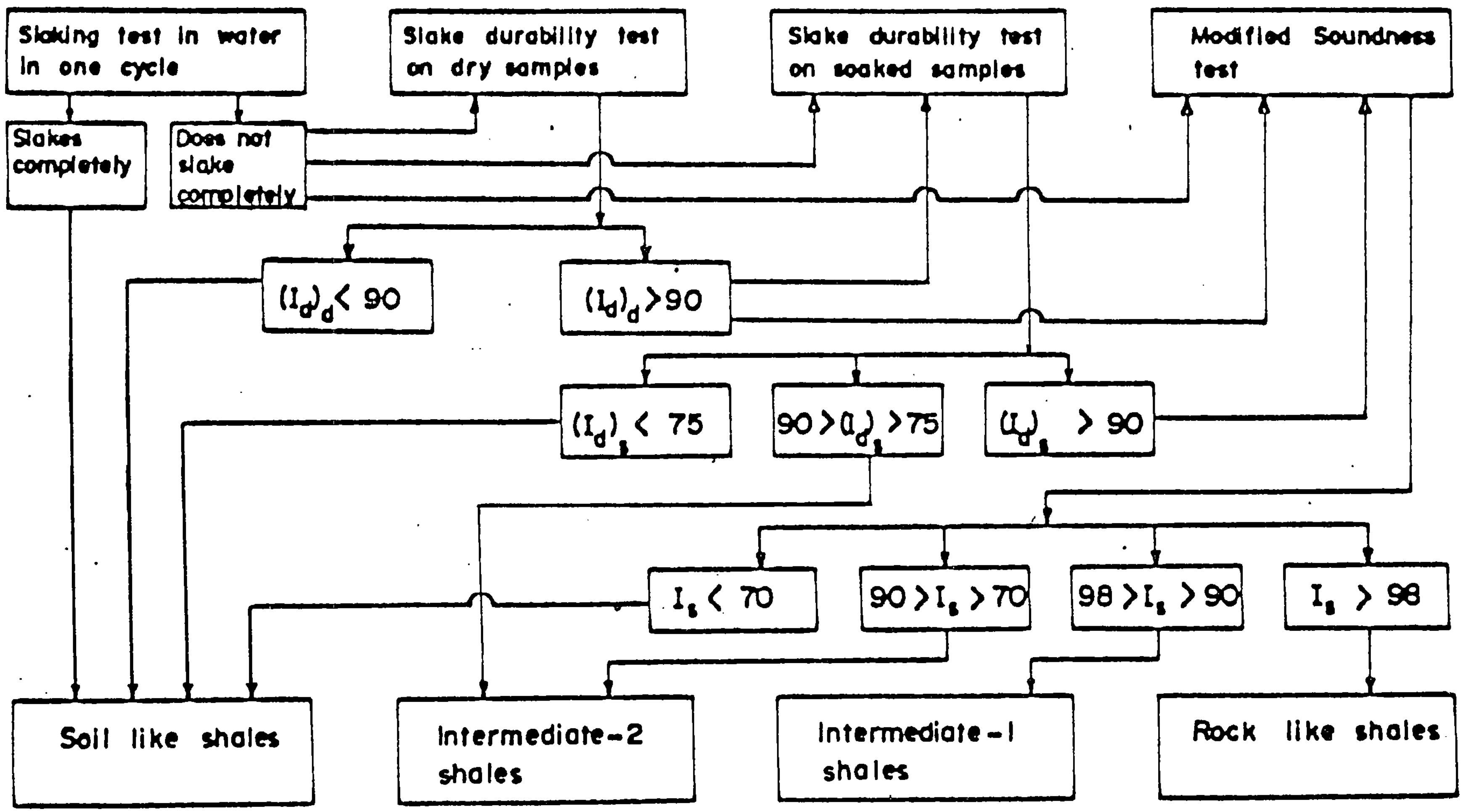
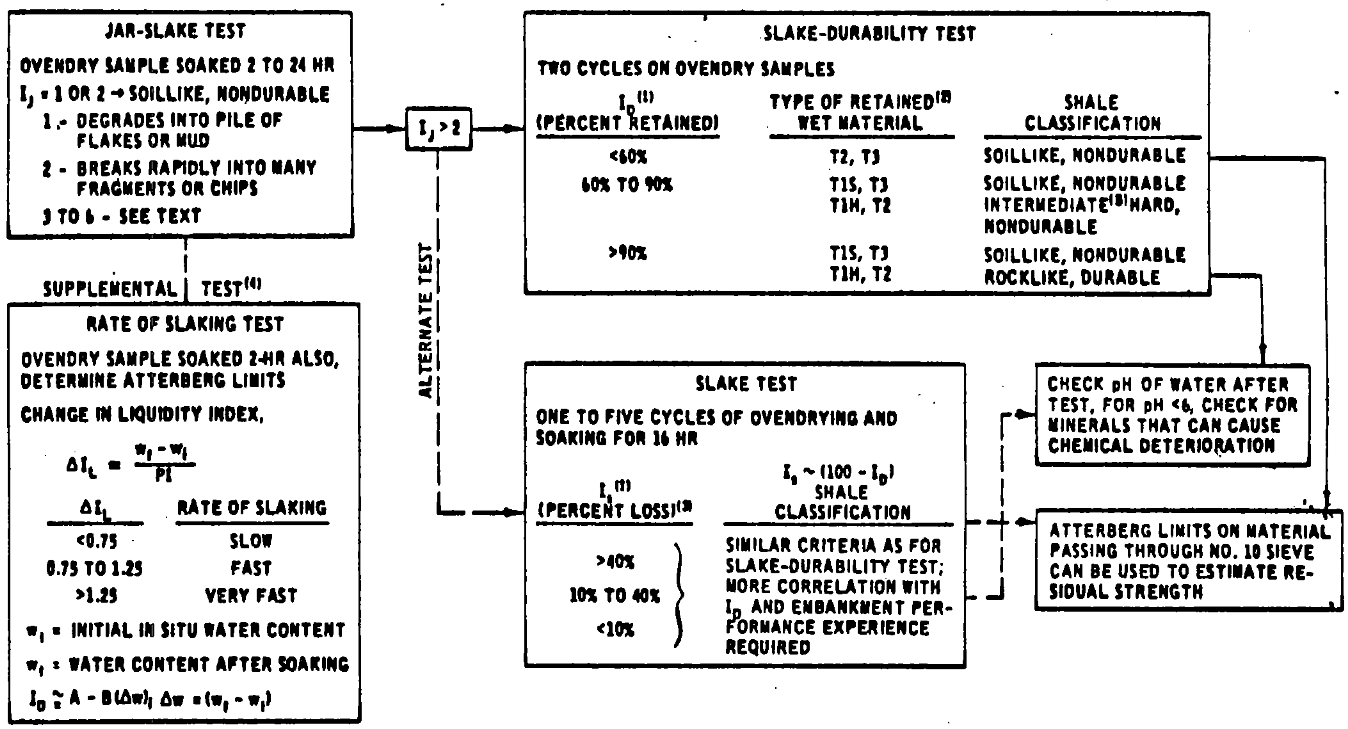


Figure 2. Proposed classification of shales for embankment construction (after Deo, 1972)

as "soil-like;" when (I_s) is between 70 and 90, the shale is classified as "intermediate-2;" when (I_s) is between 90 and 98, the shale is classified as "intermediate-1;" and when (I_s) is greater than 98, the shale is classified as "rock-like." A revision of Deo's (1972) classification system is under consideration as none of the Indiana shales tested thus far classify as intermediate-2 or intermediate-1.

Strohm and others (1978) recommended the jar-slake test and the slake durability test to assess shale durability. Figure 3 shows a schematic flow diagram taken from their report. They point out that it is necessary to consider the condition of the retained particles in the slake durability test, in addition to the value of the slake durability index (I_D) in order to classify a shale. Shales that soften but sometimes do not degrade appreciably (T1S, fig. 3), and shales that break down significantly but do not become smaller than the No. 10 sieve (T3, fig. 3) may have a deceptively high durability index.

Hudec (1978) and Franklin Trow Associates (1979) proposed new classification systems for use of Ontario shales in embankments. Hudec (1978) used a single parameter, namely the percent mass loss (M) in the Franklin slake durability apparatus, to classify a shale into four categories: rock-like ($0 \leq M < 5$), low loss ($5 < M < 25$), intermediate loss ($25 < M < 60$), and high loss ($60 < M < 100$), as shown in table 2. The percent mass loss is evaluated after five standard cycles of wetting and drying. Franklin Trow Associates (1979) use three parameters in all, namely the plasticity index (I_p), the two-cycle slake durability index (I_{d2}), and the point load strength index (I_s)50 as shown in figure 4. The slake durability index is evaluated first for all shales. The plasticity index is evaluated only for those shales whose slake durability index (I_{d2}) is less than 80, and the point load strength index is



NOTE: (1)DIFFERENT LIMITING VALUES MAY BE JUSTIFIED ON BASIS OF LOCAL EMBANKMENT PERFORMANCE EXPERIENCE.

(2)TYPE T1 - NO SIGNIFICANT BREAKDOWN OF ORIGINAL PIECES.
TYPE T1S - SOFT, CAN BE BROKEN APART OR REMOLDED WITH FINGERS.
TYPE T1H - HARD, CANNOT BE BROKEN APART.
TYPE T2 - RETAINED PARTICLES CONSIST OF LARGE AND SMALL HARD PIECES.
TYPE T3 - RETAINED PARTICLES ARE ALL SMALL FRAGMENTS.

(3)USING NO. 10 SIEVE.

(4)CAN BE PERFORMED ON JAR-SLAKE TEST SAMPLES IF IN SITU NATURAL WATER CONTENT IS KNOWN. PI SENSITIVE TO DEGREE OF PULVERIZATION.

(5)REQUIRES SPECIAL PROCEDURES TO ASSURE GOOD DRAINAGE AND ADEQUATE COMPACTION (95% T-99) FOR LOOSE LIFT THICKNESS UP TO 24-IN. MAXIMUM.

Figure 3. Recommended durability index tests and suggested classification criteria for shales used in highway embankments (from Strohmand and others, 1978)

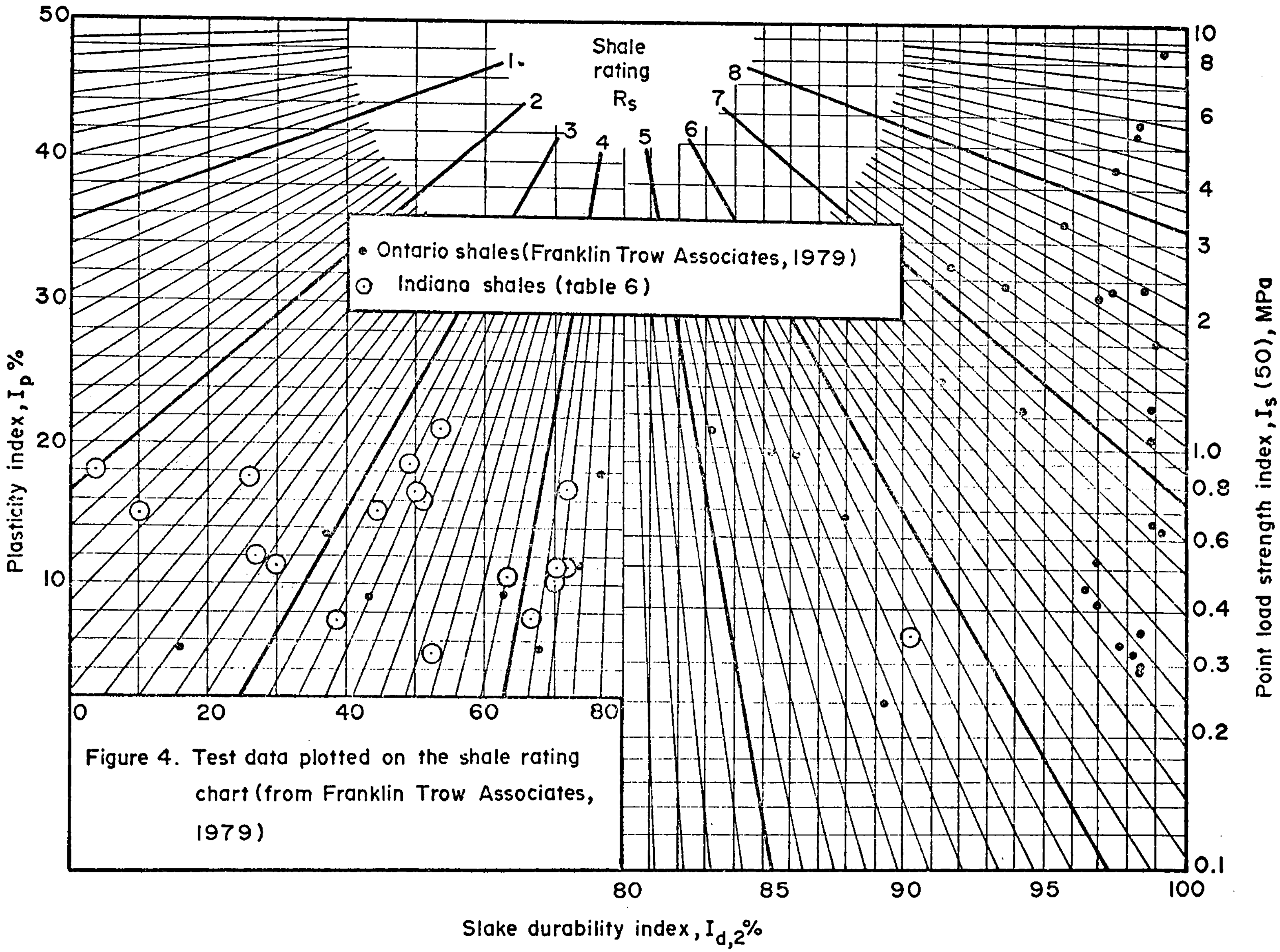


Figure 4. Test data plotted on the shale rating chart (from Franklin Trow Associates, 1979)

evaluated only for those shales whose slake durability index (I_{d2}) is greater than 80. Thus only two parameters are really required to characterize a single shale. The two parameters applicable to a shale are plotted on the chart shown in figure 4 and a single parameter called the shale rating (R_s) is obtained. The shale rating (R_s) has been correlated (using field data) with some important construction parameters for shale embankments, such as lift thickness, excavation method, compaction equipment, and compacted field density necessary for satisfactory performance. Figure 5 shows a tentative chart proposed by Franklin Trow Associates (1979).

SLAKE DURABILITY CHARACTERISTICS

When pieces of shale are subjected to cycles of drying and wetting, slaking takes place to varying degrees depending on the type of shale, the size and shape of the pieces, the chemical composition of the slaking fluid, and the duration and number of cycles. The degree of slaking is also dependent on whether the pieces of shale are at rest, as in the simple slaking test, or whether they are subjected to an abrasive action as in the slake durability test in the Franklin apparatus. The very nondurable shales weaken and disintegrate almost totally to a mud or soil-like consistency. The highly durable shales, on the other hand, are hardly affected and retain nearly their original dimensions and mass. Between these extremes of behavior, one finds a number of shales which spall along bedding planes and partially disintegrate to various sizes. The very non-durable shales and the very durable shales attain an equilibrium condition of slaking after two or three cycles of drying and wetting, whereas intermediate shales may not reach an equilibrium condition even after five cycles.

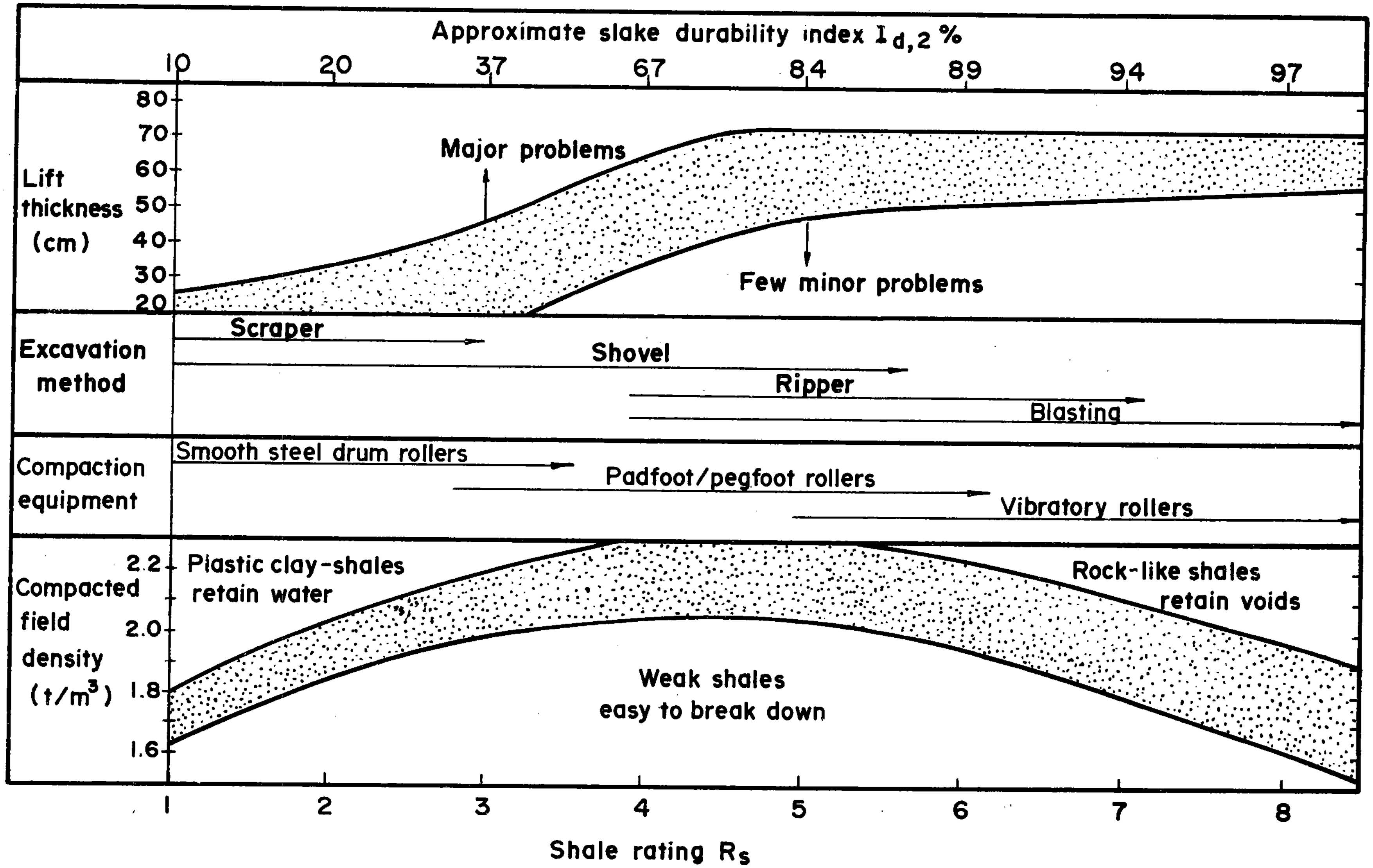


Figure 5. Tentative correlations between the shale rating, R_s, and behavior in embankment construction (after Franklin Trow Associates, 1979)

Effect of Aggregate Size and Cycles

An extensive study of the effect of aggregate size and the number of cycles of drying and wetting on the mass loss of several Ontario shales was investigated by Hudec (1978). His tests were performed on initially oven dried pieces of irregular shape in a standard Franklin slaking apparatus with the drum rotated at 20 revolutions per minute for a total of 200 revolutions per cycle, as outlined in ISRM (1972b). The following trends may be observed.

(1) The effect of aggregate size on mass loss is greatest in the first few cycles, and after the fifth cycle the losses for all sizes tested are similar.

(2) For highly nondurable and highly durable shales the change in percent mass loss beyond the third cycle is negligible for all particle sizes, whereas for intermediate shales, appreciable losses take place between the third and fifth cycles, for all sizes tested.

(3) For most shales, the mass loss is more or less complete after the fifth cycle, but some shales show a tendency to lose weight beyond the fifth cycle for all sizes tested.

In a recent study concluded by D'Appolonia Consulting Engineers (1979) for the Bureau of Mines, fragment size was shown to be important for all types of durability tests. In general the coarse fragments developed greater breakdown for all slaking fluids examined and temperature conditions. The most noticeable changes occurred for thinly bedded, anisotropic sedimentary rocks, such as siltstones and shales. The study suggests that fragments sized approximately one inch or larger be used and states that some modification of the standard slake durability apparatus may be required to accommodate this recommendation.

Effect of Shape

The slake durability test is usually performed on irregular pieces of shale, arbitrarily prepared by breaking down larger lumps to the required size or weight (see fig. 9). However, in the D'Appolonia Consulting Engineers (1979) study, cylindrical samples of shale and other rocks were subjected to the simple slaking test, the cycle wet/dry test, and the slake durability test. In all these tests the initial slaking took place along the bedding planes and resulted in disc-shaped pieces which subsequently broke across the bedding planes. As no tests appear to have been performed on irregular pieces of the same shales, no conclusions can be drawn at this point on the effects of shape (cylindrical vs irregular). However, considering the way in which mass loss is defined, namely material passing through a No. 10 mesh, it is reasonable that more mass loss would take place for irregular pieces than for cylindrical specimens, particularly in the first few cycles. This factor needs more study.

Gradation and Strength after Slaking

The partially slaked shale aggregate retained in the drum of the slake durability apparatus can have widely different gradation and strength, depending on the shale tested. In regard to the gradation it is recommended that an index of slaking be determined from a knowledge of the initial and final gradations in a manner similar to calculating the index of crushing used by Bailey (1976) to quantify degradation during compaction. In regard to strength it is recommended that a qualitative description similar to the "T" rating of Strohm and others (1978) be used.

The slake durability index measures neither the change in gradation nor the change in strength caused by slaking, and sole reliance on its value is not advisable. The value of the proposed index of slaking and the condition and strength of the pieces retained in the drum of the slake durability apparatus should also be considered in assessing the durability of the shale.

Correlation of Two Cycle and One Cycle Slake Durability Index

To classify shales according to the rating system proposed by Franklin Trow Associates (1979) it is necessary to establish values for the two-cycle slake durability index (ISRM, 1972a). At the present time, values of slake durability for many Indiana shales are available for only a single cycle of drying and wetting, as it is this index which was used by the Indiana State Highway Commission for classifying shales following the system developed by Deo (1972). When Chapman (1975) reviewed and extended the work of Deo (1972), the two-cycle slake durability index had already been accepted by the International Society for Rock Mechanics (1972a) and also had been adopted in the classification systems of Chandra (1970) and Gamble (1971). Evaluation of both indexes for Indiana shales appeared to be advisable, pending a decision as to which index to use for classification purposes. Both indexes are presently being measured by the Indiana State Highway Commission.

For Indiana shales the relationship between the one-cycle slake durability index $(I_d)_{d,1}$ and the two-cycle slake durability index $(I_d)_{d,2}$ is given by the equation

$$[(I_d)_{d,1} + 20]^2 + [(I_d)_{d,2} - 120]^2 = R^2$$

where $R^2 = 120^2 + 20^2 = 14800$. This is an equation of a circle of radius $R = \sqrt{14800} = 121.65$, and its center had coordinates corresponding to hypothetical values of $(I_d)_{d,1} = -20$, and of $(I_d)_{d,2} = 120$. The center of the circle and its trace for $0 \leq (I_d)_{d,1}$ $(I_d)_{d,2} \leq 100$ is shown in figure 6, along with the data points for the Indiana shales tested by Chapman (1975) and Hale (1979).

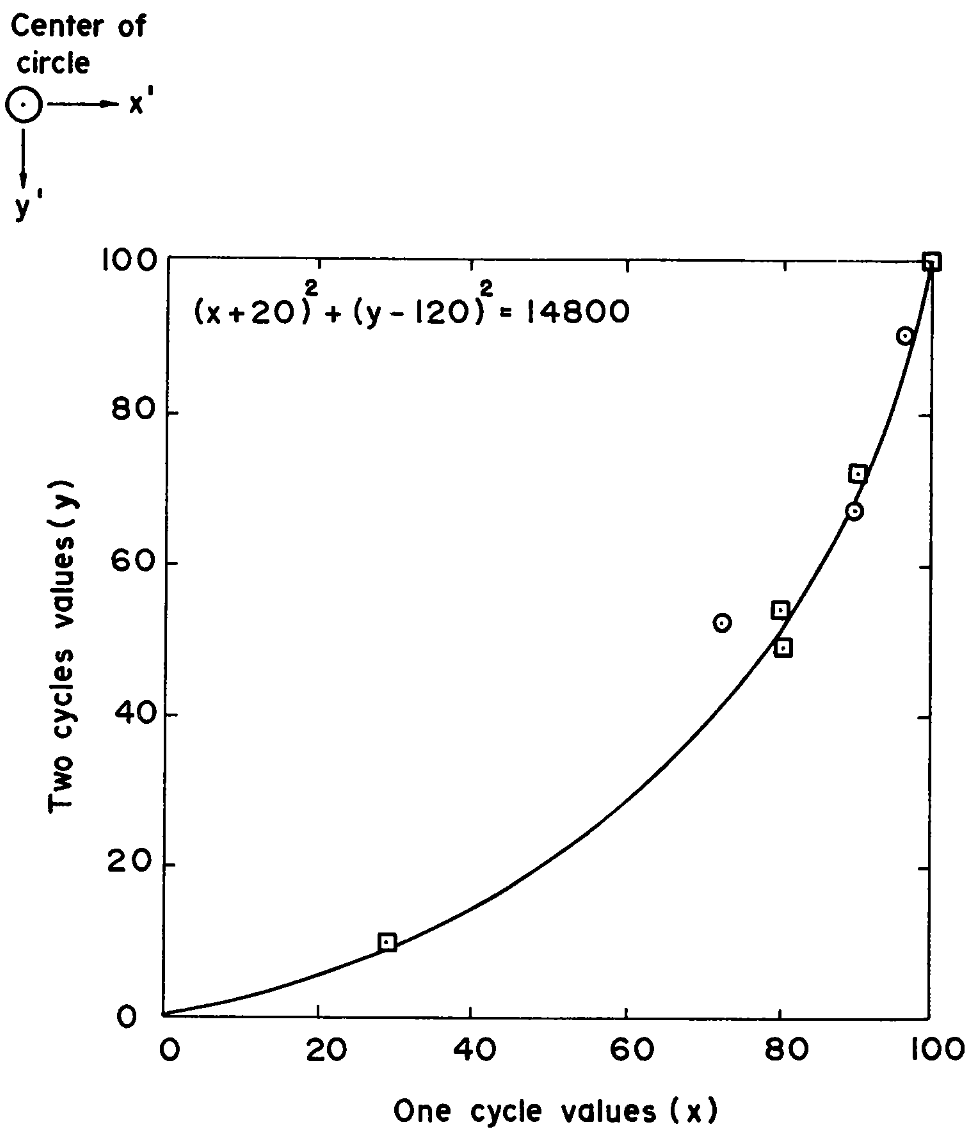


Figure 6. Relationship between one and two cycle slake durability values (dry, 200 revolutions)

This equation seems to be applicable for the shales tested by Gamble (1971) as well, as evidenced by his limits for slake durability based on one-cycle and two-cycle values given below:

Slake Durability	1-Cycle	2-Cycle
Very Low	0 to 60	0 to 30
Low	60 to 85	30 to 60
Medium	85 to 95	60 to 85
Medium High	95 to 98	85 to 95
High	98 to 99	95 to 98
Very High	99 to 100	98 to 100

When the one cycle values are plotted against the two cycle values in figure 6 the points lie almost exactly on the circle.

The equation may be used to obtain values of the two-cycle slake durability index $(I_d)_{d,2}$ from values of the one-cycle slake durability index $(I_d)_{d,1}$ for those Indiana shales for which only one-cycle values are presently available. In particular, it will be used herein to assign values for the two-cycle slake durability index of some Indiana shales tested by Bailey (1976) for the purpose of classifying them using the rating system proposed by Franklin Trow Associates (1979).

The relationship between the one-cycle and two-cycle values of the slake durability index for the Ontario shales tested by Hudec (1978) are shown for small samples in figure 7. It is apparent that the relationship is non-linear, as is the case for the shales tested by Gamble (1971), Chapman (1975), and Hale (1979). However, the difference between the one-cycle and two-cycle values is generally greater for the large and medium pieces of Ontario shales than for the large pieces of Midwestern shales. As shown by figure 7, the correlation equation fits the data for the small pieces of Ontario shales much better.

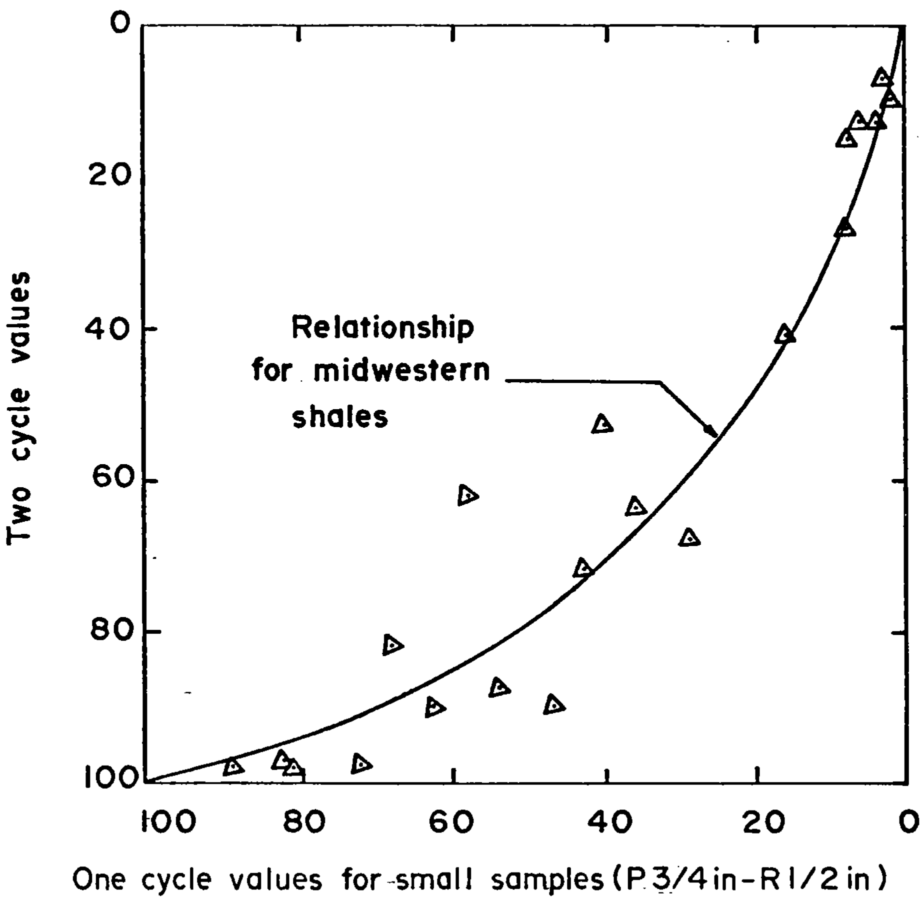


Figure 7. Percentage of mass loss, one cycle vs two cycles (data from table B-2, Hudec, 1978)

A common feature for both Ontario and Midwestern shales is that the difference between the one-cycle and two-cycle values of slake durability is greatest for the intermediate shales and it tends to zero for the very non-durable and very durable shales.

POINT LOAD STRENGTH CHARACTERISTICS

The compressive strength of rock is usually characterized by the unconfined compressive strength of right circular cylinders having a length to diameter ratio (L/D) approximately equal to two. This test is usually carried out in a laboratory due to the complexity of the equipment required to prepare the specimen and to load it axially to failure.

The point load test was used by Franklin (1970) so that an index of rock strength could be obtained in the field by means of a portable apparatus. The details of the loading platens are shown in figure 8. This index is a measure of the tensile resistance of the rock. Figure 9 shows three variations of the test, diametrical, axial, and irregular lump (ISRM, 1972b). The point load strength is defined as P/D^2, where P is the load and D is the initial distance between the loading points. The value of P/D^2 for D=50mm has been defined by ISRM as the point load strength index, $(I_s)_{50}$.

Bieniawski (1975) pointed out that the internationally recognized core size for site investigation drilling is 54 mm (NX) and that it is this size which is used for standard rock quality designation (RGD) determinations (Deere, 1968), as well as for standard unconfined compression strength tests (ASTM, 1971; ISRM, 1972b). Consequently, Bienawski recommends that the point load strength index should preferably be obtained by testing NX cores in the diametrical mode.

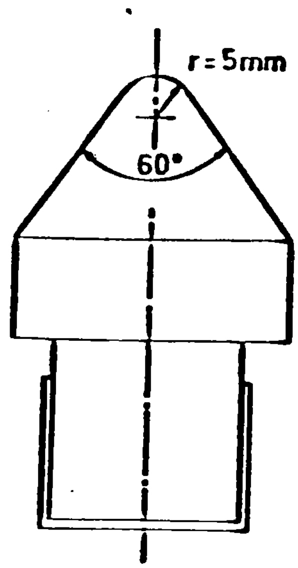


Figure 8. Platen dimensions for point load strength test

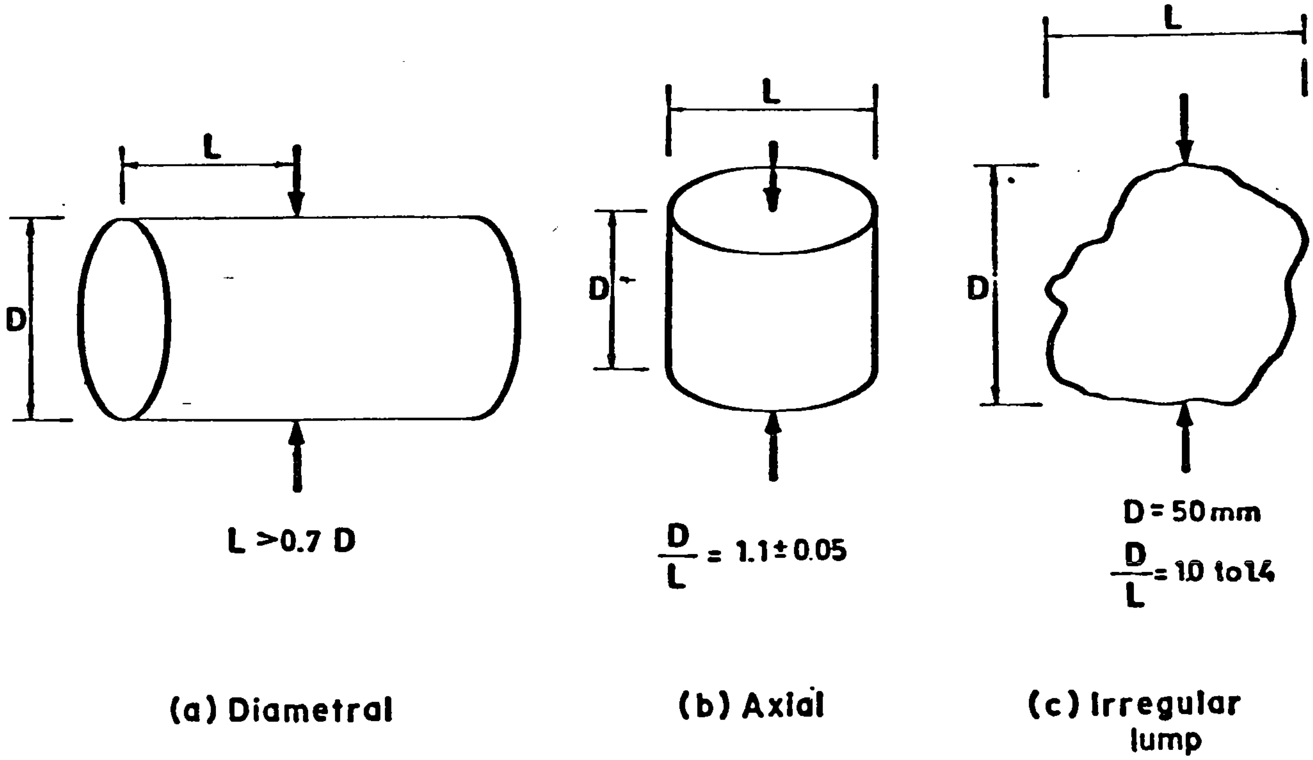


Figure 9. Variations of point load strength test

Figure 10 shows the size correlation chart proposed by ISRM (1972b) for obtaining the point load strength index $(I_s)_{50}$ indirectly from values of P/D^2 determined by testing specimens having D not equal to 50 mm. Each curve represents the variation of the value of P/D^2 with D regardless of rock type, and the dashed lines show how the value of $(I_s)_{50}$ is evaluated.

The size correlation chart is not applicable for all types of rock. To illustrate this, results by Bieniawski (1975) are plotted on the chart for EX core (21.5 mm), BX core (42 mm), and NX core (54 mm) sizes for four rock types, namely, a sandstone, a quartzite, Marikana norite, and Belfast norite. It is evident that the relationship between the point load strength P/D^2 and the core diameter D is different from the relationships in the size correlation chart for all four rock types. Table 3 gives the numerical values.

Further evidence that the correlation is not generally applicable is illustrated using data reported by Hale (1979) for irregular lump specimens of New Providence and Attica shales from Indiana. Hale's data are tabulated in table 4, and the average values of point load strength are plotted against the average size of the specimen in figure 11. It is to be noted that the two curves for the two rock types approach each other as the size of the specimen increases, whereas the correlation chart curves (figure 10) run more or less parallel to each other. Data points falling within the range of the correlation chart are shown in figure 10, for comparison with Bieniawski's (1975) data.

A limited number of point load tests was carried out by the first writer on irregular lump specimens of Osgood shale from Indiana. Out of a total of twelve specimens, nine were loaded perpendicular to the bedding planes and three were loaded parallel to the bedding planes. The results are shown in figure 12, in which the point load strength is plotted against the initial

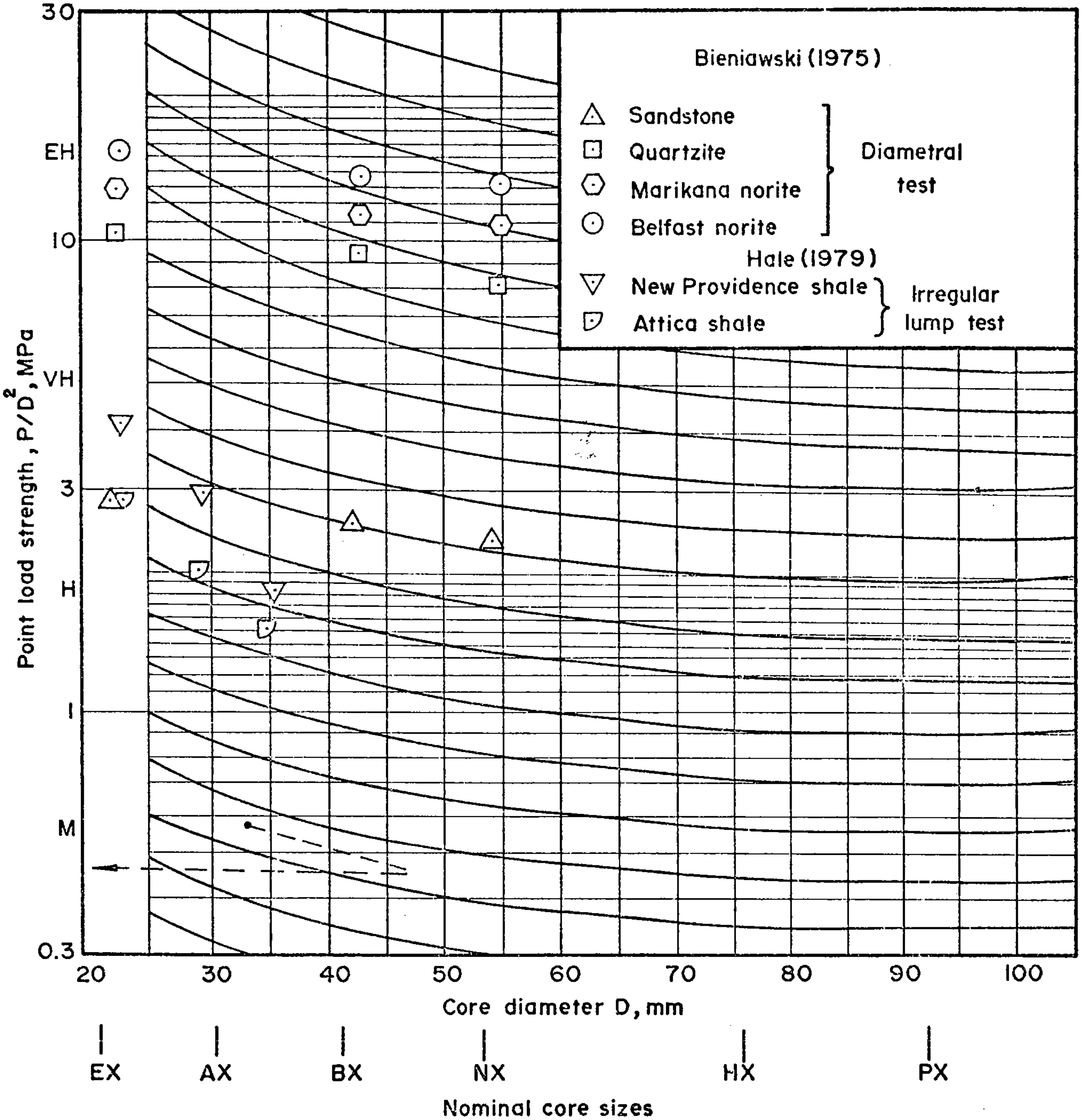


Figure 10. Size correlation chart for point load strength test

Table 3. Effect of specimen size on point load strength values in diametrical test (after Bieniawski, 1975).

Rock Material	Core Size mm	Point Load Index in Diametrical Test			
		No. of Specimens	Mean (MPa)	Standard Deviation (MPa)	(%)
Sandstone	NX 54.0	70	2.33	0.22	9.8
	BX 42.0	65	2.56	0.23	8.8
	EX 21.5	70	2.83	0.22	9.8
Quartzite	NX 54.0	45	8.30	1.35	16.2
	BX 42.0	40	9.47	2.10	22.4
	EX 21.5	40	10.37	1.82	17.5
Marikana norite	NX 54.0	40	10.84	1.57	14.5
	BX 42.0	20	11.16	2.19	19.6
	EX 21.5	20	13.05	1.16	8.9
Belfast norite	NX 54.0	70	13.13	1.21	9.2
	BX 42.0	35	13.77	2.10	15.2
	EX 21.5	40	15.92	0.86	5.4

Table 4. Effect of specimen size on point load strength values in irregular lump test (after Hale, 1979)

Shale Formation	Size Range mm	Point Load Strength in Irregular Lump Test			
		No. of Specimens	Mean Value (MPa)	Standard Deviation (MPa)	Coefficient of Variation (%)
New Providence	5.0 to 12.7	25	18.79	6.59	35
	12.8 to 19.0	29	8.81	3.55	40
	19.1 to 25.4	14	4.17	1.77	42
	25.5 to 31.8	5	3.03	1.28	42
	31.9 to 38.1	7	1.81	0.54	30
Borden	6.3 to 12.7	12	8.21	2.54	31
	12.8 to 19.0	22	4.74	2.10	44
	19.1 to 25.4	19	2.82	0.88	31
	25.5 to 31.8	9	1.95	0.39	20
	31.9 to 38.1	5	1.43	0.41	29

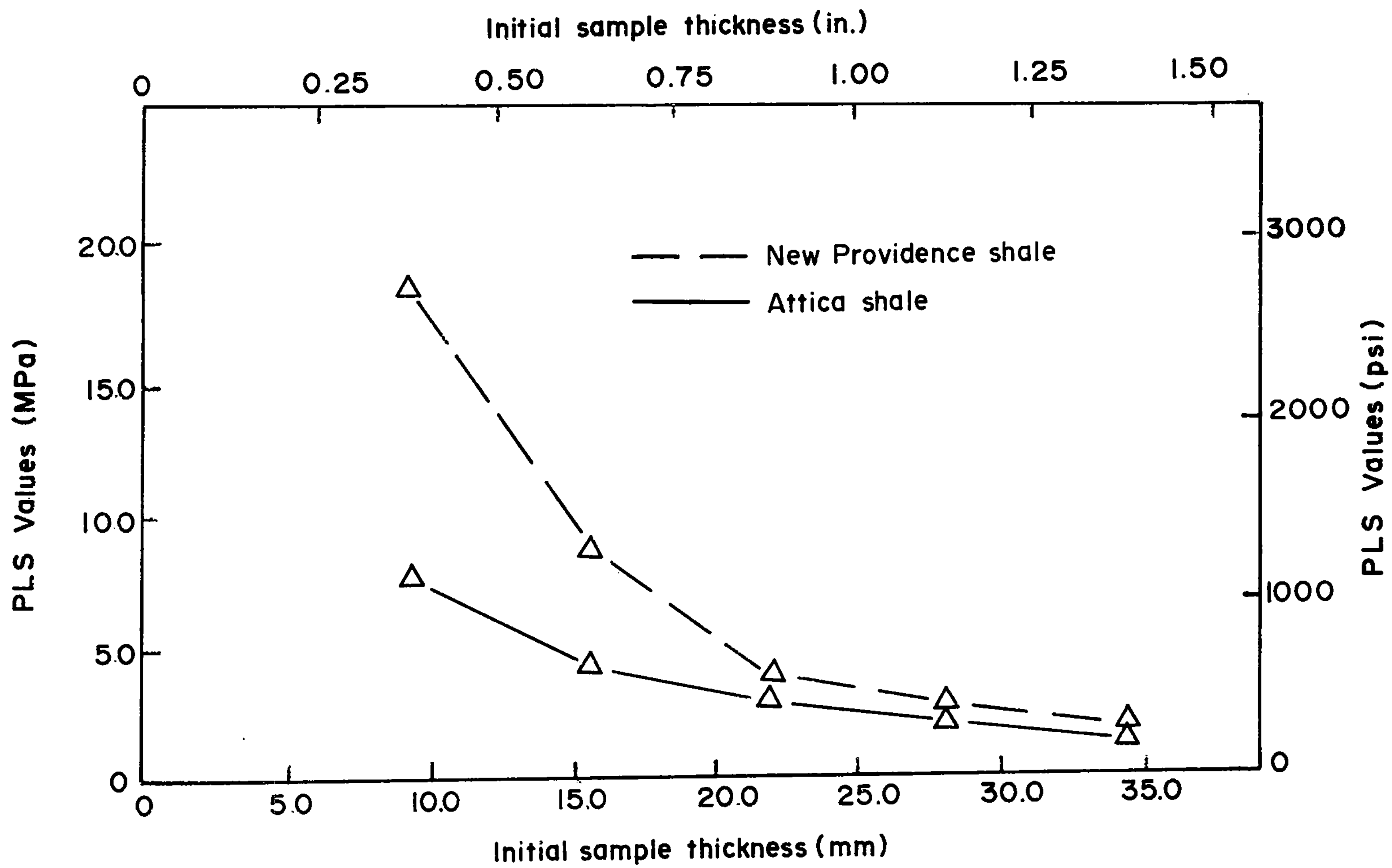


Figure 11. Size effect on point load strength values (after Hale, 1979)

thickness of the specimen between the loading points. The tabulated values are shown in table 5 along with the moisture contents of the specimens. Samples 1 to 8 had moisture contents ranging from 1.06 to 1.35 percent, whereas samples 9 to 12 had moisture contents ranging from 3.45 to 5.08 percent. Samples 5, 6, and 9 were loaded parallel to their bedding planes and the others were loaded perpendicular to their bedding planes. The results show the following trends:

(1) For samples having approximately the same moisture content, the point load strength decreases with increasing sample thickness when loaded perpendicular to the bedding planes, and the rate of decrease is very much higher for the drier samples.

(2) For samples having approximately the same sample thickness, the point load strength decreases with increasing moisture content, when loaded perpendicular to the bedding planes.

(3) For samples loaded parallel to the bedding planes, the effect of sample thickness and moisture content on the point load strength is relatively much smaller than for samples loaded perpendicular to the bedding planes.

(4) Samples loaded parallel to the bedding planes fail at much smaller strains than samples loaded perpendicular to the bedding planes.

(5) The point load strength is dependent on the direction of loading with respect to any bedding planes.

CHARACTERIZATION OF SOME INDIANA SHALES

Table 6 shows the characterization of some Indiana shales according to the classification systems proposed by Gamble (1971), Deo (1972), and Franklin Trow Associates (1979). The shale rating values (R_s) were obtained from

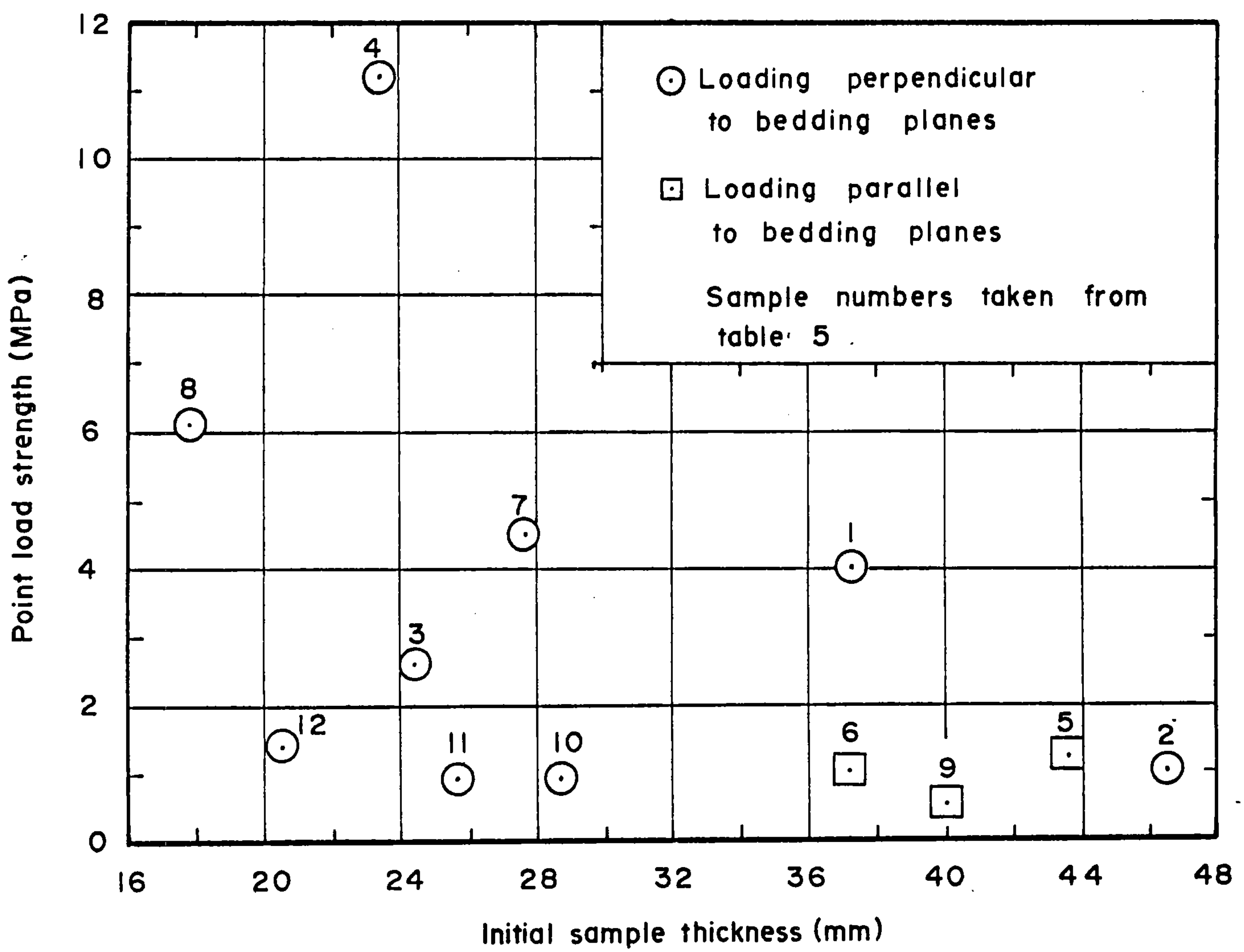


Figure 12. Point load strength of Osgood shale perpendicular and parallel to bedding planes

Table 5. Point load strength data for Osgood shale (79-55199).

Sample Number	Peak Load P (N)	Sample Thickness D (mm)	Point Load Strength P/D^2 (kPa)	Strain at		Sample Moisture Content %	No. of Broken Pieces
				Peak Load %	Rupture %		
1	5582	37.28	4014	10.22	20.44	1.29	6
2	2117	46.36	985	2.74	14.79	1.35	3
3	1539	24.43	2578	5.20	36.38	1.33	7
4	6159	23.44	11206	11.92	11.92	1.18	2
5	2309	43.59	1216	0.70	0.70	1.16	2
6	1347	37.06	981	0.82	2.33	1.12	2
7	3464	27.69	4519	11.93	11.93	1.06	3
8	1925	17.78	6088	11.43	11.43	1.14	2
9	770	40.01	481	1.33	1.75	5.07	2
10	770	28.70	934	1.77	5.30	4.26	2
11	577	25.60	881	0.99	3.47	3.45	2
12	577	20.50	1374	6.20	9.29	4.12	2

Note: Samples 5, 6 and 9 loaded parallel to the bedding planes
Remaining samples loaded perpendicular to the bedding planes

Table 6. Classification of some Indiana shales by various systems.

No.	Geologic Formation	Gamble (1971) Durability	Gamble (1971) Plasticity	Deo (1972) Degradability	Franklin Trow Assoc. (1979) Shale Rating	ISHC Laboratory Number
1	Mansfield	V. Low	Medium	Soil-like	2.75	74-45878
2	"	Low	Medium	Soil-like	3.35	74-54684
3	Clore	V. Low	Medium	Soil-like	2.00	75-55653
4	Palestine	Low	Medium	Soil-like	3.30	74-54716
5	"	V. Low	Medium	Soil-like	2.50	74-54836
6	"	Medium	Medium	Soil-like	3.80	75-55044
7	Waltersburg			Soil-like		74-54767
8	Hardingsburg		Medium	Soil-like		73-51703
9	Big Cliffty		Medium	Soil-like		74-54973
10	Haney	Low	Medium	Soil-like	3.25	74-54972
11	Borden	Medium	Medium	Soil-like	4.30	75-55564
12	"	Medium	Medium	Soil-like		75-55316
13	"	Med. High	Medium	Rock-like		75-55315
14	New Providence	Medium	Medium	Soil-like	4.25	75-55731
15	"	Medium	Medium	Soil-like	4.22	75-55505
16	New Albany			Rock-like	9.00	74-54621
17	"	V. Low	Medium	Soil-like	1.85	75-55718
18	"	High	Medium	Rock-like		75-55486
19	"	V. High	Medium	Rock-like		75-55487
20	Kope	Low	Medium	Soil-like	3.14	75-55018
21	Dillsboro	Medium	na	Soil-like		75-55291
22	Whitewater	Low	Medium	Soil-like	2.98	76-55014
23	Hardinsburg	V. Low	Medium	Soil-like	2.25	73-51703
24	New Albany	V. High	Medium	Rock-like		74-54621
25	Mansfield	Medium	Medium	Soil-like	4.25	74-54684
26	Palestine	Low	Medium	Soil-like	3.25	74-54716
27	Kope	Low	Medium	Soil-like	3.20	75-55018
28	Borden	V. High	Medium	Rock-like		75-55315
29	New Providence	Med. High	Medium	Soil-like	5.77	79-55198
30	Osgood	Medium	Medium	Soil-like	3.21	79-55199
31	Borden (Attica)	Low	Medium	Soil-like	3.65	79-55204

Note: For more information on shales 1 to 22 see Bailey (1976).
For more information on shales 23 to 28 see Chapman (1975).
For more information on shales 29 to 31 see Hale (1979).

figure 4. For some of the shales tested by Bailey (1976) the tabulation is incomplete, as values of the 200-revolution single-cycle slake durability index for dry samples are not available. These are needed to estimate the two-cycle values by using the correlation equation presented earlier. The reason these values are not always available is evident from table 1, where it is seen that they are not used in the classification system adopted by Deo (1972) and followed by the Indiana State Highway Commission.

CONCLUSIONS

The characteristics of shales in regard to durability and strength is fundamentally important when the material is to be used in highway embankments. The plasticity index is a suitable descriptor for soft shales, but it is inappropriate for the harder ones. The slake durability index is not an adequate indicator of shale durability when used alone. It should be combined with a qualitative description of the strength of the pieces retained in the drum of the slake durability apparatus. The change in gradation of the retained pieces can also be measured and expressed as an index of slaking (similar to the index of crushing for mechanical degradation), which may be more appropriate than the slake durability index to quantify the durability of shales. More research needs to be done to study the effects of shape of test specimen on the slake durability index and the proposed index of slaking, particularly for irregular and cylindrical specimens. In regard to the point load strength index, it is clear that the size correlation chart proposed by the International Society for Rock Mechanics cannot be used for all rock types, and particularly not for all shales. It is probably necessary to standardize the dimensions, shape, and mode of loading of the test specimen, if this index is to be used for classification purposes.

ACKNOWLEDGMENTS

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THE TEXAS NATURAL RESOURCES INFORMATION SYSTEM

by

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INTRODUCTION

The Texas Natural Resources Information System (TNRIS) is a part of Texas government that indexes, stores, processes, and disseminates information about the State's natural resources. TNRIS is a cooperative effort among 13 of the State's natural resource-related agencies governed by a Task Force composed of one representative from each agency. A major TNRIS objective is to make natural resource and related data more easily accessible to individuals and organizations.

This document provides a brief history of TNRIS, an overview of its organization, a description of the variety of data types contained in the System, and an explanation of capabilities and services provided and System usage.

HISTORY OF TNRIS

TNRIS originated in the mid-1960's in response to legislation establishing the Texas Water Oriented Data Bank (TWODB). Eight State agencies participated in this project. Initial activities of the Data Bank included cataloging water-related data in concert with the U.S. Geological Survey's Office of Water Data Coordination. These efforts resulted in the first comprehensive inventory of water-oriented data in Texas State agencies.

In addition to assisting users to locate hydrologic information, TWODB eliminates duplication in data procurement. It was found, for example, that several State agencies were purchasing similar National Weather Service information on a recurring basis. Once identified, areas of duplication such as this were easily eliminated.

By the early 1970's it was apparent that a broader, more comprehensive information system was needed, and in 1972 work to establish the Texas Natural Resources Information System was begun. State agency participation was expanded to include 15 State agencies; representatives from each agency compose the TNRIS Task Force, whose purpose is developing the System. In 1977 the three State water agencies were merged, which reduced the number of participating agencies to 13.

SYSTEM ORGANIZATION AND OPERATION

TNRIS primarily serves the participating agencies by helping them carry out their functions. The System encourages coordination among member agencies by participating in joint projects aimed at developing new capabilities for natural resource management. This constitutes one of TNRIS' significant strengths. TNRIS was not designed to take away control, nor was it designed to be a massive data bank. Instead of attempting to centralize all natural resources data, TNRIS ties together the information systems already existing in the State and elsewhere in order to make the data more accessible.

The organization of TNRIS consists of: (1) a linked network of users who acquire and maintain natural resources data; (2) a staff which provides information on data availability, procurement, and analysis; and (3) a central computer facility to handle storage, retrieval, processing, analysis, and, where appropriate, presentation of natural resources data and information.

The 14-member TNRIS staff is housed within the Texas Department of Water Resources, which receives budgetary support from the State Legislature for administration of the System. It is made up of systems analysts, computer programmers, remote sensing specialists, and user service specialists.

The Task Force elects a chairman and vice-chairman biennially. The Task Force Secretary, designated by the Executive Director of the Department of Water Resources, is a staff member in the program area that includes TNRIS staff support.

The estimated expenditures relating to TNRIS establishment from 1969 through 1979 total $2.469 million. Personnel costs of participating agencies are estimated at $0.520 million of this total figure. TNRIS Systems Central staff personnel costs total $1.111 million. Other operating costs, including computer time and supplies, are about $0.838 million.

SYSTEM DATA

TNRIS data are indexed among the following six categories: hydrological, meteorological, geological, biological, socioeconomic, and base data (maps and remotely sensed data). Currently more than 300 automated files and over 150 manual files are indexed in the System.

Files available online include daily and monthly precipitation, minimum temperature, maximum temperature, wind movement, and pan evaporation data from the National Weather Service and State agencies. Other online data files are: State agency gross and net lake-surface evaporation and daily and monthly relative humidity data; daily and monthly precipitation and evaporation data from the United States Section of the International Boundary and Water Commission; State agency coastal-zone biological information; and daily and monthly USGS streamflow data. Daily and monthly State agency suspended sediment-load data, and daily USGS surface-water temperature and surface-water conductance data are accessible through the Monitor System described in the section on System usage.

Additional files include State agency coastal-zone hydrographic information, USGS surface-water quality data, daily and monthly United States Section International Boundary and Water Commission streamflow, sediment-load, reservoir content, and daily and monthly USGS reservoir-content data. State agency ground-water quality and water-level measurement data from observation wells throughout Texas are likewise available online through remote terminal access.

As previously mentioned, TNRIS has frequent contact with a number of Federal data systems. For example, TNRIS is an assistance center for USGS's National Water Data Exchange (NAWDEX) and was the first state affiliate of the National Cartographic Information Center. Since early 1976, TNRIS has been a Summary Tape Processing Center recognized by the U.S. Census Bureau.

Additional Federal system contacts are maintained with (1) the Environmental Protection Agency's Storage and Retrieval System (STORET); (2) the USGS's Water Data Storage and Retrieval System (WATSTORE), and EROS Data Center (EDC); (3) the U.S. Department of Commerce's National Technical Information Service (NTIS), and National Weather Service (NOAA-NWS); (4) the U.S. Department of Agriculture's Federal Assistance Programs Retrieval System (FAPRS); and (5) the Water Resources Council's Water Resources Scientific Information Center (WRSIC).

Bibliographic data are available to TNRIS users through two online private sector sources: the System Development Corporation's ORBIT system and the Lockheed Corporation's DIALOG system. Access to data and information from certain Federal systems is accomplished through private contractors.

SYSTEM CAPABILITIES AND SERVICES

TNRIS capabilities and services are varied and extensive. They include (1) computer-printed reports; (2) graphic output; (3) interface with remote terminals; (4) statistical packages; (5) computer-generated microform; (6) geocoding/geographic information handling; (7) analysis of remotely-sensed data; (8) catalogs/indexes; (9) responses to inquiries concerning the availability of (a) computerized data, (b) aerial photography, (c) satellite imagery/data, (d) cartographic products, and (e) technical publications; and (10) ordering services.

TNRIS indexes sensed, monitored, measured, and collected data existing in both machine-processable form as computer cards, tapes, and disks and in non-machine-processable form existing as documents, maps, and remotely sensed imagery. In addition to providing data from its own centralized information base, TNRIS disseminates data from other data bases with which it is interconnected, refers inquiries to other data sources, and adjusts and organizes data into forms suited to storage/retrieval and analysis. It also provides services for manipulating and processing data into graphic representations, models, and study plans. TNRIS may lead to the development of specifications and inventorying and monitoring systems for natural resources management.

An ongoing educational program on TNRIS includes short courses on methods of accessing the TNRIS data files and on other aspects of the system. TNRIS regularly publishes several items, including a newsletter.

TNRIS remote sensing/cartographic activities can be classified into four areas: (1) indexing and cataloging, (2) data retrieval, (3) education and consultation, and (4) data analysis. As a state-level affiliate of the National Cartographic Information Center (NCIC), TNRIS has been indexing all known sources of imagery for the State, including Federal and State agencies, universities, and the private sector.

In retrieving data on remote sensing and cartography, TNRIS assists many users in procuring imagery and map data. The TNRIS computer terminal interface with the EROS Data Center and the 16 mm browse file of the Data Center's principal holdings are particularly helpful. This equipment makes several hundred thousand frames of imagery covering the State of Texas available to TNRIS users.

In education and consulting, TNRIS offers several short courses in image interpretation and remote sensing for State and private agency personnel. A four-part series given throughout the year includes "Fundamentals of Remote Sensing," "Air Photo Interpretation," "Landsat Image Interpretation," and "Principles of Landsat Digital Processing."

In data analysis of remote sensing and cartography, TNRIS has been involved in a number of activities using Landsat data to assist Texas State agencies in natural resource-related projects. One such project led to use of Landsat digital data for updating the Dam Inventory maintained by the Texas Department of Water Resources. TNRIS has also been involved in a long-term project to develop a generalized Geographic Information System for storing and reproducing map-related data. The System manipulates data in topological (polygonal) form as well as grid cell form.

Using the Geographic Information System, data can be extracted from cartographic products in the form of areas, lines, and points; stored on computer files, along with textual information associated with the data; and reproduced in the form of map overlays. Base information such as soil type locations, biologic assemblies, oil and gas wells, pipelines, highway locations, and dam locations have been stored in the System. Using the TNRIS Geographic Information System, a TNRIS user can extract data from several different base maps at any scale or projection and can subsequently combine them on a single plot.

SYSTEM USAGE

As TNRIS evolved, it has come to serve quite a large user community, including State agencies, industries, individuals, private businesses, educational institutions, municipalities, county governments, councils of governments, river authorities, water districts, and Federal agencies. The basic goal of the TNRIS Systems Central staff is service to the users. TNRIS staff assist users in obtaining natural resources data, either that readily available through computer linkages, or that which must be retrieved from material located in several different places.

As TNRIS has developed, its staff has provided users with quick turnaround on data requests submitted by letter, telephone, or personal visit to TNRIS Systems Central. Depending on the form of the request and the source of the data, answers may be provided the same day or within several days.

To assist users who require direct, immediate access to data, the TNRIS Monitor has been developed. The Monitor is a system by which users can access the data base through remote terminals in different parts of the State. It is designed for easy operation by non-technical users. It provides ready access to over 50 percent of the more than 300 TNRIS automated natural resource and related data and information files. Files accessible through the Monitor include those held by the TNRIS Systems Central facility and also those available from other entities on computers located at a distance from TNRIS which can be linked up to users through TNRIS computer facilities.

TNRIS Monitor users receive top computer priority to ensure fast turnaround. Previous experience at using computer terminals is not necessary in order to use the Monitor. Monitor users mistakenly asking for data they do not want may revise their requests.

During the year ending August 31, 1979 (State fiscal year 1979) 772 users made 2,546 different data requests generating 3,494 accessions to TNRIS files. These accessions produced a total of 4,088 sets of output products. Output formats include: one or more computer-printed reports in 1,914 instances; magnetic tapes in 190 intances; 59 sets of punched cards; one or more graphs or plots in 76 instances; 615 aerial photography indices; various numbers of aerial-photo paper prints in 295 instances; one or more aerial-photo transparencies in 66 instances; varying quantities of microfilm output in 10 cases; and 863 outputs in other format (file documents, published reports, maps, etc.). About 13 percent of the accessions to TNRIS results in more than one type of output product; in many instances a single accession created large numbers of one type of output product. Eighty-nine percent of the data requests for fiscal year 1979 were made either by telephone, letter, or in person, while the remainder were processed by computer terminal.

CONCLUSION

Over the years the TNRIS has been effective in making information more readily available, in reducing duplication in data acquisition, and in fostering cooperation among Texas State agencies. The success of the System may be attributable to a number of factors; most important among them is perhaps the obvious need for and benefits of such a system. It is also noteworthy that the System has evolved slowly rather than being developed full blown over a short period of time. It is also important that the System uses existing facilities instead of additional expensive computer equipment.

In response to numerous requests for information on TNRIS, a considerable amount of documentation is available on the System. Inquiries should be made to TNRIS, P.O. Box 13087, Austin, Texas 78711.

THE GEOTECHNICAL DATA BANK

by

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INTRODUCTION

Geotechnical information generated by field and laboratory measurements for various geotechnical projects in Indiana exists in subsurface investigation reports and is retained in the files of public agencies and private consultants. To encourage use of this information, a computerized, user-oriented, information storage and retrieval system has been developed (Goldberg, 1978). This data bank has many uses, including planning, route or site selection, design, construction, and teaching and research. Since the body of data as well as the number and types of users will increase with time, every effort has been made to keep the system both inclusive and flexible.

The Indiana data have been collected from private consulting firms, private soil testing firms, and from tests conducted by the Indiana State Highway Commission (ISHC). All of the data collected to date (some 10,000 sets) were generated for the purpose of building modern highways and bridges in the state. The data bank is quite new; this is the second of a series of papers (Goldberg and others, 1979) to discuss various uses with a number of potential user groups.

Two principal types of information are generated with a geotechnical data bank: distributions of materials parameters and correlations among these parameters. Common statistics of the distributions are of particular interest in sensitivity analyses and in probabilistic models. Correlations have multiple uses also: among these are the predictions of parameters values, which are relatively few (because of the cost of obtaining them) from more simple (and abundant) values. Common examples of the latter are predictions of strength and compressibility from classification tests.

Data Stored

The Data Input Form (DIF) of figure 1 was developed to fit the data sources for the Indiana bank. Not all items are available for all samples, but the listing includes the following:

(1) Project identification -- a. project number, b. contract number, c. road number, d. data collection agency;

(2) Sample location -- a. county, b. highway district, c. township, d. range, e. section, f. line number, g. station number, h. offset and the left or right direction from the center line;

(3) Sample identification -- a. boring number, b. laboratory number, c. sampling procedure;

(4) Date the sample was taken from the hole;

(5) Physiographic region;

(6) Parent material from which the soil has been derived;

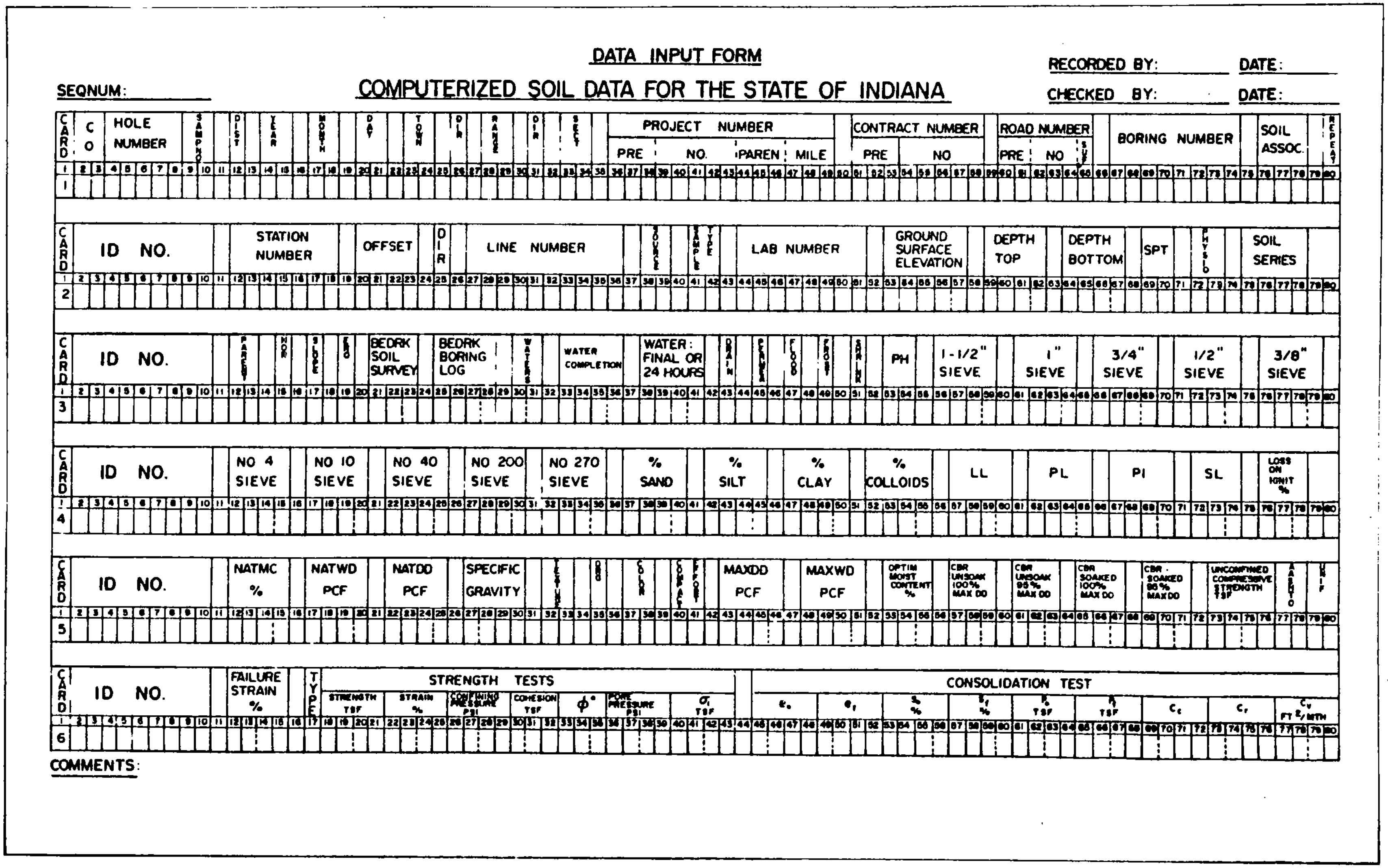
(7) Ground surface elevation;

(8) Depth from which the sample has been removed;

(9) Depth to the bedrock;

(10) Depth to groundwater;

(11) Standard penetration resistance (SPT);


DATA INPUT FORM

COMPUTERIZED SOIL DATA FOR THE STATE OF INDIANA

SEQNUM:

RECORDED BY: DATE:

CHECKED BY: DATE:

CARD 1: CO; HOLE NUMBER; SAMPNO; DIST; YEAR; MONTH; DAY; TOWN; DIR; RANGE; DIR; SECT; PROJECT NUMBER (PRE, NO., PAREN, MILE); CONTRACT NUMBER (PRE, NO); ROAD NUMBER (PRE, NO, SUF); BORING NUMBER; SOIL ASSOC.; REPEAT

CARD 2: ID NO.; STATION NUMBER; OFFSET; DIR; LINE NUMBER; SOURCE; SAMPLE TYPE; LAB NUMBER; GROUND SURFACE ELEVATION; DEPTH TOP; DEPTH BOTTOM; SPT; PHYSIO; SOIL SERIES

CARD 3: ID NO.; PARENT; HOR; SLOPE; ERO; BEDRK SOIL SURVEY; BEDRK BORING LOG; WATERS; WATER COMPLETION; WATER: FINAL OR 24 HOURS; DRAIN; PERMEA; FLOOD; FROST; SWELL; PH; 1-1/2" SIEVE; 1" SIEVE; 3/4" SIEVE; 1/2" SIEVE; 3/8" SIEVE

CARD 4: ID NO.; NO 4 SIEVE; NO 10 SIEVE; NO 40 SIEVE; NO 200 SIEVE; NO 270 SIEVE; % SAND; % SILT; % CLAY; % COLLOIDS; LL; PL; PI; SL; LOSS ON IGNIT %

CARD 5: ID NO.; NATMC %; NATWD PCF; NATDD PCF; SPECIFIC GRAVITY; TEXTURE; ORG; COLOR; COMPACT; FROST; MAXDD PCF; MAXWD PCF; OPTIM MOIST CONTENT %; CBR UNSOAK 100% MAX DD; CBR UNSOAK 95% MAX DD; CBR SOAKED 100% MAX DD; CBR SOAKED 95% MAX DD; UNCONFINED COMPRESSIVE STRENGTH TSF; AASHTO; UNIF

CARD 6: ID NO.; FAILURE STRAIN %; TYPE; STRENGTH TESTS (STRENGTH TSF, STRAIN %, CONFINING PRESSURE PSI, COHESION TSF, ϕ°, PORE PRESSURE PSI, σ_1 TSF); CONSOLIDATION TEST (e_o, e_f, S_o %, S_f %, P_o TSF, P_c TSF, C_c, C_r, C_v FT²/MTH)

COMMENTS:

Figure I. Data input form (DIF)

(12) Pedological soils information -- a. soil association name, b. soil series name, c. horizon, d. slope (topographic) class, e. erosion class, f. natural soil drainage class, g. generalized permeability, h. generalized flooding potential, i. generalized frost heave susceptibility, j. generalized shrink-swell potential, k. generalized pH;

(13) Gradational characteristics based on standard sieve sizes and hydrometer analysis;

(14) Atterberg limits;

(15) Visual textural classifications;

(16) Color based on moist conditions;

(17) Organic content (loss on ignition);

(18) In-situ moisture content;

(19) In-situ dry and wet densities;

(20) Specific gravity;

(21) Compaction test results;

(22) California bearing ratio (CBR);

(23) Unconfined compressive strength and failure strain;

(24) Strength data from triaxial and direct shear tests; and

(25) Consolidation test results.

Using the above information from (13) to (16), a computer program classifies the samples by the American Association of State Highway and Transportation Officials (AASHTO) and Unified Soil Classification (UNIF) Systems (Goldberg, 1978).

A user's manual, explaining in detail the operation of this data storage and retrieval system, is now available (Goldberg, 1978), and a revised manual is in preparation.

Uses of Data Bank

As mentioned previously, data bank uses can be divided into two categories, the reduction of raw data to a usable form via descriptive statistical methods, and the generation of correlation and prediction equations via statistical inference. Each use will be illustrated by examples.

Reduction of Raw Data

To measure the variability of selected soil characteristics numerically, the frequency distributions of these characteristics are examined and described. The distributions are sometimes normal but frequently they are skewed or bimodal. This suggests that non-parametric (distribution-free) methods be employed. In fact, Snedecor and Cochran (1969) state that when the investigator does not know the type of distribution being sampled the nonparametric methods are always helpful. They are highly efficient relative to classical techniques under the assumption of normality and are often more efficient in other situations (Hollander and Wolf, 1973; Larson, 1973). A complicated distribution is that of the ground water level; table 1 shows this sort of analysis for the total state and for some of the 13 physiographic regions within the state.

Table 2 summarizes an ambitious effort to reduce distributional data on all possible soil parameters to "statistical soil profiles." The best population grouping for this purpose seems to be the pedologic soil association. Another approach is shown in table 3, where the population groups are AASHTO classifications within physiographic units. The primary purpose of each of these tables is to give expected values for reasonably large but grossly homogeneous soil groupings by non-parametric estimates, such as median and other order statistics.

Table 1. Ground water level with respect to ground level (ft)

Physiographic region	Mean	s.d.	Median	95% C.I. of median	$t_{.25}$*	$t_{.75}$**	I.R.***	Minimum	Maximum
Whole state (2395 cases)	5.40	4.80	4.33	4.13--4.53	2.11	6.73	4.62	0.10	60.00
Tipton Till Plain (834 cases)	5.16	3.41	4.10	3.86--4.37	2.50	6.12	3.62	0.20	20.00
Dearborn Upland (89 cases)	4.93	2.96	4.21	3.54--4.86	2.54	5.86	3.32	0.50	15.00
Norman Upland (83 cases)	7.67	5.52	6.54	5.78--7.23	4.44	8.50	4.06	0.50	17.00
Mitchell Plain (140 cases)	8.54	7.22	6.29	5.61--7.03	4.33	8.84	4.51	1.00	38.00
Crawford Upland (144 cases)	8.77	8.01	6.45	5.70--7.18	4.07	10.00	5.93	0.50	60.00
Wabash Lowland (348 cases)	5.88	5.38	4.12	3.75--4.61	2.54	7.06	4.52	0.10	37.00
Calumet Lacustrine Section (204 cases)	5.84	5.63	4.08	3.45--4.82	1.89	8.07	6.18	0.20	35.00
Valparaiso Moraine (150 cases)	2.26	1.72	1.86	1.55--2.28	0.94	3.31	2.37	0.10	10.00
Steuben Morainal Section (335 cases)	3.98	3.81	2.96	2.59--3.33	1.39	4.92	3.53	0.10	35.00

* 25% quartile ** 75% quartile *** Interquartile Range

Table 2. Typical soil profile for soil association.

Fincastle-Ragsdale-Brookston

General description: Nearly level, somewhat poorly drained, silty Fincastle in windblown silt and glacial till, very poorly drained, silty Ragsdale in windblown silts and loamy Brookston in glacial till.

Parent material: Soils formed in moderately thick loess deposits over loamy Wisconsin age glacial till.

Distributions: Physiographic region: Tipton Till Plain
Counties: Tippecanoe, little in Clinton

Ground elevation:

	Mean	s.d.	Median	$t_{.25}$	$t_{.75}$	I.R.	95% C.I. of Median	Minimum	Maxi-mum	N. of Cases
	728.12	70.26	708.33	673.81	788.88	115.07	685.71--761.11	645.90	866.80	47

Ground water level:

Season	Mean	s.d.	Median	$t_{.25}$	$t_{.75}$	I.R.	95% C.I. of Median	Minimum	Maxi-mum	N. of Cases
Total	723.17	69.76	708.33	672.50	785.00	112.50	685.00--700.00	641.70	854.90	47
Winter	846.68	13.85	875.00	850.00	883.33	33.33		826.10	854.90	4
Spring	759.11	62.15	725.00	695.00	806.25	111.25	700.00--800.00	662.60	808.90	17
Summer	653.26	7.16	671.43	653.57	689.29	35.72	650.00--663.00	641.70	663.50	9
Fall	695.18	36.03	700.00	671.87	737.50	65.63	675.00--733.33	642.50	767.30	17

Ground water level with respect to ground level:

Season	Mean	s.d.	Median	$t_{.25}$	$t_{.75}$	I.R.	95% C.I. of Median	Minimum	Maxi-mum	N. of Cases
Total	4.95	2.67	4.00	2.91	6.00	3.09	3.36--5.17	1.60	11.90	47
Winter	9.53	4.36	11.00	4.00	11.33	7.33	10.00--11.90	3.00	11.91	4
Spring	3.33	1.38	3.17	2.42	3.92	1.50	2.50--3.83	1.60	6.00	17
Summer	4.42	0.87	4.86	4.14	5.57	1.43	4.00--5.40	3.00	5.40	9
Fall	5.78	2.40	6.00	3.28	7.28	4.00	3.43--7.14	2.50	10.00	17

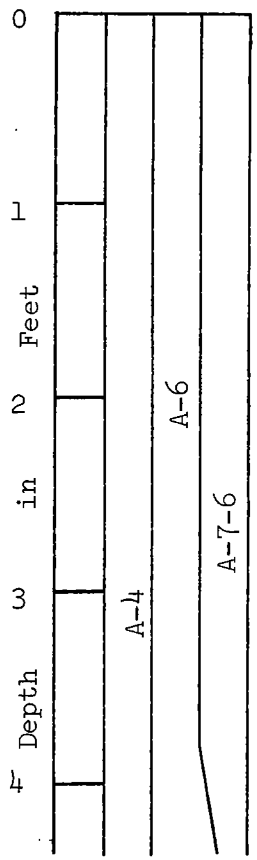


Table 2 (continued). Typical soil profile.

Unified classification: CL Texture: clay and silty loam

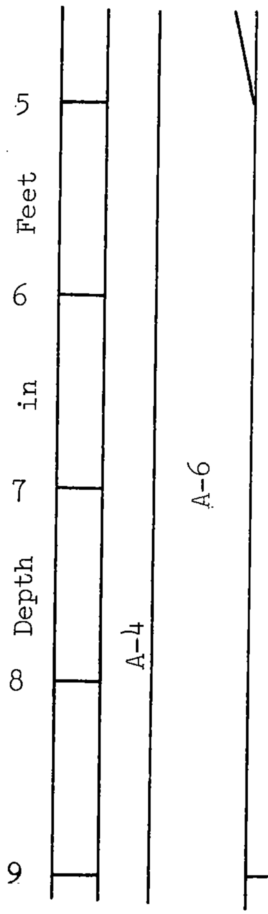
PH. 5.10 - 6.00 Organic material: not traceable

Unified classification: CL, CH Texture: clay, silty clay and silty clay loam

PH: 5.60 - 6.00 Organic material: not traceable

Variable	Mean	s.d.	Median	$t_{.25}$	$t_{.75}$	I.R.	Min.	Max.	No. of Cases
SPT	5		5						1
SL	15.82	4.18	15.16	12.88	17.89	5.01	2.00	29.00	102
NATMC	23.00	8.11	22.50	17.78	27.67	9.89	8.00	65.00	67
NATDD	93.37	6.50	92.40	85.00	97.50	12.50	84.60	98.40	4
SPECGR	2.72	0.03	2.72	2.70	2.75	0.05	2.67	2.77	6
MAXDD	105.18	13.23	104.00	100.50	117.50	17.00	80.00	125.00	13
OMC	21.65	13.42	20.00	15.50	24.50	9.00	10.00	63.00	13
CBRS01	7.86	2.14	7.67	6.50	10.33	3.83	4.50	11.30	13
CBRS02	4.71	1.68	4.67	3.25	5.83	2.58	2.00	9.00	13
Qu	1.43	0.49	1.20				1.10	2.00	3

Table 2 (continued).

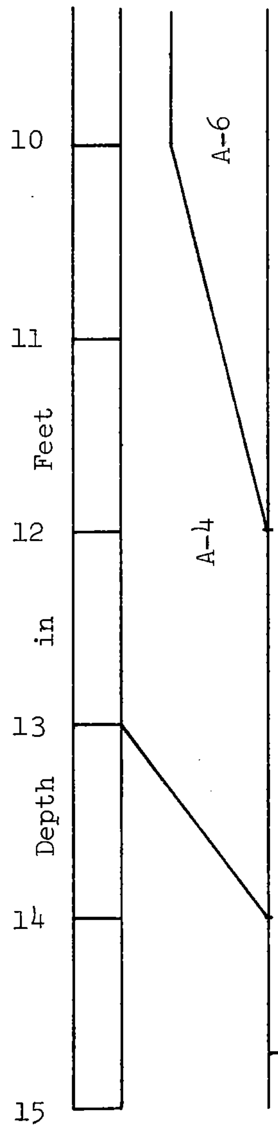


Unified classification: CL, CL-ML　　　Texture: clay loam and sandy loam

PH: 6.10 - 6.50　　　Organic material: not traceable

Variable	Mean	s.d.	Median	$t_{.25}$	$t_{.75}$	I.R.	Min.	Max.	No. of Cases
SPT	7	4	7				5	10	2
SL	13.38	3.32	13	13.00	10.90	4.02	1.90	24.00	45
NATMC	21.50	11.12	18.54	15.00	23.57	8.57	11.00	65.00	34
NATDD	86.13	7.03	85.30	80.00	90.00	10.00	77.60	93.50	4
SPECGR	2.72	0.007	2.72				2.71	2.72	2
MAXDD	125.65	0.50	125.65				125.30	126.00	2
OMC	10.25	0.35	10.25				10.00	10.50	2
CBRSO1	6.60		6.60						1
CBRSO2	3.90		3.90						1
Qu	0.67	0.115	0.65				0.60	0.80	3

Table 2 (continued).



Unified classification: CL   Texture: clay, silty clay loam, and sandy loam

PH: 7.40 - 7.80   Organic material: not traceable

Variable	Mean	s.d.	Median	$t_{.25}$	$t_{.75}$	I.R.	Min.	Max.	No. of Cases
SPT	63	23	63				47	80	2
SL	13.69	3.35	13.50	10.13	16.50	6.37	9.00	18.00	13
NATMC	17.82	6.49	18.12	12.50	25.00	12.50	10.00	27.00	12
NATDD	103.00	2.48	103.33	100.83	105.00	4.17	99.80	106.20	5
SPECGR	2.76	.021	2.76				2.74	2.77	2
MAXDD	128.45	0.50	128.45				128.10	128.80	2
OMC	10.20	0.14	10.20				10.10	10.30	2
CBRS01	9.50	2.12	9.50				8.00	11.00	2
CBRS02	5.50	0.71	5.50				5.00	6.00	2
Qu	0.70	0.28	0.70				0.50	0.90	2

Note:

SPT-Standard penetration resistance
NATMC-Natural moisture content (%)
SPECGR-Specific gravity
OMC-Optimum moisture content (%)
Qu-Unconfined compressive strength (TSF)

SL-Shrinkage Limit (%)
NATDD-Natural dry density (PCF)
MAXDD-Maximum dry density (PCF)
CBRS01-CBR soaked value at 100% maximum dry density
CBRS02-CBR soaked value at 95% maximum dry density

Table 3. Characteristics of soil properties obtained from disturbed soil testing within Tipton Till Plain

Physiographic region: Tipton Till Plain
AASHTO classification: A-4
Texture: sandy loam, clay loam
Unified classification: CL, CL-ML

Variable	Mean	s.d.	Median	$t_{.25}$	$t_{.75}$	I.R.	95% C.I. of median	Minimum	Maximum	No. of Cases
NATDD	104.33	21.51	106.17	94.14	120.75	26.61	100.57--111.17	12.40	132.00	41
SPECGR	2.70	0.046	2.69	2.66	2.73	0.07	2.66--2.73	2.64	2.77	13
SL	14.05	3.28	13.23	11.21	15.65	4.44	12.89--13.57	0.10	26.00	552
MAXDD	120.53	7.12	122.26	115.76	126.29	10.53	120.00--124.52	98.20	130.70	49
OMC	12.27	3.04	11.25	10.00	13.43	3.43	10.55--15.95	8.30	24.70	48
CBRSO1	10.38	6.61	8.20	6.30	12.50	6.20	7.00--10.00	3.00	42.10	37
CBRSO2	6.07	2.70	5.00	3.33	7.67	4.34	3.87--6.33	2.00	14.00	39

Data Correlations and Prediction Models

Two types of prediction equations were used: (1) median models, and (2) regression models. The median model was used to describe the data structure or distribution, rather than the conventional constant mean model. The form of the median model is

$$x_i = t_{.50} + \varepsilon_i ,$$

where $t_{.50}$ is the median value and ε_i is a random variation.

If the distribution is asymmetrical, the median model gives more reasonable values than the mean model in most cases. If the distribution is symmetrical about its mean, the two models give essentially the same value. When the objective is to predict, the value of $\varepsilon_i \rightarrow 0$, or $\hat{X}_{i + k} = t_{.50}$.

Only past values of the variable being predicted are used in the median model, and a considerable population of values for this variable is required. Regression models define empirical relationships among variables and may be based on limited data fields. Regression models often correlate the soil properties that are difficult or costly to measure with simple soil indices.

Regressions for certain dependent variables for the Indiana soil data have been undertaken and reported by Goldberg (1978) and Goldberg and others (1979). These include: (1) compression index (C_c) and compression ratio (C_r, which equals $C_c/(1 + e_o)$, where e_o is the initial void ratio); (2) unconfined compressive strength (q_u); and (3) standard Proctor maximum dry ($\rho_{d_{max}}$) and wet ($\rho_{m_{max}}$) densities and optimum moisture content (w_{opt}). Additional correlations for the compression index are shown in figures 2 and 3.

The independent variables uses in these regressions included: (a) initial void ratio (e_o), natural moisture content (w_n), natrual dry density (ρ_d); (b) liquid limit (w_L), plastic limit (w_p), plasticity index (I_p), shrinkage limit

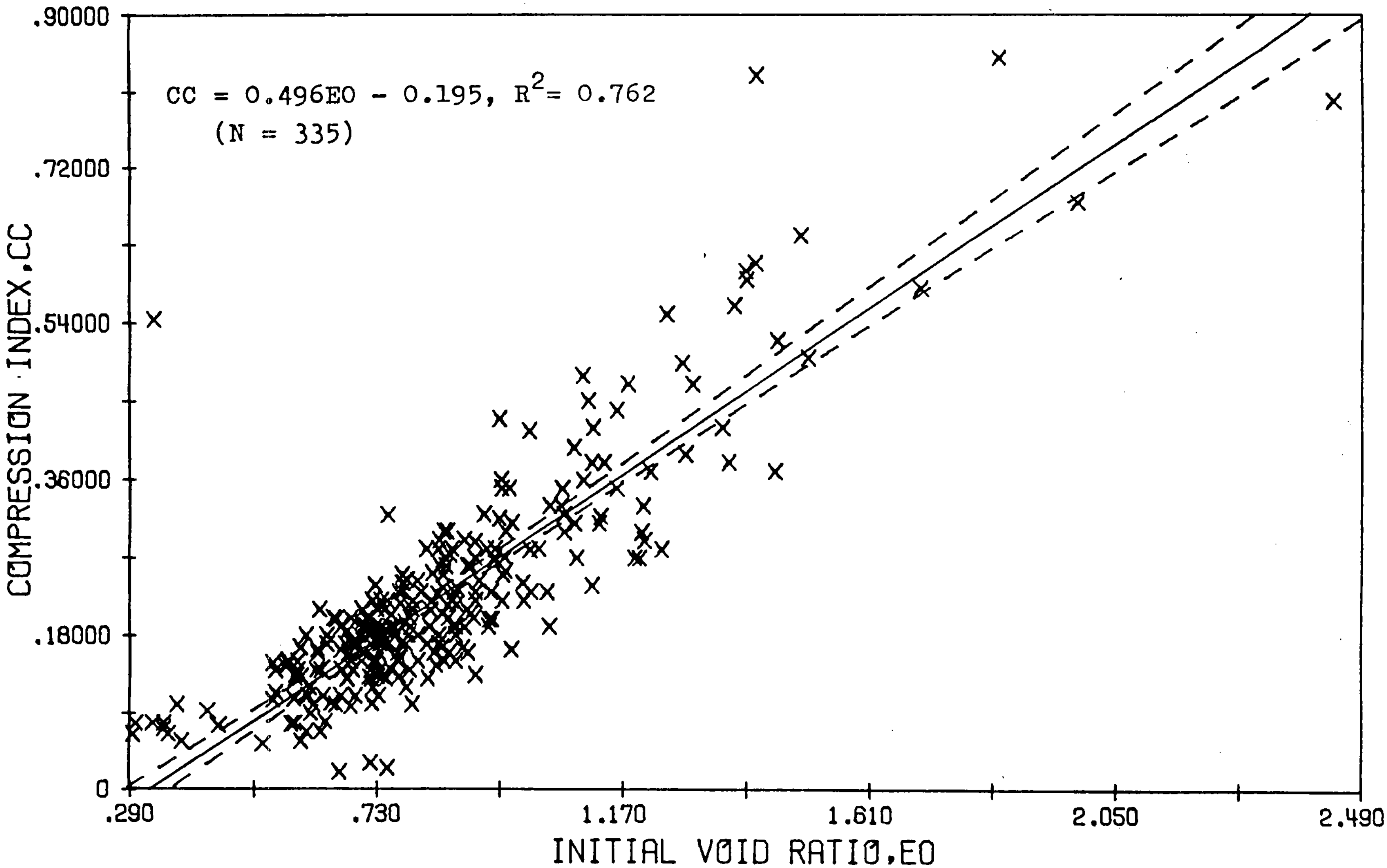


Figure 2. Regression line and 95% confidence interval of compression index with initial void ratio for Indiana soils

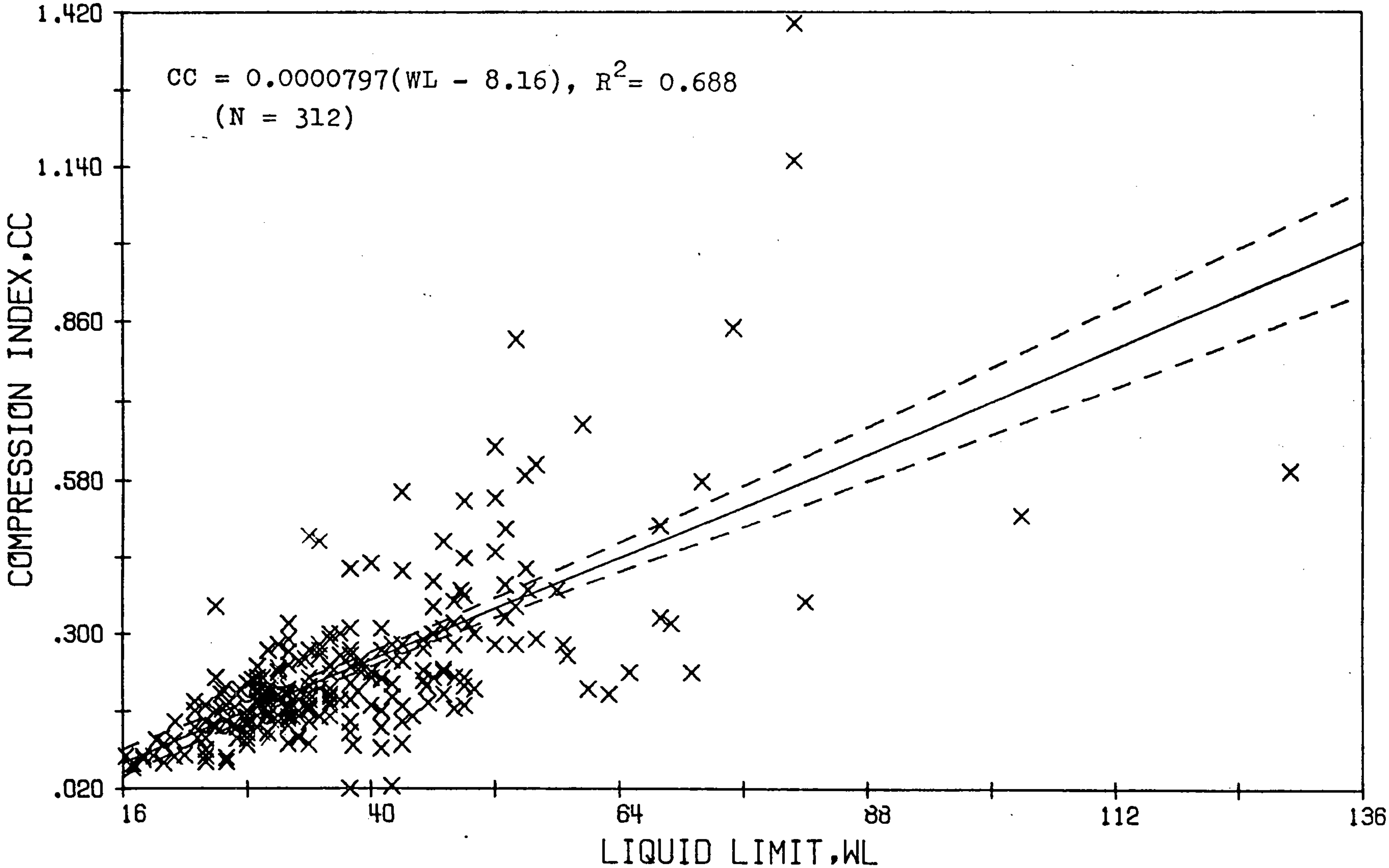


Figure 3. Regression line and 95% confidence interval of compression index with liquid limit for Indiana soils

(w_s), and (c) percent sand, percent silt, and percent clay.

SUMMARY

The data bank provides secure storage of geotechnical information generated for a specific project and often lost after project completion. Data may be retrieved by many users for a variety of purposes.

In some instances, the non-parametric statistical methods are more satisfactory than the classical ones for describing the distributional characteristics of soil parameters. Examples of distributions have been given for ground water levels within physiographic subsections, statistical soil profiles within pedologic soil associations, and selected soil parameters within the same AASHTO soil classification group in a physiographic unit.

Some regression models have also been presented. As more data are stored, it is possible to improve the statistical credentials of coarsely divided data groupings and to develop correlations for smaller and more homogeneous groupings.

ACKNOWLEDGMENTS

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ENGINEERING PETROGRAPHY: HIGHWAY APPLICATIONS

by

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INTRODUCTION

Petrography is the study of the classification and composition of rocks. In a general sense, engineering petrography attempts to explain the behavior and to relate the engineering properties of construction materials such as rock, soil, cement, lime, concrete, fabrics, metals, and plastics to optical characterization by means of a microscope. For the purpose of this paper, engineering petrography involves the application of petrography to the examination of construction materials, both natural and synthetic, used in engineered structures.

Construction materials are required to comply with specifications relevant to the job. Testing is necessary to determine if a material meets specification requirements. Often the reason materials pass or fail a test can be determined through microscopic examination. Most inorganic substances (minerals) have crystalline structures that can be identified optically. Even the mineraloids, amorphous glasses, and inorganic compounds display optical features that can be recognized and compared using optical (polarizing or petrographic) microscopes. A stereoscopic microscope, equally indispensable in a petrographic laboratory, is shown along with a polarizing microscope in figure 1.

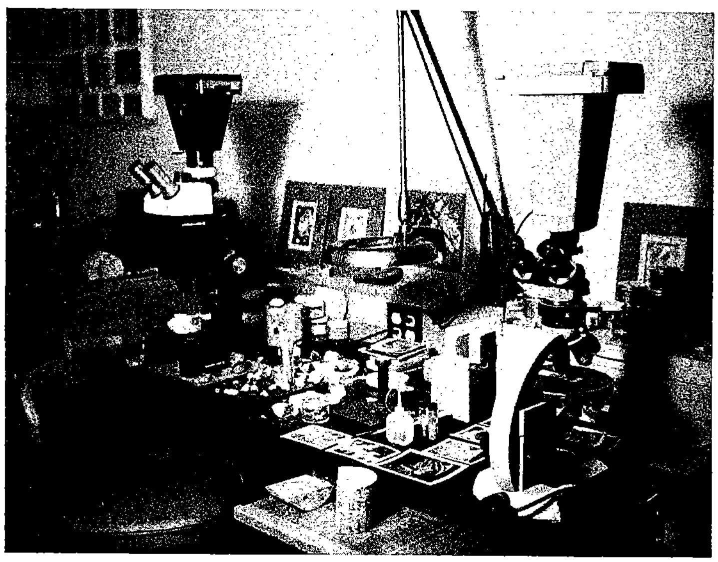


Figure 1. A polarizing microscope (right) equipped with transmitted- and reflected-light accessories along with a stereoscopic microscope (left) are essential for petrographic analysis of constructional materials.

The petrographic microscope can be used to identify rocks and concrete, paint pigments, and extenders, such as asbestos, fiberglass, mica, and an assortment of other common minerals and additives, such as fly ash for concrete. Various thermoplastic fabrics such as polypropylene, nylon, and polyester exhibit optical features under polarized light that aid identification and provide the basis for comparative studies.

HIGHWAY APPLICATIONS

Bridge Deck Deterioration

Petrographic analysis was first applied to highway materials by the Department's Materials and Tests Division during the 1960's. The Division's research section undertook an extensive bridge-deck survey to determine the causes of deterioration in Texas structures and to recommend corrective measures (Elmore, 1967). As a part of this project, 224 concrete cores were subjected to petrographic examination. Results of these petrographic studies were combined with other laboratory and field observations to provide significant information about distressed concrete.

Industrial Mineral and Resource Assessments

In 1968, the Materials and Tests Division established a geologic and petrographic research section to undertake a thorough assessment of our statewide aggregate sources and to provide a complete petrographic laboratory equipped for microscopic examination of all types of aggregate material.

By the early 1970's, several studies had been conducted as industrial mineral and specific aggregate assessments (Patty, 1968a, 1968b, 1971, 1972, 1973). Each study incorporated petrographic analysis as a basis or as a complementary part of the study. Figures 2 and 3 demonstrate a typical use of transmitted polarized light for identifying sand-grain mineralogy as well as the nature of the cementing compounds in a sandstone.

In searching for, testing, and evaluating new deposits of rock materials for proposed highway use, petrographic studies have been applied to limestones, rhyolite, scoria, sandstone, caliche, and river gravels in an effort to explain and predict their behavior when used as coverstone, hot-mixed asphaltic paving materials, asphalt stabilized base, and portland cement concrete (Patty, 1971, 1974a, 1974b).

Reactive Aggregate

Studies of the petrography of portland cement concrete reached a turning point in 1940 when failure and deterioration of hardened concrete were found to be the result of reactive aggregate. Since the early 1950's certain types of aggregate from West Texas have been suspected to cause alkali-silica reactions in concrete structures (Holland and Cook, 1953; Mielenz, 1954). In 1970, Patty (unpublished report) identified evidence of alkali-silica reaction from three IH-10 structures in West Texas that had been open to traffic for only five or six years. Subsequent studies found evidence of reactive aggregate in other areas of the State. Silica gel deposits from alkali-reactive siliceous aggregates are illustrated in figure 4. An example of a mechanically unsound aggregate is shown in figure 5.

Specific petrographic studies applied to concrete aggregate include the identification of iron pyrite and lignite as potentially deleterious to the physical and aesthetic qualities of structural concrete (Patty, 1970). Results from this examination were used to denote objectionable mineralogical and organic materials. When these results were coupled with field performance,

Figures 2 and 3. Photomicrographs of sandstone aggregate proposed for use in an asphaltic paving mixture as viewed in plain light (top) and polarized light (bottom). The sand grains are composed of quartz, feldspar, and chert whereas the cementing materials are opal and fibrous chalcedony. (Magnification 125X)

Figure 4. Alkali-silica deposit formed as the result of certain siliceous aggregates chemically reacting with the alkalies in portland cement concrete. Expansive forces capable of disrupting and cracking concrete may result from these reactions. (Magnification 25X)

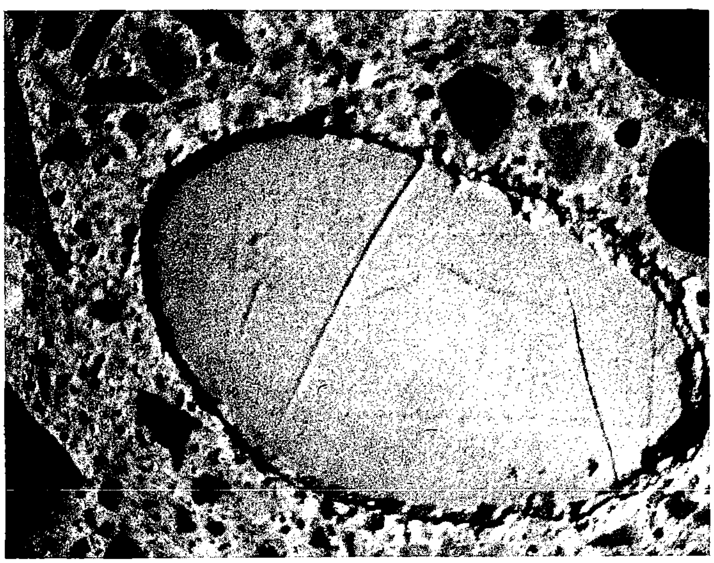


Figure 5. Mechanically unsound rock materials incorporated in concrete mixtures can have pronounced effects on strength and durability of structures. Aggregate particle exhibiting high shrinkage characteristics which disrupt the bond with the cement paste. (Magnification 8X)

recommendations were made to restrict the use of aggregates with these deleterious components in certain concrete mixtures.

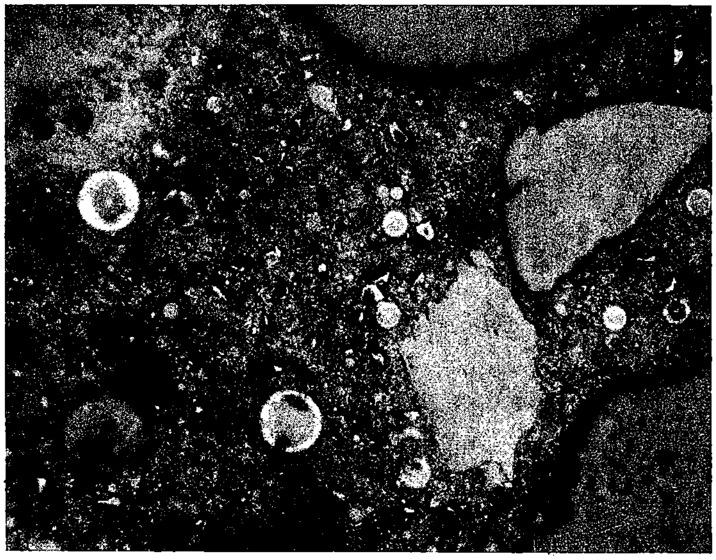
Expansivity, Secondary Mineralizations, and Admixtures

Application of petrography to samples from concrete structures has also revealed conditions associated with unsound cement (Patty, 1972b). Unaccommodative expansion of the cement paste resulted in a network of microcracks and complete disruption of bridge deck slabs and retaining walls.

Condition of the cement paste, air-void system, aggregate-bond interface, relict cement particles, hydration products, solid admixtures (such as fly ash), and secondary or abnormal chemical deposits and mineralization are well defined through petrographic observations and examinations. Figures 6 and 7 illustrate fly ash admixtures as found in concrete. A well-developed hexagonal crystal of portlandite (calcium hydroxide) removed from an air void in concrete with higher than normal water cement ratio is shown in figure 8.

Other in-depth reports on reactive aggregate have relied on petrographic support for cause-and-effect relationship of hostile environments, such as the Gulf waters, and incorporation of synthetic glass into concrete batches (Patty, 1973, 1974a).

In addition to the problems of concrete deterioration from alkali reactivity, documentation of abnormal chemical reaction from sulfate attack has been delineated through petrographic analysis. Radially formed needle-like crystals of calcium sulfoaluminate taken from a sample of deteriorated concrete caused by sulfate reaction are shown in figure 9.



Figures 6 and 7. Industrial by-products from power generating plants in the form of spherical glass fly ash particles have found uses as pozzolanic admistures in portland cement concrete. The glassy beads, about the size of cement grains, can have subtle as well as pronounced effects on the properties of concrete mixtures. (Vertical illuminated reflected light, magnification 200X and 400X)

Figure 8. Hexagonal crystals of portlandite (calcium hydroxide) form in void spaces in portland cement concrete mixes generally when excessive water is used during batching. (Magnification 500X)

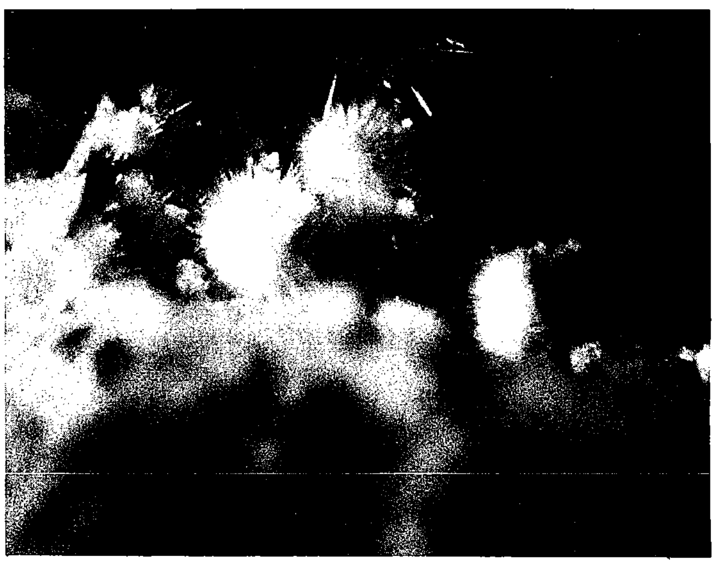


Figure 9. Delicate clumps of needle-like crystals of ettringite (calcium sulfoaluminate) occur as secondary mineralizations in portland cement concrete which has experienced chemical alteration or deterioration from sulfate reaction. (Magnification 30X)

Linear Traverse Examination

Microscopic examination of the air-void system of hardened concrete provides subtle and conclusive comparisons of concrete samples from the same job or samples of variously designed concrete. Comparisons of concrete samples with widely differing air contents are shown in figures 10 and 11. Graphic presentations of the air void parameters, namely, air content, voids per linear inch, average chord length, specific surface area, and void-spacing factor, visually represent quantitative measurements. These graphic measurements provide a "fingerprint" of a specific piece of concrete, and several pieces of concrete can be compared in a repeatable and specific test procedure that can easily be conducted if the equipment is available (ASTM Designation: C 457).

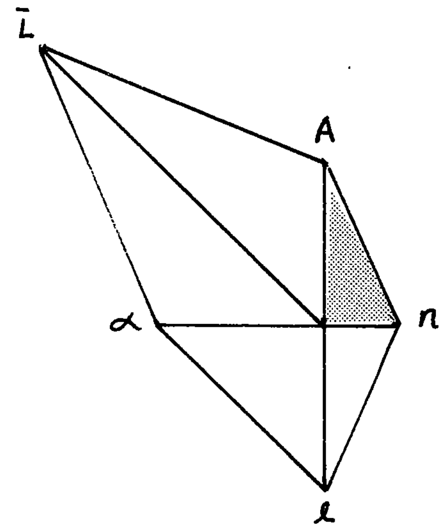
Plotting the mathematical interrelationship between the various measurable air void parameters provides a method for comparing a sample to a standard or, as previously stated, to other cores or pieces of concrete. Graphs of the air void parameters measured for the concrete samples illustrated in figures 10 and 11 are compared in figure 12.

Skid Resistance

A number of reports and investigations outlining the behavior of aggregates have revealed that petrography can explain relationships between composition, rock fabric, and performance. This is especially well demonstrated with potential skid resistance of aggregates (Patty, 1971a, 1973a, 1974b, 1974c, 1975). Petrographic evidence can relate to freeze-thaw susceptibility, soundness, abrasion, and polishing characteristics.

Figure 10. An entrained air system measuring 3.2 percent by volume. The spacing factor is 0.75 inches. (Magnification 20X)

Figure 11. An entrained air system measuring 14.5 percent by volume. The spacing factor is 0.002 inches. (Magnification 20X)



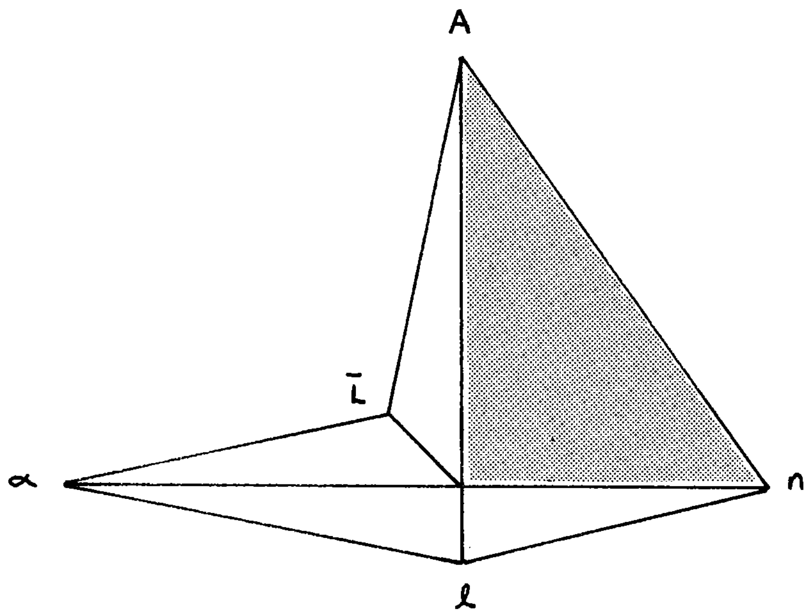


Figure 12. Air-void parameter graphs showing relative values of the internal void system as measured by linear traverse methods outlined in ASTM Designation: C 457. Five selected parameters, air content (A), voids per linear inch (n), average chord length (l), specific surface area (α), and void spacing factor ($\overline{L}$), are graphed for the two concrete samples illustrated as figures 10 and 11 respectively.

Non-Minerals

Epoxy and polymer substances represent exotic material that was used either to fill cracks in concrete or to seal against weathering and solutions by means of impregnation or by the placement of polymer sealcoats. Polymer treatments have primarily been applied to bridge decks but have other practical applications.

Detection and identification of polymers in hardened concrete by petrographic techniques are useful in evaluating bridge deck sealing or impregnation procedures. The normal method of examining concrete for the presence of an applied polymer involves coring the structure, dipping the cores into an acid bath for about one minute, rinsing, and drying. Color enhancement of the cement paste with phenolphthalein aids in locating the paste that was not affected by the polymer impregnation. The polymer "acid-proofs" the paste and the phenolphthalein will not stain the impregnated areas. The definitive examination is always conducted with the aid of a binocular microscope. A study of polished sections viewed from 200 to 400X, as illustrated in figure 13, was found to be the most effective means of identifying and determining the homogeneity of impregnated polymer systems in hardened concrete (Patty, 1978).

CONCLUSIONS

Engineering petrographers often use microscopes to examine both rocks and minerals and exotic construction materials. Although greatly expanded during the past three decades, petrographic studies of portland cement concrete have come of age and become more important in view of optional mix designs, admixtures, reactive aggregates, and specification changes. Petrography has wide application in selecting, testing, and processing concrete-making materials and

Figure 13. Photomicrograph of a polish section of polymer impregnated concrete. The large particles are quartz sand grains. The cement paste shows a darkened zone of polymer compared to the impregnated lighter colored paste. (Vertical illumination, magnification 200X)

in evaluating concrete in service, particularly in investigating causes of damage or failure. Many petrographic observations are such that they are used unilaterally to support important engineering decisions. Applied microscopy is used to relate textural properties to potential skid resistance of paving materials as well as to help explain the service behavior of bituminous mixes, metals, and polymers.

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EVALUATION OF CHANNEL STREAM BANK EROSION IN URBANIZING WATERSHEDS IN THE BLACKLAND PRAIRIE, NORTH CENTRAL TEXAS

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INTRODUCTION

Problems associated with urban development and streams can be classed into four categories: (1) flooding, (2) channel bank erosion, (3) lowering water quality through pollution/sedimentation, and (4) loss of aesthetic quality (Leopold, 1968). This paper focuses on the problems and potential solutions for channel erosion in urbanizing watersheds, Dallas, Texas.

The City of Dallas has spent $500,000 in recent years on measures to halt the loss of private structures and yard areas due to streambank erosion (City of Dallas, 1979, p. 31). Similarly, lateral migration of stream channels caused an estimated loss of 300 bridges and problems in 1,000 others in the State of Texas (F.H.A., 1978, p. 19).

Study Area

The study area lies at the apex of a triangle formed by Dallas to the north, San Antonio to the southwest, and Houston to the southeast that contains over half the population of the state (fig. 1). Conversion of rural to urban land is occurring at a rate of approximately 3,500 acres a year in the City of Dallas, a rate representative of the rapid growth in and adjacent to the metropolitan centers with the region. The study area lies within the

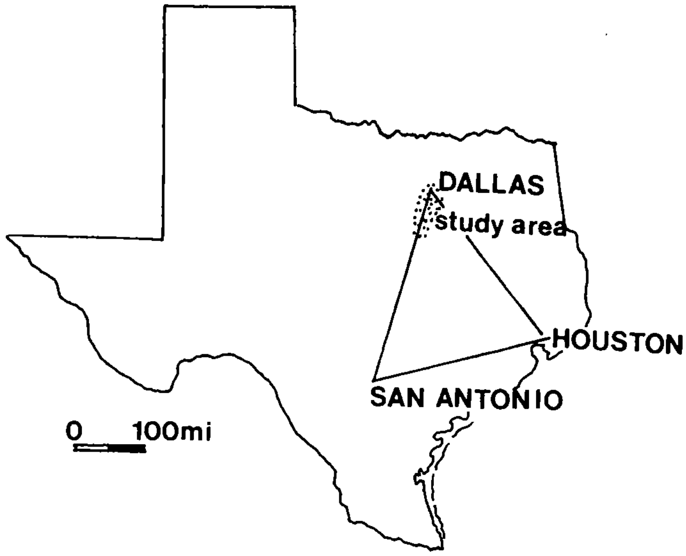


Figure 1. Location of study area in Texas.

Blackland Prairie physiographic province of Texas, a fertile band of black clay and brown silty clay soils, underlain by chalk and shale bedrock. These geologic units strike north-northeast and dip gently toward the Gulf Coast at 40 to 60 ft/mile (fig. 2).

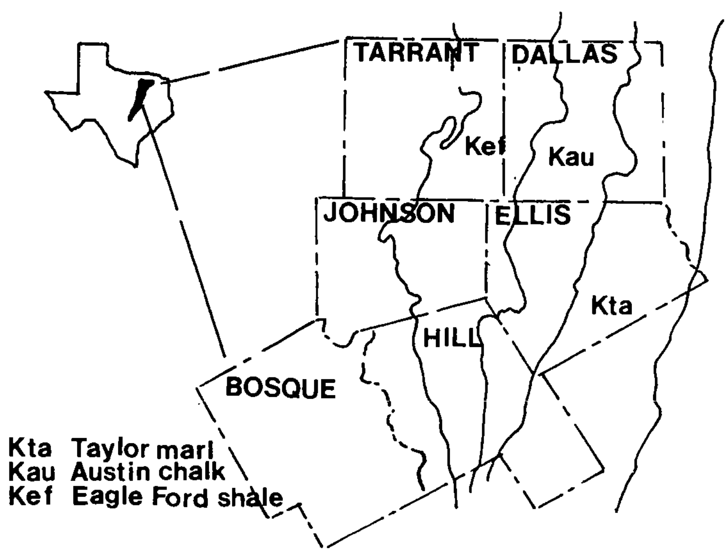


Figure 2. Geology of the Blackland Prairie, north-central Texas.

The landscape is typlified by flat to gently undulating prairie which is intermittently dissected by west to east flowing streams.

The study area lies within the extreme northern part of the humid subtropical belt that extends northward from the Gulf of Mexico. Average seasonal temperatures range from 46 to 85°F, and annual precipitation is about 36 inches. Rainfall in the area is produced from various types of storms. During the fall and winter, long duration, low intensity storms triggered by the southward moving continental polar fronts are common. The most common storm occurring from April to September is the squall line thunderstorm. Individual excessive rains causing serious flooding occur most frequently during the spring (Smith and Welborn, 1967).

Methodology

Analysis of channel erosion in the Dallas area involved four interrelated areas of study (fig. 3): (1) an overview of stream channel geomorphology and its relationship to the areal extent of channel erosion within urban and natural watersheds, (II) analysis of the magnitude and processes of channel erosion within urban and natural watersheds and related hydrology of urbanizing watersheds, (III) analysis of the geology, soils, and their engineering properties in relationship to channel erosion in the study area, and (IV) evaluation of guidelines proposed by the City of Dallas for channel erosion.

(I) Overview of stream channel geomorphology

Before introducing a geomorphic classification of streams developed for urbanizing watersheds, the genetic factors differentiating one stream from another must be cited: (1) stream discharge, (2) longitudinal slope, (3) sediment load, (4) resistance of banks

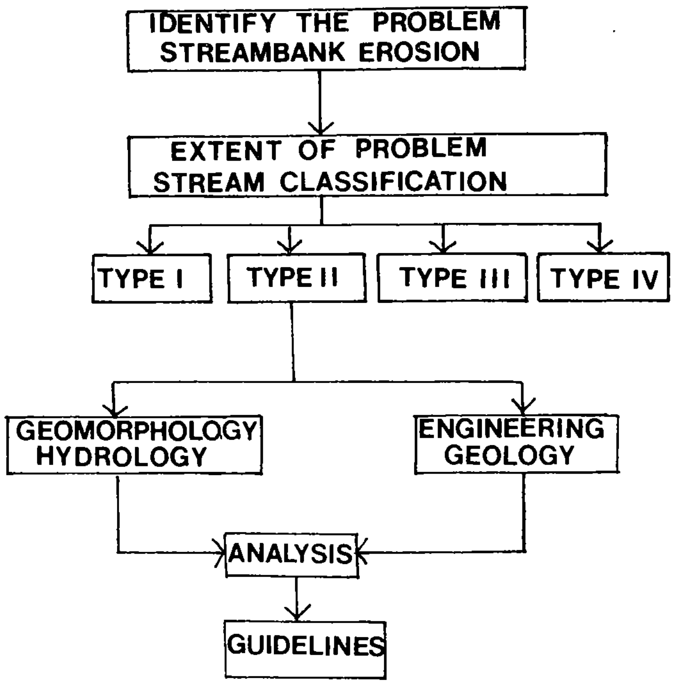


Figure 3. Methodology followed in evaluation of streambank erosion.

and beds to movement of flowing water, (5) vegetation, (6) temperature, (7) geology, and (8) works of man (Lane, 1957, p. 8). A stream system, fully developed within its drainage basin, has undergone a long period of adjustment of its geometry. The actual profile of a stream is determined by the geology and local base level. Within this profile, individual streams and their tributaries are joined together into a drainage net. All surface waters discharge from the watershed through the drainage net and the mouth of the main channel. Horton (1945) showed that streams within the drainage net could be ordered within a hierarchy from fingertip tributaries to major channels. Within this hierarchy Horton (1945) found channels of the same order to have similar lengths and drainage areas.

Given these general similarities in drainage morphology illustrated by Horton (1945) and others, it is here assumed that streams within a region of homogeneous geology and climate can be classed into four basic types based on similar readily identifiable geomorphic, geometric, and hydrologic characteristics. Because each stream type described differs over a definite range in its physical and hydrologic characteristics, it can be assumed to present similar problems to the city with subsequent urbanization. Therefore, the basic premise under which this classification is proposed is that: (1) natural stream channels provide the least expensive means for carrying away surface water, (2) that there is a hierarchy of stream channel types within every drainage basin, (3) in each stream type the magnitude of ongoing processes of erosion, deposition, and channel form are similar, and (4) once identified, these channel types allow prescription of proper safeguards and guidelines for urban development within a watershed along those parts of the stream channel.

These assumptions were tested through field reconnaissance of known problem areas, conversations with city hydrologist (N. Maier, personal communication, 1977) and work with the City Planning Department (R. Stanland and M. Krout, personal communication, 1977). Results of these discussions are presented (fig. 4, tables 1 and 2): figure 4 and table 1 describe the geomorphic characteristics of stream types in Dallas and table 2 indicates the relationship of stream types to urbanization.

Stream types were delineated on topographic maps (1" = 400 ft., five foot contour interval), of the City of Dallas. The 100-year water surface was superimposed on this map from past drainage studies. The areal extents of stream types (as described in fig. 4, table 1) were tabulated by laying a standarized grid over each floodplain area (fig. 5). Of the approximately 45 mi^2 of floodplain land within the City of Dallas, 25 percent falls within those stream

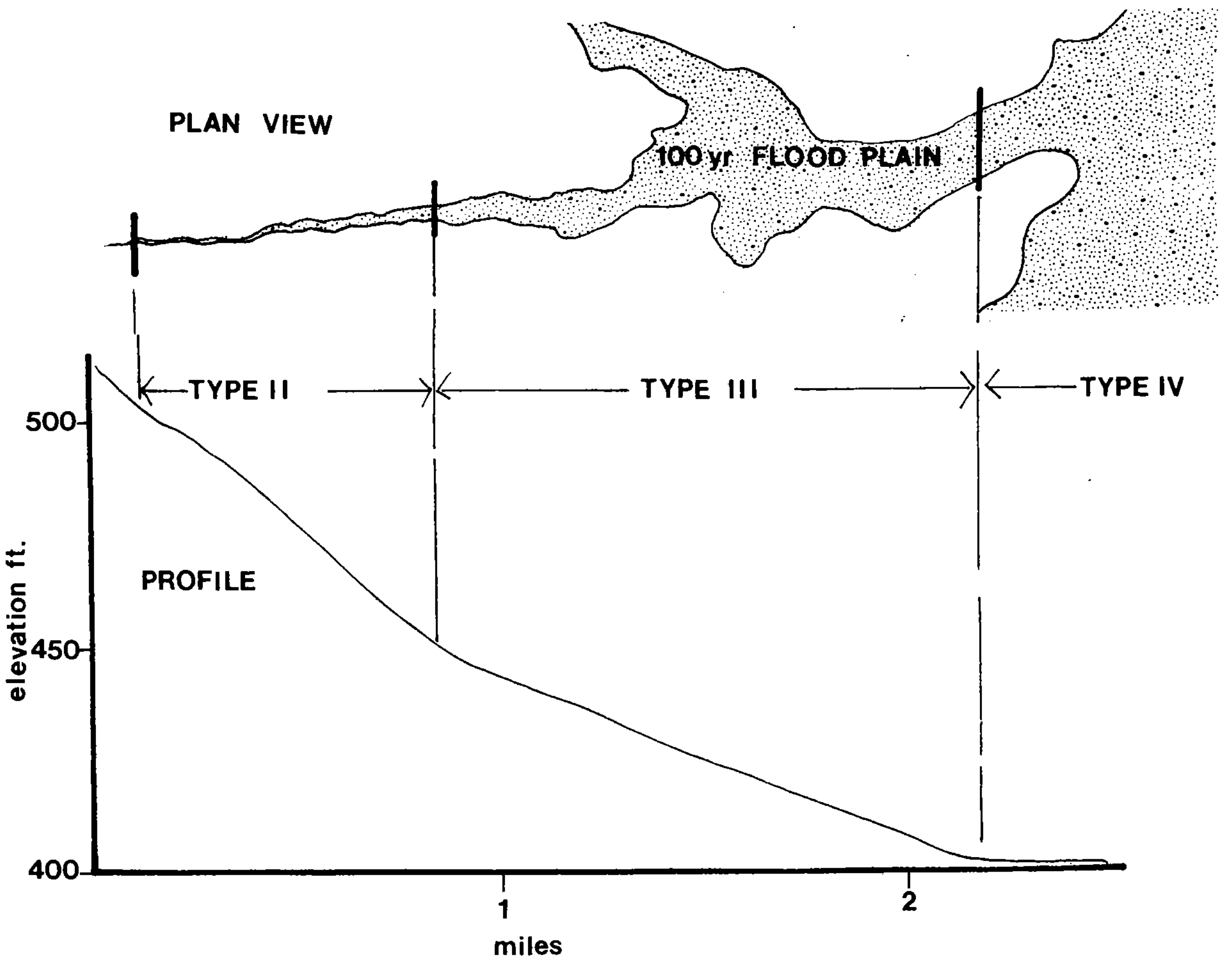


Figure 4. Plan view and stream profile indicating approximate boundaries of stream types.

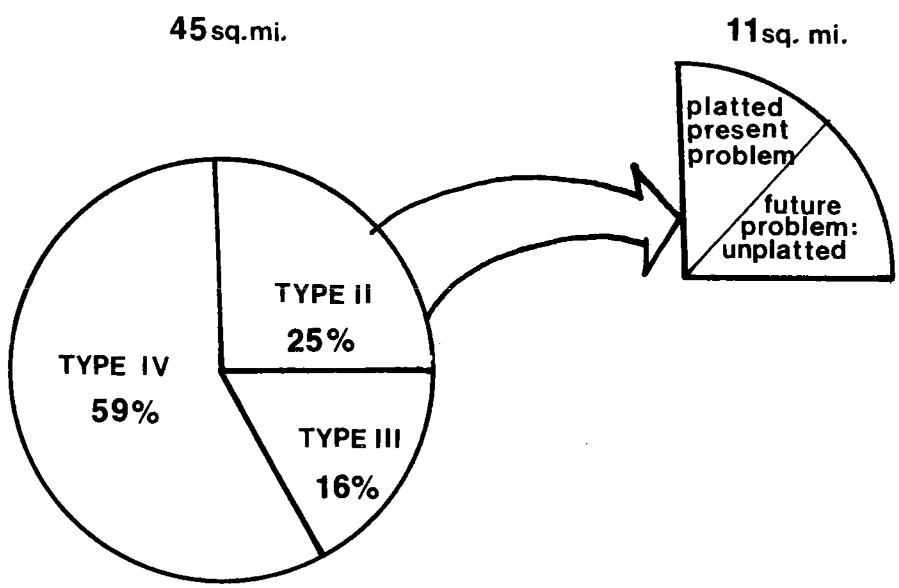


Figure 5. Percentage of floodplains within City of Dallas according to stream classification.

Table 1. Geomorphic Characteristics of Stream Types

STREAM TYPE	GEOMORPHIC CHARACTERISTICS
I	Small fingertip tributaries with contributing drainage areas of less than 130 acres (0.2 sq.mi.); steep channel gradients (13-16 + ft/mi); small trapezoidal stream channel cross-sections with channel bottoms typically cut into bedrock (chalk or shale) and channel banks into residual soils (typically clays-silty clays); 100-yr. water surface is typically within 10-20 feet of the top of the bank; 100-yr. discharges typically less than 1,000 c.f.s. Riffle sequence dominates; streams dry most of the year.
II	Small,youthful stream channels with contributing drainage areas ranging from 0.2-10 sq. mi.; steep channel gradients (10 + ft/mi); small, deep, steep-walled trapezoidal channels; valley walls beyond top of channel bank relatively steep; channel bottoms cut into bedrock (chalk or shale) or alluvium; 100-year water surface is typically contained within very close proximity to the top of the channel banks (less than 200 ft.); 100-yr. discharges range from 1,000-16,000 c.f.s.. Riffle sequences dominate the channel bottom; few pools; ratio 5 riffles to 1 pool; channel velocities 7-14 f.p.s.
III	Moderately large stream channels with small to moderately wide (200-1,000 ft) floodplains; contributing drainage areas range from around 10-150 sq. mi.; moderately steep channel gradients (2-10 ft/mi); steep walled trapezoidal channels; channel bottoms cut into bedrock or alluvium; channel banks typically alluvium with occasional bedrock cliffs on the outside of meanders; 100-yr. water surface spreads well beyond the channel banks tops onto a well developed floodplain; 100-yr. discharges range from around 16,000 to 40,000 c.f.s.; over-bank velocities range from 2-4 f.p.s.; channel velocities 5-8 f.p.s.; well developed pool-riffle sequence; pools 2 to 1 ratio to riffles.
IV	Large meandering rivers with wide (1,000-6,000 ft) floodplains; contributing drainage areas in excess of 150 sq.; low channel gradients (5 ft/mi or less; steep walled trapezoidal channels with wide channel bottoms (40 ft +); channel cut into alluvium; flood water (100 yr) spread well beyond top of channel; floodplain cleared of most trees in leveed areas; floodplain in natural state supports well developed canopy; location of economic gravel deposits; floodplain farmed in places or used for recreational activities (nature preserves, golf courses); pool sequence dominates channel bottom.

Table 2. Relation of Stream Types to Urbanization of the Watershed.

STREAM TYPE	RELATION TO URBANIZATION OF WATERSHED
I	Problems associated with cost of maintenance of open swale channel versus cost of putting stream in a pipe. Typically, in Dallas, these streams are put into pipes (up to 72 in) in order to reclaim land for building.
II	This channel area is too large to be economically put into a storm sewer for most developments; exceptions would be high density multifamily or commercial. Major problems developing adjacent to this channel type relate to channel bank instability due to erosion or slumping. This ultimately leads to installation of expensive retaining walls, lined channel sections, or purchase of structures.
III	This stream type with its wider floodplain area is usually under tremendous development pressure to fill the floodplain and raise it above the 100-year water surface. Reclaimed land is worth upwards of 20,000 dollars an acre. Major problems are associated with the hydrologic effects of such fill on the floodplain/100-yr. water surface up and downstream and the ultimate loss of this ecologically and esthetically important resource. Filling in the floodplain is now regulated by City Council Ordinance; Creek Assessment Methodologies have been developed in order to assess floodplain land for potential purchase by the Parks Department.
IV	Major problems associated with this area in terms of flood hazard have been reduced by levee construction and alteration of the floodplain. Other problems relate to regulation of mineral (sand-gravel) acquisition in the floodplain, treatment of abandoned gravel pits, and to filling in areas prior to levee construction. Water quality in this stream type is generally very low due to upstream sewage treatment plants.

types in which channel erosion has been observed to be a problem: type II streams. The remaining undeveloped (unplatted) area located along these streams types indicates that a large part of the city has the potential for future channel erosion problems.

(II) Analysis of the magnitude and processes of channel erosion

The volume of runoff from a given storm event is controlled primarily by infiltration characteristics of the soil cover complex: slope, soil-bedrock, and vegetation. The flow regimen of the stream is controlled by the characteristics of the soil cover complex and the rate at which water is transmitted across the land to the channel. Urbanization affects the flow regimen in that it both increases the percentage of impermeable surface within the watershed (Nelson, 1970; Dempster, 1973; Leopold, 1968) and changes the size, density, and characteristics of tributary channels (Leopold, 1968). Since stream channels form in response to the flow regimen, changes in this regimen cause adjustments in the stream channels: urbanized channels should be expected to adjust to greater total runoff, decreased channel lag, and related increased peak discharge. Dempster (1973) found urban watersheds in Dallas had flood peaks from 1.2 to 1.4 times those of undeveloped watersheds.

Leopold, Wolman, and Miller (1964) and Hammer (1972), among others, have suggested that stream channels tend to maintain a state of quasiequilibrium with the flow regimen of the stream such that there is a constant frequency of bankfull flows. These are estimated to occur approximately once every 1.5 years. Urban development of a watershed is known to cause changes in this streamflow regimen (Leopold, 1968; Hammer, 1972; Fox 1976; among others).

Park (1977) outlined three approaches to the identification and testing of such humanly induced changes to the stream channel: (1) monitoring the actual channel changes at monument sites through time, (2) monitoring the changes in discharge and sediment yield which accompany urbanization and using theories such as the regime theory to predict channel changes, and (3) using some form of spatial interpolation technique in which channel form properties observed under modified conditions can be compared with estimates of the form properties of the same site under natural conditions. In this study, the third method was chosen and modified from Fox (1976), Hammer (1972), and Robinson (1976). Twenty-six watersheds ranging in size from 0.23 to 46.5 mi^2 were chosen. Twenty rural watersheds and six urbanized watersheds were evaluated (fig. 6). Although more urban watersheds were investigated, only five proved to contain unaltered channels in the study area at selected cross sections.

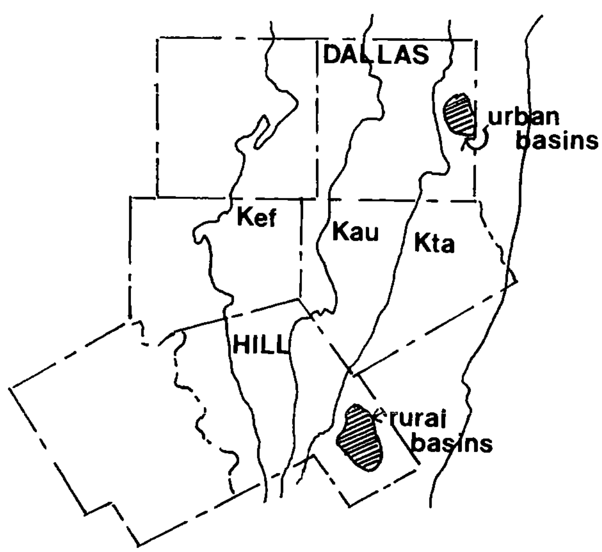


Figure 6. Location of basins utilized in analysis of channel erosion.

Although both chalk and shale bedrock crop out in the study area, areas underlain by shale bedrock were chosen for detailed evaluation based on the assumption that maximum channel enlargement would occur in these channels. While this assumption is supported by engineering tests and field evaluation; work is currently being done by Narramore (personal communication) to test this assumption.

Aside from choosing areas with similar bedrock, soils, drainage areas, slope and vegetation, sites chosen for indirect determination of channel capacity were also based on criteria indicated in figure 7.

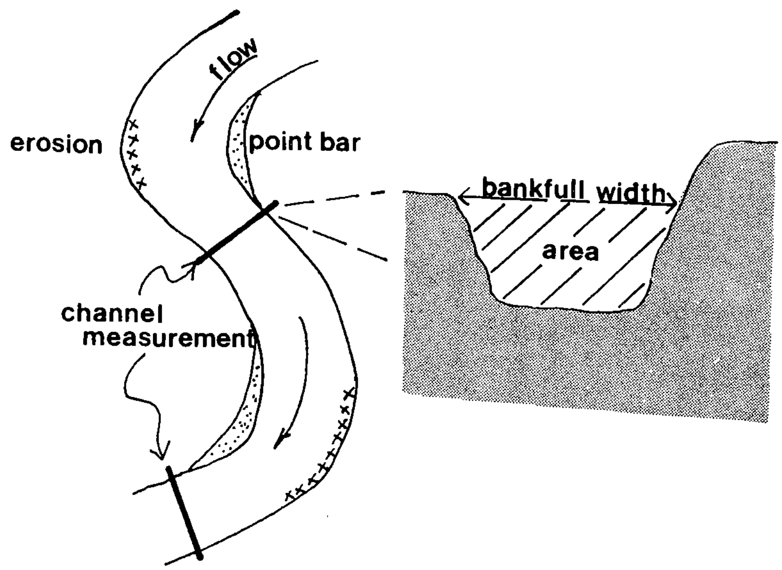


Figure 7. General criteria for measuring stream channel cross sections.

Methods used in measuring channel cross sections were defined by Leopold and Dunne (1978) and Hammer (1972) and consisted of stretching a thin nylon string across the top of the channel section. Measurements were then taken at one-foot intervals or at significant breaks in slope across the channel from the string to the channel bottom with an extendible range pole, thus establishing the channel configuration.

Because one of the objectives of the study was to correlate the cross sectional area of bankfull stage of urban versus natural watersheds, a uniform criterion had to be devised to determine bankfull elevation. The top of the lowest bank was used to define bankfull stage (after Fox, 1976). The percentage of basin urbanized was determined from analysis of recent (1978) air photographs of the study area. Since the majority of the area (80 percent) proved to be residential uses, no differences were noted in computing the percentage of different land use types within basins. Based on the U.S. Geological Survey's 7.5-minute quadrangle maps, the majority of the surveyed area was urbanized prior to 1973. This is significant because as Hammer (1972) noted there may be a lag time between urbanization of the watershed and channel enlargement of up to four years.

Table 3 lists the data corresponding to cross sections measured. The relationship of channel width, depth, and cross-sectional area for urban and natural watersheds are shown (figs. 8 and 9).

SECTION NUMBER	DRAINAGE AREA (sq.mi.)	BANKFULL AREA (sq.ft.) BA	TOP WIDTH (ft.) W	MEAN DEPTH (ft.) 1/	PERCENT URBAN
1	0.23	10	8	1.17	-
2	0.29	6.7	15	0.4	-
3	1.09	32.5	12	2.5	-
4	1.12	57.5	29	2.1	-
5	1.23	58.75	22	2.7	-
6	1.54	47	22	2.1	-
7	1.82	55	19	3.7	-
8	2.01	49.5	22	2.6	-
9	2.25	37.5	20	1.6	-
10	2.5	55.8	29	2.7	-
11	2.76	55	24	2.2	-
12	2.93	95	36	2.6	-
13	3.54	66	20	3.3	-
14	5.73	108	44	2.5	-
15	8.67	137	55	2.3	-
16	11.09	156	48	3.3	-
17	13.5	340	43	7.6	-
18	16.04	240	42	6	-
19	40.41	280	57	5	-
20	46.5	314	65	4.8	-
21	0.6	50	28	1.8	51
22	1.06	95	25	3.8	53
23	1.3	137.5	49	2.8	95
24	3.6	126.25	49	2.6	62
25	8.6	122.5	34	3.6	13
26	20.9	190	36	5.27	39

1/ Depth= BA/W

Table 3. Measurement data for streams shown in figures 8 and 9.

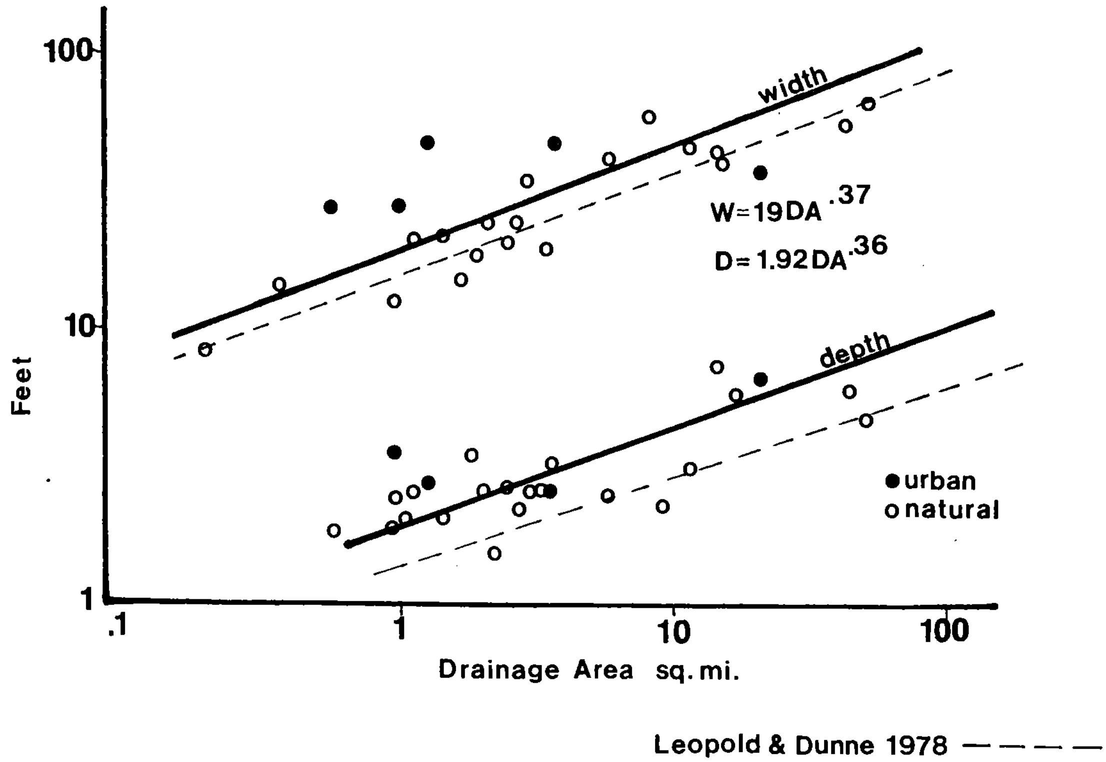


Figure 8. Channel width and depth plotted as a function of drainage area. (Dashed lines are after Leopold and Miller, 1978).

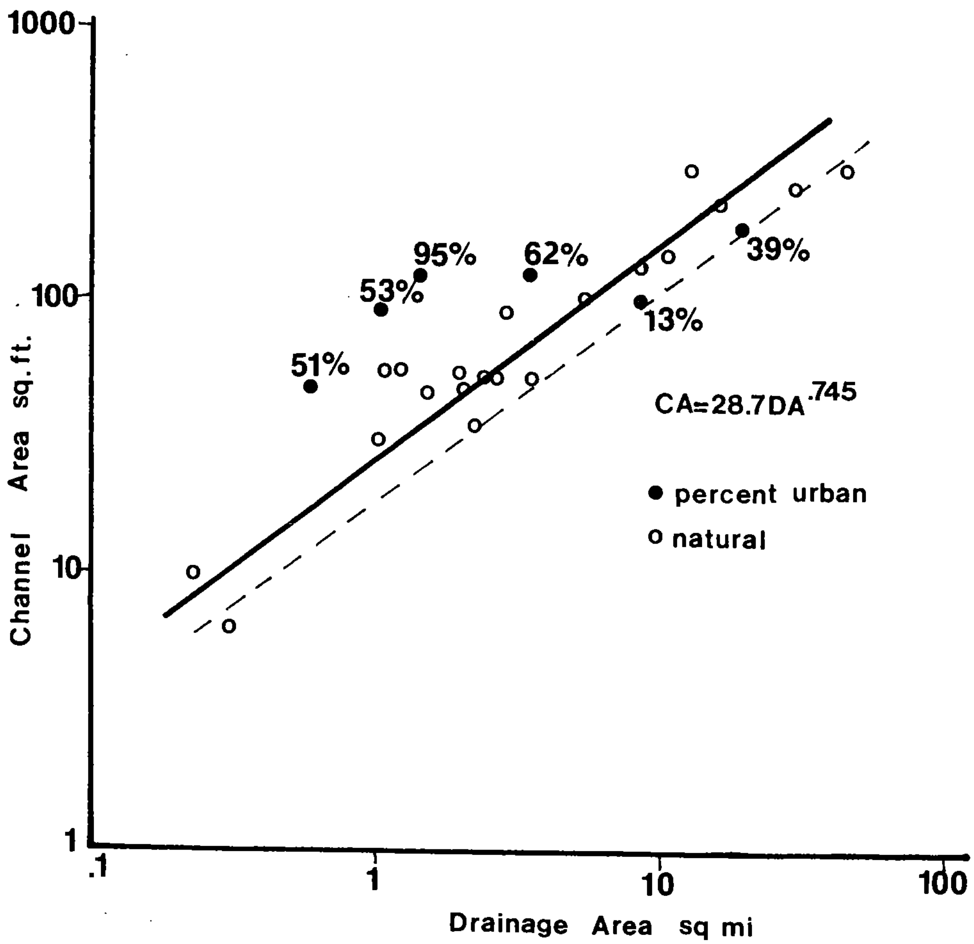


Figure 9. Channel area plotted as a function of drainage area.

Best-fit lines were drawn through the points in order to estimate the general relationship between the variables. Measurements given by Leopold and Dunne (1978) are shown for comparison. It can be noted that recorded data have a similar slope but appear to plot consistently higher than those of Leopold and Dunne (1978). This fact seems consistent with the greater point rainfall events (inches per hour) and annual precipitation norms (34 to 36 inches) of the study area in contrast to those studied by Leopold and Dunne (1978), where annual rainfall cited was 30 inches.

Although results should be viewed with caution owing to the small available number of measured urban channels, the following inferences are probably justified (figs. 8 and 9):

(1) Urban channels tend to be wider and deeper than natural channels with ratios of urban to natural channel width (1.3 to 2.3) being slightly greater than depth ratios (1 to 2.0).

(2) Urban channel cross-sectional areas are from 1.7 to 3.7 times greater than predicted bankfull channel areas for similar size watersheds, in the rural state.

(3) The greater the precent of urbanization recorded within the watershed, the greater the increase in channel cross section area for drainage areas up to approximately 5 square miles.

(4) Larger watersheds showed no comparative increase in channel dimensions with measured degrees of urbanization (13 percent and 39 percent).

The greater observed increase in channel erosion in small channels (less than 5 sq. miles drainage area), is attributed to: (1) greater alteration of channel sections in these areas, (such as channelization and straightening of channel reaches), owing to the fact that it is less costly; (2) increased storm sewer discharge density and discharge per unit length of channel, thus augmenting peak flow rates; (3) decreased woody vegetation lining and protecting channel banks (due to less moisture availability); (4) greater channel slope, and (5) perhaps more rapid reaction to the altered flow regime than the larger channels.

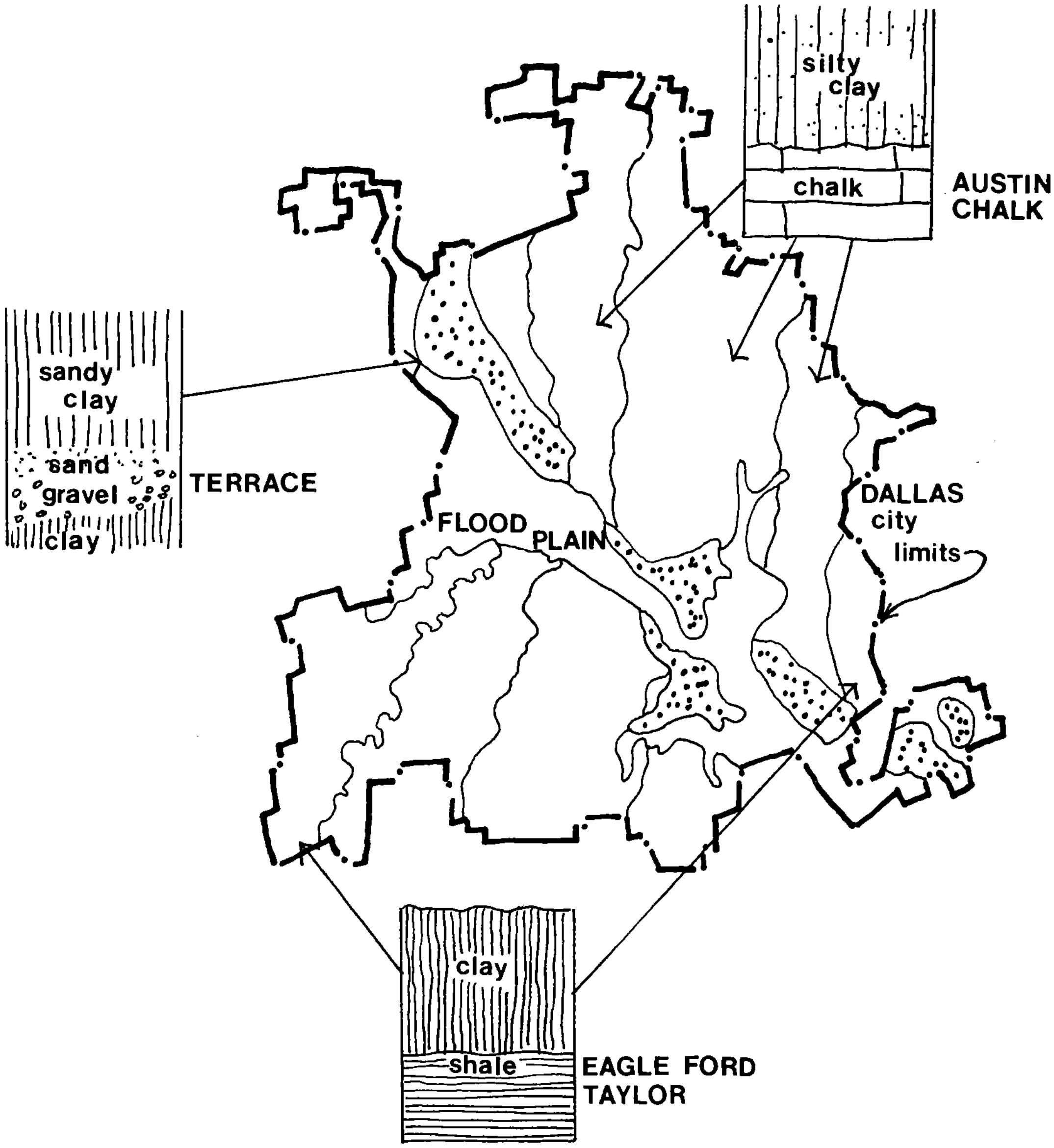
(III) Analysis of engineering properties of bank materials of Type II streams

The City of Dallas was divided into zones based on similar engineering properties of mapped soil-bedrock units (Clark, 1976; Allen, 1975). Detailed soil series information (Soil Conservation Service, 1979) at a mapped scale of 1:20,000 was superimposed photographically on mapped geologic units modified from Clark (1976) at a scale of 1:24,000. Correlations between soil types and bedrock within Type II creek areas were delineated. The delineated soil-bedrock units fell into three basic groups: silty clay alluvium over chalk bedrock, sandy clay alluvium over terrace deposits, and clay alluvium over shale bedrock.

Representative samples were taken from these mapped soil-bedrock units at stream channels for more quantitative engineering evaluations of Type II channel characteristics. Standard geotechnical tests were conducted in the laboratory by Font (personal communication, 1980) to determine the engineering properties of the channel bank soils in question. Tests were performed to determine the Atterberg limits and indices, the potential volume change, and the shear strength of these soils.

Values of Atterberg limits and indices and potential volume change are summarized in table 4. These values are indicative of highly plastic soils with high shrink-swell properties. The exception to this are the sandy terrace soils.

Values of undrained shear strength are summarized in table 5.



Soil Type	Liquid Limit (Average Range in %)	Plasticity Index (Average Range in %)	Potential Volume Change (Expected Shrink-Swell)
Eagle Ford Clay Soils	42 - 61	26 - 35	Critical to Very Critical Swell index pressures ranging from 6000-15,000 PSF+
Austin Chalk Silty Clays	29-55	22 - 33	Critical to Very Critical Swell index Pressures ranging from 4500-13,000 PSF+
Alluvial Terrace Sandy Soils	28 - 35	6 - 12	Marginal to Non-Critical Swell index pressures generally less than 3500 PSF

Table 4. Index properties of alluvial material and general location within the City of Dallas.

Soil Type	Cohesive Strength "C_{cu}" in TSF	Angle of Internal Friction ϕcu ϕcu (degree)	Coefficient of Friction ϕcu Friction (tan ϕ)
Eagle Ford Clay Soils	Neglegible or 0	14°	0.25
Austin Silty Clays	Neglegible or 0	19.29°	0.35
Sandy Terrace Soils	Neglegible or 0	21.5°	0.39

Table 5. Values of cohesive strength and coefficient of friction for alluvial soils and based on CU direct shear tests simulating embankment heights up to 20 ft.

(IV) Evaluation of proposed guidelines - City of Dallas

Two sets of guidelines are proposed by the City to protect future structures from damage due to channel erosion: guidelines for lands where the channel floodplain is dedicated to the public and guidelines for where the creek floodplain remains privately owned (fig. 10).

a. Public Dedication Guidelines:

Road R.O.W. line along creekbank (where 100-year floodplain is contained within the channel banks) of greater of: (1) 10 feet beyond the top of the bank (4:1 tangent slope) and (2) point of intersection with bank of 3:1 slope drawn from the creek bottom or rock line (4:1 in clay or shale). Rear lot line along creekbank (where the 100-year floodplain is contained in the channel banks) the greater of: (1) 20 feet beyond the top of the bank (4:1 tangent slope) and (2) 10 feet beyond the point of intersection with the bank of 3:1 slope drawn from the creek bottom on rock line (4:1 in clay or shale).

b. Where the creek is kept in the Private Ownership; not dedicated and the 100-year channel banks the greater of: (1) 10 feet beyond the top of the bank (4:1 tangent slope) and (2) point of intersection with bank of 3:1 drawn from the creek bottom or rock line (4:1 in clay or shale).

In order to assess the guidelines properly, two interrelated issues are here discussed: (1) evaluation of channel erosion potential, and (2) evaluation of channel slope stability.

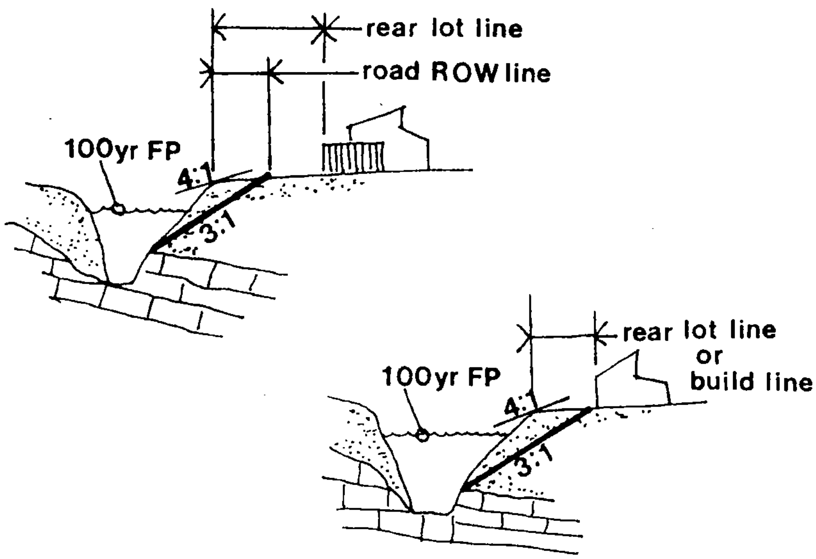


Figure 10. Proposed channel set-backs for City of Dallas, Texas. Taken from City of Dallas (1979).

In regard to the former, data indicate that maximum increases in urban (95 percent urbanized) to natural channel width approaches a ratio of 2.3. Therefore, to evaluate the proposed channel setback criterion, the width is assumed to increase up to 2.3 times that of the natural channel width. For channels from 10 to 60 ft wide (the approximate range of type II channels), potential channel bank erosion could range from 6.5 to 39 ft as indicated by the equation

$$\frac{Wn}{2} \times 2.3 - \frac{Wn}{2} = \text{potential bank erosion in feet,}$$

where

Wn = width of natural channel

Given a maximum increase of approximately 39 ft, the guidelines appear to require adequate setbacks considering the fact that structures must be set back 10 ft beyond the intersection of the 4 to 1 setback line in shale and 3 to 1 in chalk channel material.

In regard to the subject of channel slope stability, according to the Soil Conservation Service (1977), the design of channel banks from a standpoint of strength will probably depend largely upon local experience and past performance. The objective of a stability analysis is to determine the factor of safety for the most critical combination of stress and boundary conditions anticipated (S.C.S., 1977).

For this analysis, Taylor's charts (fig. 11) were used to estimate stable slope configurations based on completed shear strength values (table 5), and discussions with Ray Mason (1979, personal communication). The Soil Conservation Service (1977, p. 6-63) states "although Taylor's charts of stability numbers do not consider seepage, these charts may be used for rough determinations and preliminary solutions in homogeneous soils provided a conservative factor of safety is used."

Results indicate that for the Eagle Ford clay soils, the slope ratio should not exceed 4 to 1 (4 horizontal to 1 vertical). For the Austin Chalk silty clays, this ratio should not exceed 3.5 to 1. For the sandy terrace soils the ratio should not surpass 3 to 1. These values are consistant with setbacks proposed in the City of Dallas guidelines.

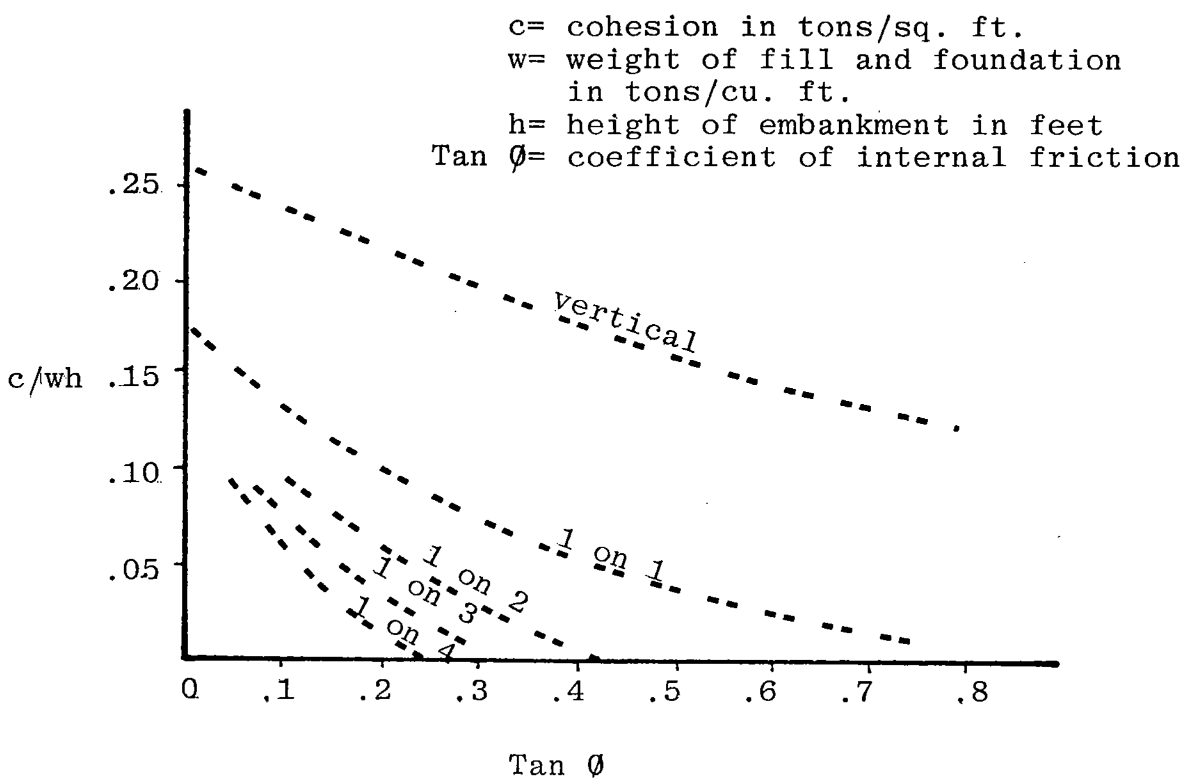


Figure 11. Slope stability chart for uniform foundation and embankment not subject to seepage or uplift forces. From Mason-Johnston (1979).

SUMMARY

Hammer's (1972) methodology provides for a simple estimate of channel enlargement which can be utilized with an appropriate factor of safety in preliminary design of structures adjacent to small (Type II) channels in urbanizing watersheds.

Results and conclusions with respect to City of Dallas guidelines and this investigation indicate that:

based on analysis of the engineering properties of the channel bank material and analysis of potential for increased channel erosion with increasing percentages of impermeable surfaces within a watershed, the concept of guidelines as presented by the City is an excellent, easily verifiable method for establishing setbacks that have a factor of safety consistant with the conditions found within the City of Dallas, Texas. It is also assumed that the concept of the guidelines as presented could be applied to other areas if proper adjustments are made for hydrologic and engineering properties of the channels as indicated in this analysis.

ACKNOWLEDGMENTS

I wish to recognize Nathan Maier of the Department of Public Works and Marvin Krout and Ray Stanland of the Department of Planning, City of Dallas, for their input and ideas underlying many of the concepts given in this report. I am also indebted to O. T. Hayward of Baylor University, Waco, Texas and Robert Font of Conoco Oil, Midland, Texas for engineering analysis of channel material and manuscript review. Data for stream channel cross sections were compiled in part by students (Urban Geology Class, 1980), Baylor University.

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THE TYPE AREA CONCEPT

A Practical Method of Integrating Natural Resources with the Planning, Development, Maintenance, and Landscaping of Transportation Systems

by

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INTRODUCTION

Definition

A type area is a geographical area with uniform characteristics. It has uniform geology, soil associations, vegetational associations, topography, and climate. Although descriptions of type areas emphasize uniform natural characteristics, their societal characteristics are usually uniform as well. A type area may be characterized by urban or agricultural use and sometimes affects the size and type of community or agriculture. Where plans for urbanization have disregarded the inherent uniformity in an area, conventional construction has resulted in uniform failures. However, because type areas are based on natural resource characteristics, once they are mapped and categorized a permanent data base exists which can be used as a guide for all subsequent planning and construction. Exceptions commonly occur but are usually insignificant; where significant exceptions occur, they can be detected and then described within the data base.

Purpose

The type area concept is not a panacea for all planning and construction woes; instead it offers a manageable approach to solving many problems. It can provide increased efficiency due to its simple yet realistic approach. Applications of the type area concept demonstrate its usefulness and its adaptability to transportation systems.

Previous Works

Most of the foundational work for this paper came from research at Baylor University. The Urban Geology of Greater Waco, a series of theses published by the Baylor Geology Department, laid the groundwork. Burkett (1965) mapped the geology and presented descriptions and physical characteristics for formations of the Waco area. Elder (1965) provided information on soils. Spencer (1966) described the surface waters of the Waco area and Font and Williamson (1970) discussed the impact of engineering geology on construction.

In 1974 and 1975 Ellwood Baldwin and Peter Allen described the geology between Dallas and Belton, Texas, along Interstate Highway 35. The work of Allen and Baldwin provided many of the basic ideas in the type area concept.

Yelderman (1974) studied the U.S. Highway 84 growth corridor between Waco and McGregor using the type area concept. In 1976 he refined the concept by applying it to McLennan County and the Extraterritorial Jurisdiction of Waco, Texas. Finally, in 1978 the Ark-Tex Council of Governments published a natural resource management plan that used the type area concept.

AN EXAMPLE OF THE CONCEPT

The Austin Prairie type area in McLennan County (fig. 1) has uniform geology composed of alternating chalk and marl beds found in the Austin Chalk Formation. Austin and Houston black soils form a uniform association that blankets the Austin Chalk Formation. The predominant vegetation is "open land," and although trees are found along fencelines and there is some variety between farming and ranching, the open land feature is dominant. The topography is flat to gently rolling. The Austin Prairie type area is therefore characterized by flat to gently rolling topography, open land vegetation, the Austin-Houston black soil association, and the Austin Chalk geologic formation.

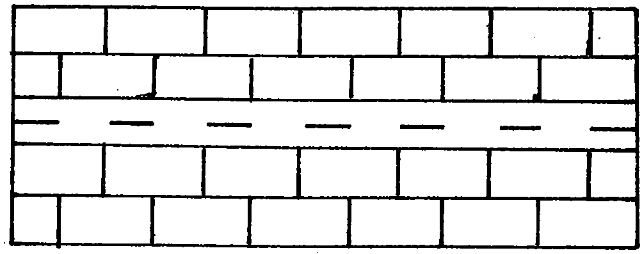
Any human activities planned on the Austin Prairie must interact with the characteristics of the area. For the activities to be effective, one must plan, develop, maintain, and landscape with these features in mind.

THE TYPE AREA FORMAT

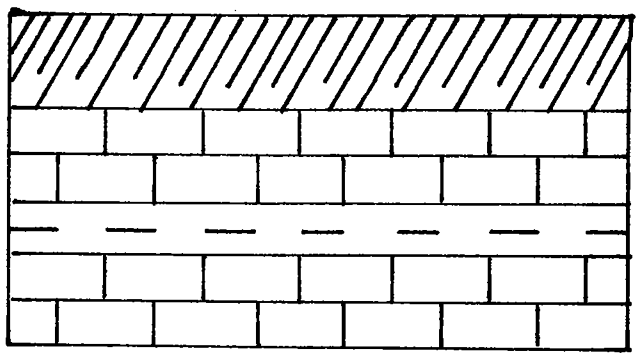
The presentation of the type area data base is critical. The basin format is that of maps and charts. Using a topographic base map at the scale of one inch equals one mile, the geology, soils, and vegetation are displayed. Information on the respective characteristics is presented on charts. Climatic information is also displayed in chart form for easy reference.

Once this information is gathered, one can fashion type areas by using overlays or by drawing the boundaries at the location closest to the change in uniform characteristics. A chart relating the features of each type area to different human activities complements the type area map.

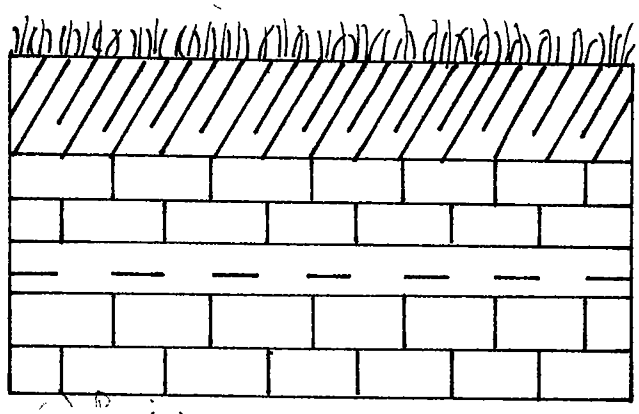
Finally, guidelines for each type area are developed so that the most common human interactions can be conducted efficiently. When all the data are placed



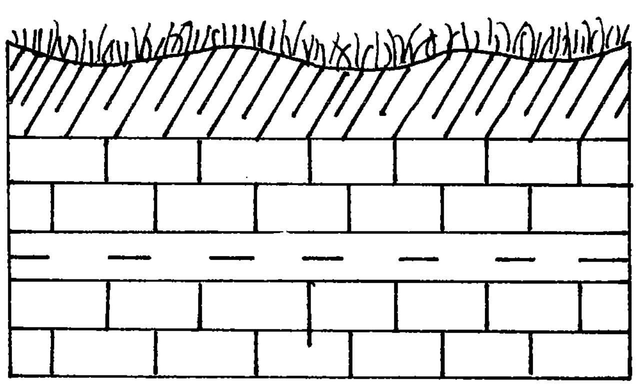
(a) Lithology, chalk and marl.



(b) Uniform soil.



(c) Prairie vegetation.



(d) Rolling topography.

Figure 1. Characteristics of the Austin Chalk Formation.

together in an atlas, the user has the uniform type area map and chart for general assistance, the specific data bases for geology, soils, topography, vegetation, and climate, and a set of implementation guidelines (fig. 2).

APPLICATIONS

Location

The type area concept is useful in siting. Before choosing the location of a road or highway, planners may use the type area map and chart to determine the best area for highway construction. Often the type area system can be used to avoid troublesome areas.

Construction

Once the location has been decided, the type area system provides data needed to make decisions regarding type of construction. The topography, shrink-well potential, slope stability, and excavation difficulty can be foreseen and an economically efficient transportation system can be developed.

Resources

Roads and highways are essential to economical resource extraction. The resource has to reach the market within time and economic constraints. If the resources in a particular area are known or anticipated, a transportation system can be constructed to serve the resource industry. Highways can be constructed to support the vehicles used for resource transportation. Without foreknowledge of these resources, highways may be located on resource deposits or constructed in such a way that the resource transportation deteriorates the roadway prematurely.

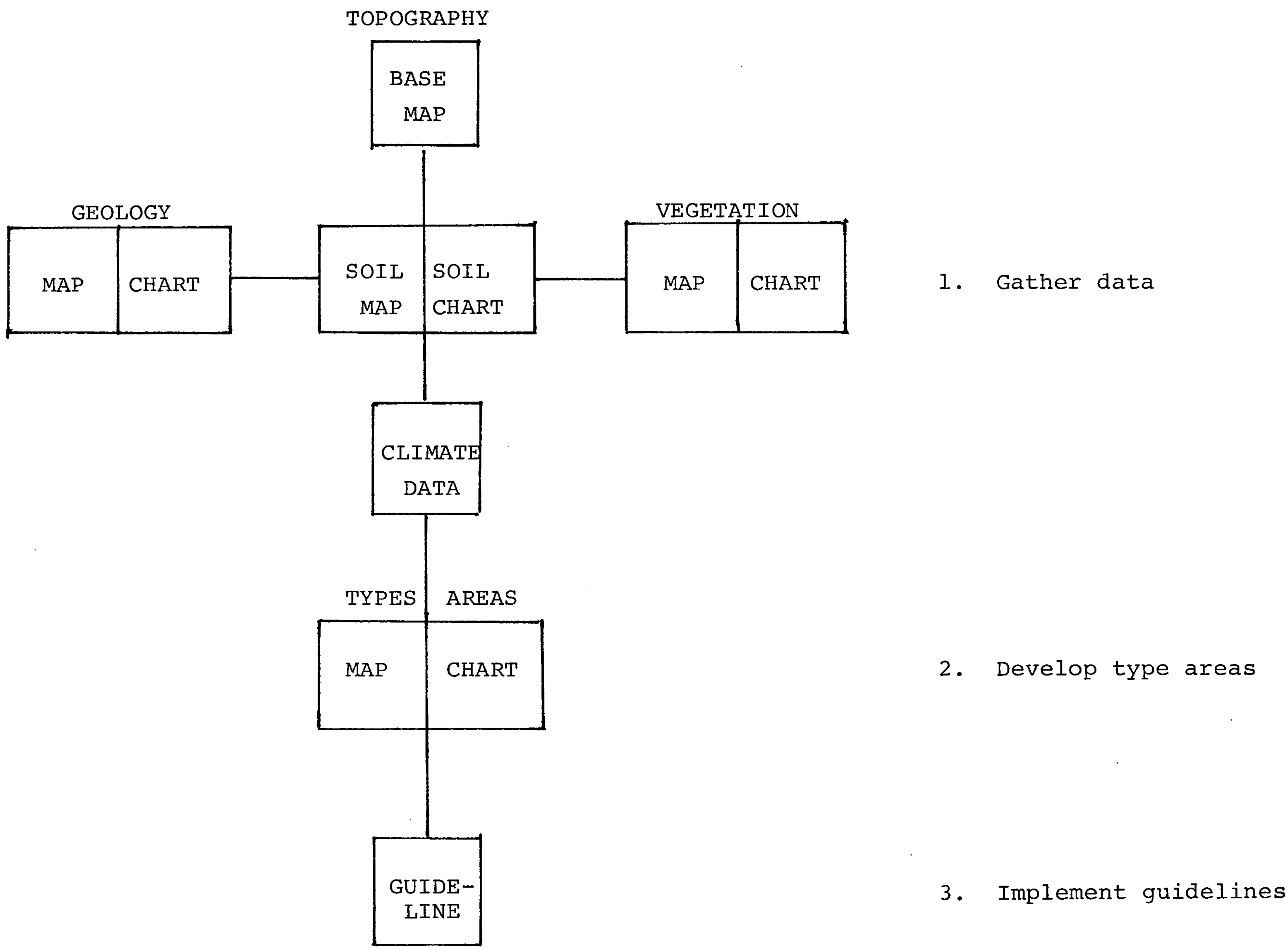


Figure 2. Components of the type area concept.

Maintenance

Economics is the overriding factor in most engineering decisions. However, often only the cost of initial construction is considered in depth. Maintenance costs are actually much more significant, especially in an inflationary era. Additional costs expended during initial construction may pay for themselves several times over in decreased maintenance costs. The increased cost of locational changes may pay off in decreased maintenance costs.

A comparative example from McLennan County shows how costs vary in contrasting substrate types. A large part of the urban setting in McLennan County includes two predominant type areas: the Chalk Prairie and the Shale Prairie. Although topography, vegetation, and soils are almost identical, the geologic formations are radically different. The chalk is difficult to excavate but has low shrink-swell properties, whereas the shale is easy to excavate but has high shrink-swell properties. Installation and maintenance costs were compared for one year (table 1).

Table 1. Construction costs, Chalk Prairie and Shale Prairie, McLennan County, Texas.

	Costs*	
	Installation	Maintenance
Chalk Prairie	$3 per foot	$1 per foot
Shale Prairie	$1 per foot	$3 per foot

*Estimated in 1975 dollars.

The comparison seems to indicate that there is no cost difference between chalk and shale areas. However, when inflation is considered, the chalk is shown to be more economical since installation is a one-time expenditure and the maintenance cost is an annual item.

Landscaping

The cost of landscaping highways is analogous to maintenance costs. Initial plantings are not as costly as long-term maintenance of the roadways, especially since increased fuel prices have caused many maintenance crews to mow less often and to mow only the areas immediately adjacent to the roadway. The best vegetation to use for efficient landscaping is that best suited to the particular soil, topography, and climate of the area: native vegetation usually meets this criterion. Native vegetation needs less maintenance and when combined with less mowing also provides habitat for native wildlife. Native grasses as well as shrubs and trees offer a buffer zone between farms and pastures, which results in decreased noise levels, decreased emission effects, and increased edge effects for wildlife including game species. In addition, the roadway becomes scenic and educational for motorists. A map of the type areas designates the different areas of uniform soil, topography, and climate. This concept is now being adopted in Illinois (National Audubon Society, 1980).

Education

The roadsides of the United States are unexceeded for accessibility and are widely used for field trips. When hills are excavated for highways, the exposed rock and soil provide a unique opportunity for education. Excavations that take advantage of the inherent slope stability of the material and do not cover exposures unless it is absolutely necessary maximize the highway's educational value and minimize maintenance. If native vegetation and wildlife are enhanced in their natural locales, the educational benefits from our highways are multiplied both in diversity and in extent.

Implementation

Once the type area map and chart have been completed, they form a data base useful for aiding many types of decisions. However, implementation needs still exist. One of the most efficient strategies is to use guidelines specific to different activities for each type area. Since there will always be exceptions to the uniformity, hard and fast rules will not always be successful. Guidelines offer the needed guidance with the essential flexibility.

CONCLUSIONS

The gently dipping beds of sedimentary rock and the covering of residual soils cited in the example are common to Central Texas. This is not the only setting where the type area concept is practical; stratigraphically and structurally complex areas can also be adapted to type areas (Ark-Tex Council of Governments, 1978).

The use of the type area concept is still in its infancy. Although similar to many overlay and integrated mapping systems with rating and suitability classifications, the type area system differs in its ability to be used successfully within economic and political frameworks. This attribute is a result of the experience obtained by its founders, many of whom worked within governmental departments or agencies that were responsible for implementation.

The maximum benefit can be obtained when all the features of the system are used. The educational benefits of our roadways cannot be maximized if all the type areas are planted with the same ground cover and the economic benefits cannot be maximized if the data base is used for construction decisions but not maintenance decisions.

The roadway systems of the United States offer numerous opportunities for developing creative ideas in wise resource use. The type area concept can be an important step in the efficient use of our resources.

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DRUMLINS AS POTENTIAL SOURCES OF SAND AND GRAVEL IN GLACIATED REGIONS[1]

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INTRODUCTION

Drumlins are elongate hills of glacial drift that usually exhibit some symmetry and are consistently aligned with regional ice flow directions. First described in Ireland in the mid-1860's (Close, 1866), groups of drumlins are present in glaciated regions throughout Canada and the northern United States. To optimize highway construction through drumlinized terrain, it is useful for highway engineers to understand the potential complexity of the landscape and the numerous possible sources of coarse aggregate in these areas.

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CHARACTERISTICS OF DRUMLINS

Although widespread, drumlins are exceptional glacial landforms that probably require special ice or geologic conditions in order to form (Muller, 1974). Where present, drumlins are typically concentrated in broad bands several tens of kilometers wide up-ice from major end moraines left by continental glaciers. The description of the "ideal" drumlin as an inverted spoon with the blunt end pointing up-ice (Flint, 1971) holds true for many drumlins although some drumlins deviate widely from this particular shape. Along the axis of ice flow, drumlin shapes can progressively change to outlines either more or less elongate. Lundquist (1969) recognized a progressive change from streamlined drumlin forms to Rögen moraine lying perpendicular to ice flow direction. Smith (1949), Lemke (1958), and Gravenor and Meneley (1958) described the transitions between very elongate grooves and fluting in drift or bedrock and the drumlins present down-ice. Although such significant changes in form are not present in every drumlin field, a systematic variation of shapes commonly exists, particularly near the edges of the drumlin field (Muller, 1974).

Several recent studies in Finland (Glückert, 1971) and Wisconsin (Simpkins, 1979; Whittecar and Mickelson, 1979) suggest that drumlins are commonly clustered atop distinct uplands comprised of drumlinoid features (fig. 1). Bordered by distinct scarps, the drumlinized uplands in Wisconsin are often partly buried by kame terraces and broad plains of outwash terraces, glacial lake sediment, and poorly drained, peaty wetlands deposited after drumlin formation. Other glacial deposits draped across the drumlinized uplands by waning ice masses include end moraines, kames, and eskers.

The contents of drumlins vary dramatically. In New England and North Dakota, bedrock cored drumlins have been reported that have only a thin veneer of till (Flint, 1971; Steven Moran, personal communication, 1976). Embleton and

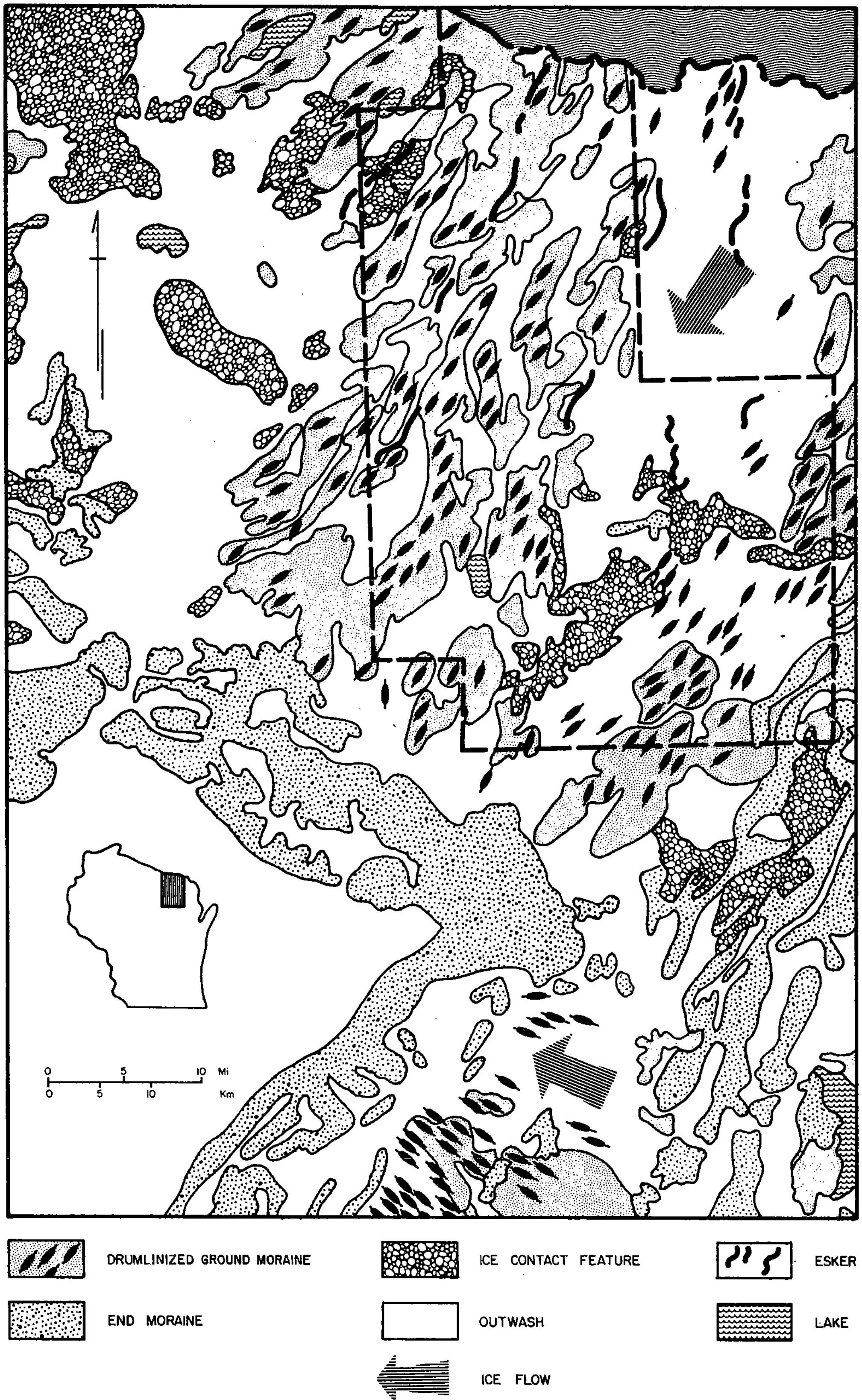

Figure 1. Map showing the glacial landforms in Forest County, Wisconsin, and surrounding area covered by the Langlade and Green Bay Lobes.

King (1975) reported a drumlin described by I. von Schaefr in Upper Bavaria that contained till, gravels, lake clays, peat, and fine sands. Between these two extremes is a sequence of drumlin compositions including completely homogeneous till, stratified sand and gravel, and bedrock (Flint, 1971). Most drumlins seem to be covered by till regardless of their internal composition (Flint, 1971). In many cases, the till blanket truncates underlying structures within the drumlin (Upham, 1894; Alden, 1905; pp. 31-32; Kupsch, 1955, p. 311; Lemki, 1958, p. 275; Garnes, 1976, p. 11; and Whittecar and Mickelson, 1977).

THEORIES OF DRUMLIN FORMATION

Because of the diversity of drumlin shapes and compositions, a single model of drumlin formations has not been universally accepted. Some early workers argued that till was deposited at the base of glaciers in streamlined shapes accreted about either bedrock or drift cores. Others maintained that partial erosion of the surface overridden by glacier ice produced drumlins. In recent years, many studies have concluded that both deposition and erosion were somehow involved in drumlin formation (Muller, 1974).

Whittecar and Mickelson (1979), following the lead of Gravenor (1953), proposed that most drumlins can be explained as erosional features by recognizing the presence of "advance" and "retreat" tills. In well-exposed drumlins in eastern Wisconsin, the surface till layer lies parallel to underlying outwash in some places yet markedly truncates the gravel bedding elsewhere within the same drumlin (fig. 2). Furthermore, the stony, sandy till is usually thickest beneath the crest of the drumlins. Clearly the pre-drumlin drift cores of these hills were eroded into their drumlin shape by the overriding ice. The structurally dissimilar tills are explained as deposits left by the drumlin-forming ice either before or after the erosion. According to Whittecar and Mickelson (1977), till is probably deposited under thin ice near the glacier margin. The conformable till represents glacial deposition over outwash during initial advance of the ice margin. Later, under increasingly thick ice, deposition ceased and partial erosion of the outwash and "advance" till began to form drumlins. Ice margin retreat once again resulted in thin ice and subsequent till deposition over the drumlin shapes. Both simple stratification and more complex deformation features formed within the drumlin under thick ice were thereby truncated by the initial drumlin form and later "retreat" till.

In theory, the "advance" and "retreat" tills should have the same general composition unless the source of glacially eroded debris changes during glaciation. Unless the "advance" till is thin relative to drumlin height and overlies deformable or stratified material, the two till phases may be difficult to identify. Therefore, an all-till drumlin, once interpreted as an accreted knob, may actually be a sculpted mass of thick "advance" till.

The hypotheses of Whittecar and Mickelson (1979) explain drumlin features in modes substantially different from other theories of drumlin formation that emphasize till deposition. Dilatancy theory forms the basis for a model of drumlin formation proposed by Smalley and Unwin (1968). Concentrating upon till drumlins, this model depends upon the property of granular sediment either to resist or to mobilize under shear deformation. Resistant sediment may provide the core for accretion of drumlin till, according to Smalley and Unwin (1968). A similar theory developed by Menzies (1979) focuses upon the role of pore-water pressures in sediment deformation and resistance.

These three theories are not mutually exclusive, however. All recognize the potential for the thermal regime of the glacier, pore-water pressures, dilatancy differences, and local stratigraphy and sediment properties to influence drumlin formation. We may eventually conclude that drumlins are examples of geomorphic

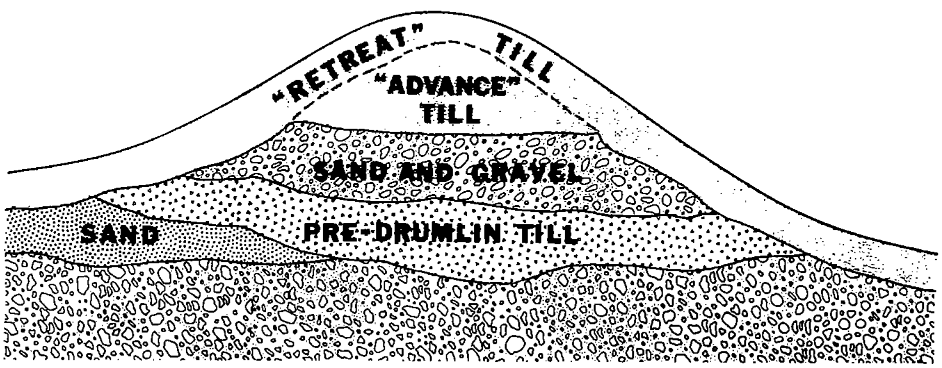

Figure 2. Diagrammatic cross-section of stratigraphic relationships found in most drumlins, Waukesha County, Wisconsin. Section drawn along short axis of drumlin. All stratigraphic symbols are diagrammatic and are only meant to represent till and stratified drift.

convergence; that is, creation of similar forms by dissimilar processes (White, 1944). On the other hand, it seems equally likely that one complex set of processes involving both deposition of "advance" and "retreat" tills and the erosion of drumlin shapes (Whittecar and Mickelson, 1979) may constitute the only necessary drumlin-forming model.

DRUMLINS AS A POTENTIAL SOURCE OF SAND AND GRAVEL

In order to maximize resource potential, it is necessary that highway geologists and engineers understand these various models of drumlin formation when analyzing geotechnical data in drumlinized areas. Because the dilatancy-based theories of Smalley and Unwin (1968) are well-established, many geologists assume that a till-covered drumlin is probably till-cored. However, it has been our experience that many drumlins in eastern and northern Wisconsin contain a great deal of high quality sand and gravel. Simpkins (1979) reported that large drumlins formed by a glacial lobe carrying non-calcareous glacial till and gravel were actually comprised of thick beds of coarse dolomitic outwash with only a thin bed of till on top (fig. 3). As mentioned before, Whittecar (1976) studied a drumlin field in eastern Wisconsin being extensively mined for coarse outwash. Field work in Alaska (Goldthwait, 1974) located recently deglaciated drumlins formed from outwash gravels. In all of these areas, the drumlins were buried under a uniform blanket of till, presumably deposited during a glacial retreat.

In essence, we believe that drumlins are carved out of pre-existing materials on the land surface. Since proglacial outwash (sand and gravel) and other types of stratified drift are common in glacial landscapes, gravel-cored drumlins shoud make up a significant percentage of the total drumlin population. Because these hills will probably be covered by a blanket of "retreat" till which masks the coarse materials, on-site subsurface investigations would be required to substantiate any potential gravel deposits.

Figure 3. Photograph (this page) and sketch (next page) of till and outwash within a drumlin near Crandon, Wisconsin. The non-calcareous till was deposited by Langlade Lobe ice and the dolomitic gravels contain Green Bay Lobe lithologies. Ice flow was from left to right; gravel was deposited by streams flowing directly into the page. M. C. McCartney for scale; pit face is roughly 13 m (44 feet) high.

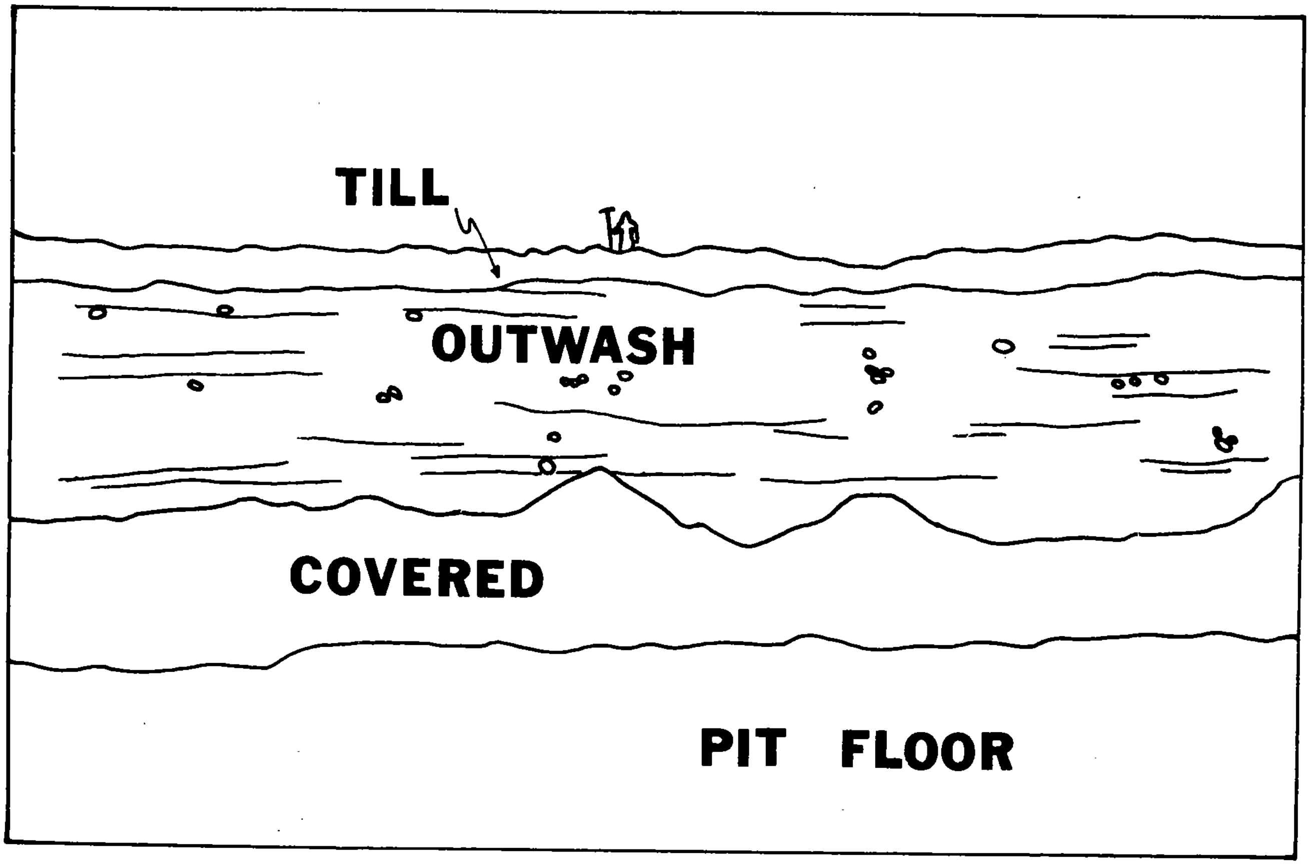

Figure 3. (cont.)

COMMON CHARACTERISTICS OF COARSE AGGREGATE

Characteristics of gravel within drumlins are generally the same as found in surficial outwash deposits. In Forest County, Wisconsin, Simpkins (1979) noted that outwash deposits exhibit a wide range of grain sizes from cobbles and coarse gravels to medium and fine-grained sands. Typically they generally are the most well-sorted (poorly-graded) of any stratified drift deposit (fig. 4). Although the greatest percentage of surficial outwash is sand, the grain size often increases downward in vertical outwash sequences. This upward-fining of outwash probably results from the systematic retreat of the ice margin and subsequent increase in distance of sediment transport during deposition of the sand and gravel. Surficial deposits of coarse aggregate are present on high outwash terraces, in pitted outwash, and in outwash fans. Thickness of the outwash can range from as little as 3 m (10 ft) in interdrumlin swales to over 40 m (130 ft) in large outwash channels (Simpkins, 1979).

Tunnel valleys, known to be associated with several drumlin fields in Wisconsin, may also contain coarse aggregate. These valleys, carved by concentrated subglacial drainage, are recognized by long clusters of deep, steep-sided kettles where they are buried by moraines and outwash plains. Deposits along these features closely resemble outwash in their degree of sorting although they occasionally may contain abnormally large boulders (Paul N. Dolliver, personal communication, 1979).

Often drumlin landscapes contain sand and gravel resources in landforms deposited on top of drumlins in a zone of rapidly melting ice margins. This group of features includes eskers, kames, kame terraces, ice-contact deltas, and large areas mappable as undifferentiated ice-contact features. As a group these deposits are usually more coarse-grained and more poorly-sorted (well-graded) than outwash (fig. 4). Deposits of ice-contact stratified drift range in thick-

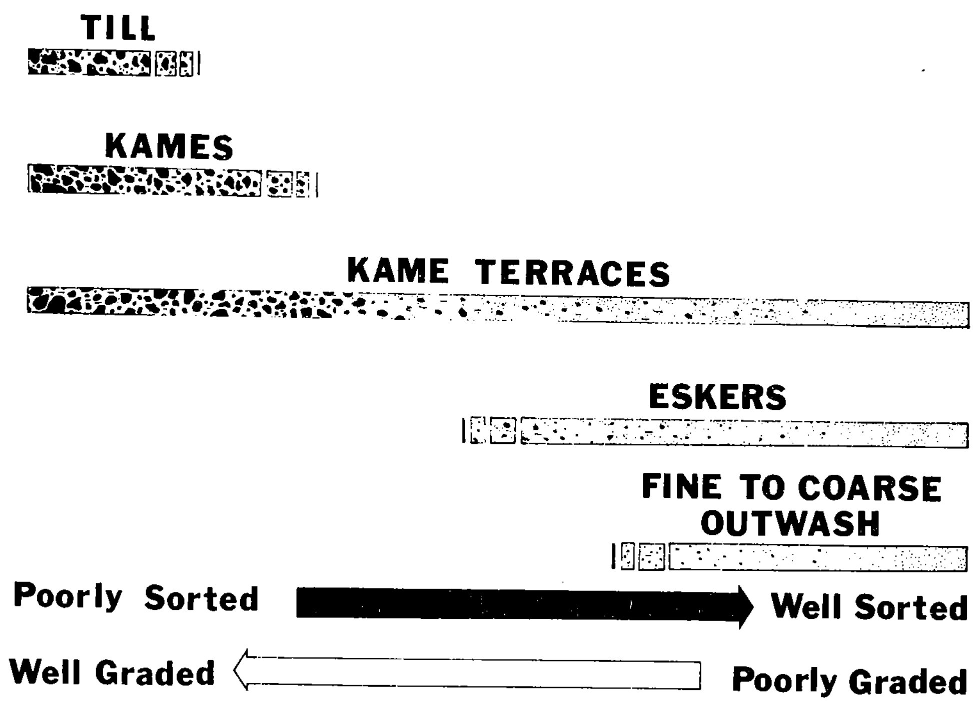

Figure 4. Diagram illustrating relative sorting and grading of material in landforms in Forest County, Wisconsin. Diagram is schematic and no scale is implied.

ness from 3 m (10 ft) in crevasse fills to 18 m (60 ft) in large esker systems, and perhaps more in large kame-and-kettle complexes (Simpkins, 1979).

The potential for finding ice-disintegration deposits is enhanced in areas of drumlinized uplands because of the greater relief. During deglaciation isolated ice blocks usually last longest in major valleys. Deposition of debris sliding off the stagnant ice and carried by meltwater tends to concentrate coarse sediment along the margins of the uplands.

In brief, therefore, drumlin landscapes can contain a great many sources of coarse aggregate. Poorly-sorted (well-graded) deposits are commonly found in surficial ice-contact materials whereas better washed aggregate is often present in proglacial outwash plains, terraces, and fans. We emphasize, however, that a useful source of well-sorted (poorly-graded) coarse aggregate may lie unrecognized within till-covered drumlins.

MINING AND USE OF GRAVEL FROM DRUMLIN LANDSCAPES

Clearly many of the stratified drift deposits in drumlin landscapes provide good potential sources of aggregate. Areas of well-sorted coarse outwash are good for use as concrete aggregate and drainlines, although certain amounts of sandier drift may be mixed in with the coarse outwash. Areas containing poorly sorted ice-contact stratified drift are also potential sources for concrete aggregate, but are better suited for asphalt aggregate and road-base material because they are generally more poorly sorted and variable in grain size. Both sources are easily mined except in areas where the water table is close to the surface as it is in areas of peat between drumlins or areas adjacent to streams or lakes. Gravels in drumlins and in most kame terraces, eskers, crevasse fills, and undifferentiated ice-contact features, generally rise above the level of the valley floor and can be mined without the restrictions of the water table.

Use of local aggregate in highway construction can reduce transportation costs significantly, particularly by mining well-sorted gravels near the proposed highway. Furthermore, since a great deal of sub-grade fill is often needed to construct roads across interdrumlin swales containing thick peat or fine-grained lake sediment, even drumlins made of solid till can provide a valuable local source of aggregate.

Several peculiar problems can beset highway construction through drumlin fields. Although some drumlins have gravel cores, other drumlins contain cores of bedrock that require blasting. Simple geophysical testing or drilling is necessary to predict rippability of drumlin materials. Furthermore, highway planners should be aware that the orientation of any future transportation route relative to the drumlins' long axes may greatly affect the design and cost of construction. By building parallel to the drumlin trend, contractors may avoid the costs of reducing drumlins to grade. However, the costs of construction over unstable lowland clays or organic material and the advantages of mining drumlins bearing gravel may dictate building through drumlin uplands wherever possible.

Gravel pit operators mining drumlins should be alert for deformation features (fig. 5). Very large wedges of fine-grained sediment are present in drumlins near Milwaukee where they have forced their way upwards toward the drumlin surface (Whittecar and Mickelson, 1979). The wedges warp the overlying gravels into nearly vertical attitudes. The clastic dikes create walls of till, silt, or clay that can impede groundwater flow. Sudden removal of the wall by digging or blasting can release unexpectedly large volumes of stored water. Other problems common to gravel pits in drumlins include an over-abundance of boulders too large for crushers or trucks, truncated gravel deposits due to faulting, and the thick overburden of "advance" till occasionally present beneath the drumlin crest.

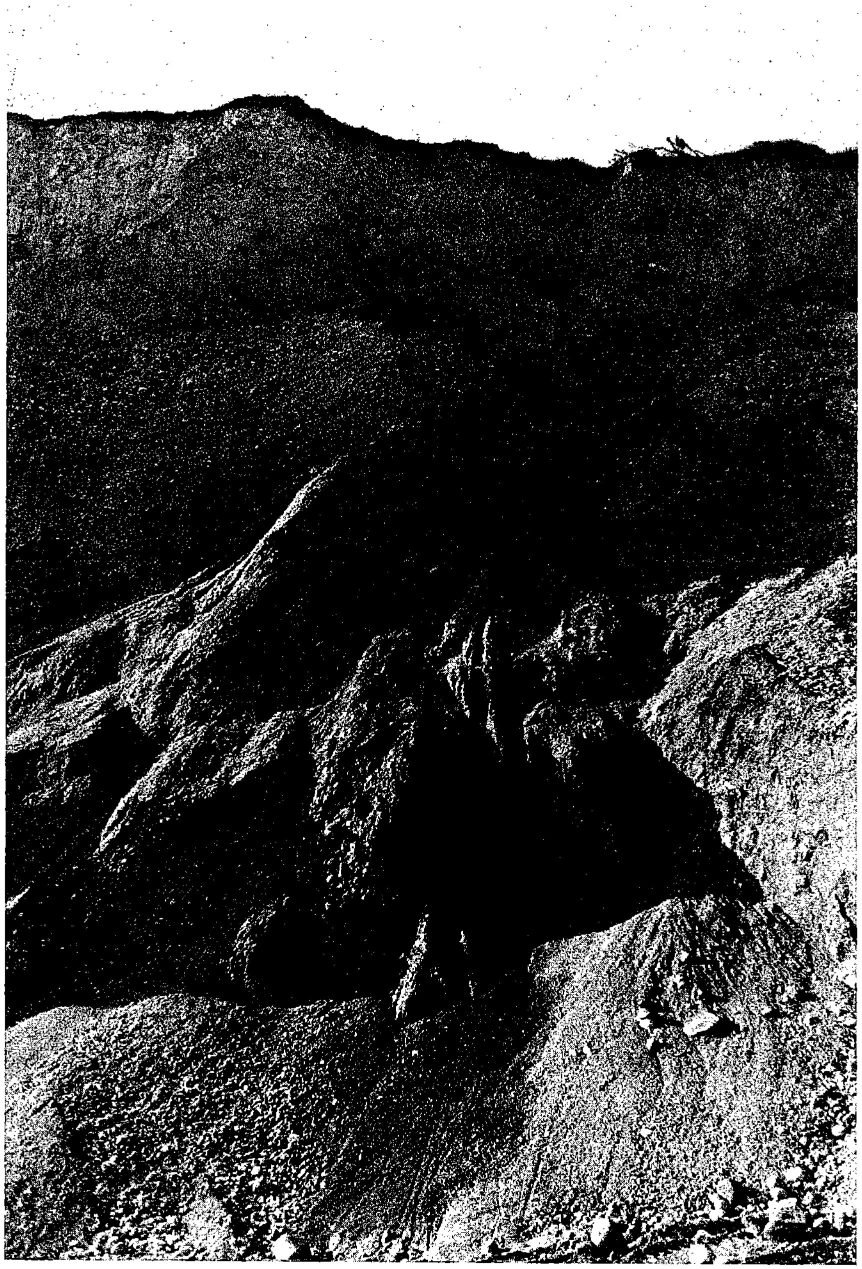

Figure 5. Photograph (this page) and sketch (next page) of faulted and upturned outwash gravels covered by till within a drumlin, Waukesha County, Wisconsin. Based upon repeated observations at this site, this structure is interpreted as deformation induced above an active clastic dike (see Whittecar and Mickelson, 1979). Pit wall is roughly 18 m (60 feet) high. Ice flow was right to left. Dark diagonal line on exposure is old surface of slumped material before latest digging.

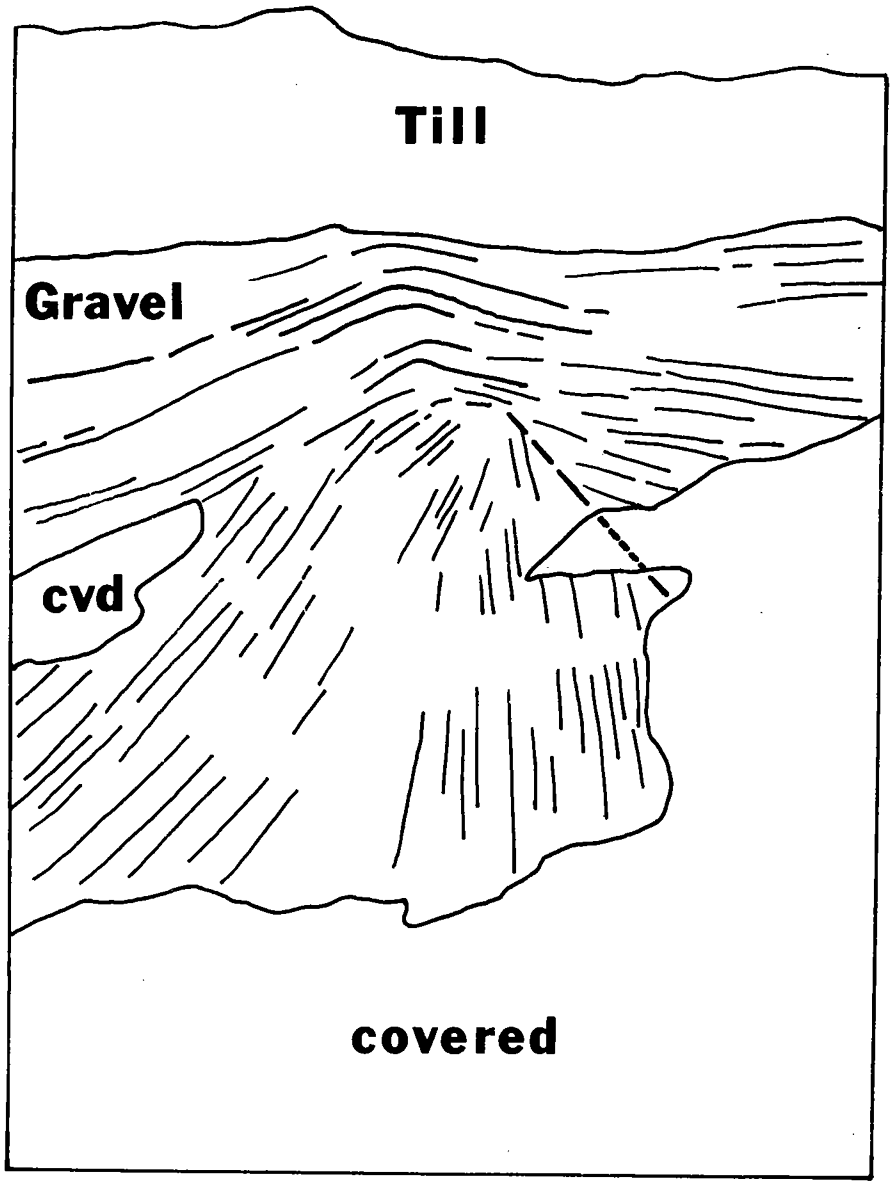

Figure 5. (cont.)

CONCLUSIONS

Highway geologists and engineers should be aware of the complex relationships present between landforms and deposits in drumlin landscapes. If identified, local supplies of coarse aggregate can reduce the costs of transportation during construction. One source of valuable sand and gravel, that within some drumlins, might easily be overlooked because of the blanket of till which covers most drumlins.

To make best use of the aggregate resources that may be available, a geologist familiar with glacial terrain should be consulted during the planning stages of any highway construction project in drumlin landscapes. Also, glacial geologists should be encouraged to map landforms and surficial deposits for potential use by highway geologists, planners, and engineers (Simpkins, McCartney, and Mickelson, 1979).

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