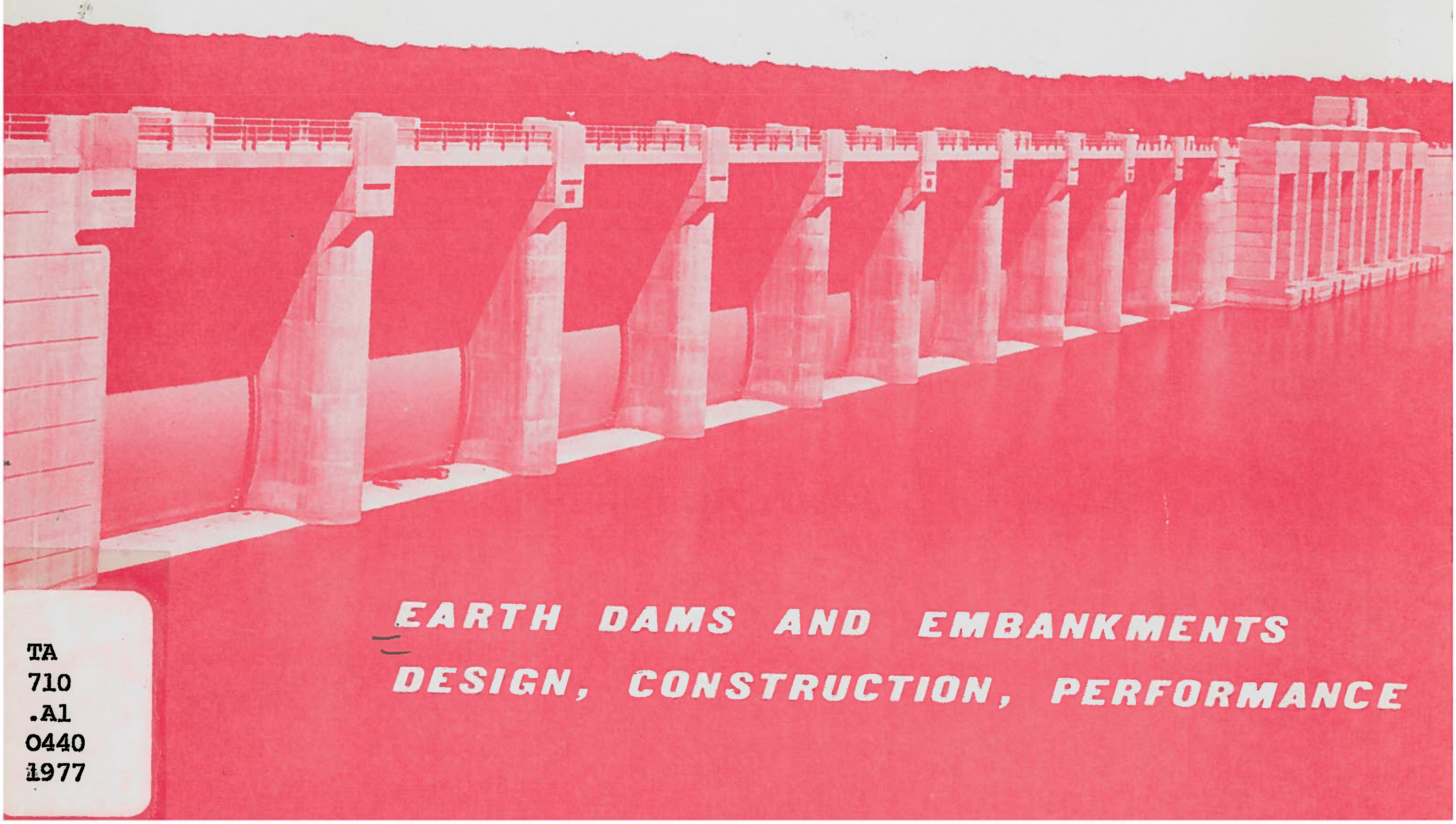
R. C. Deen

OHIO RIVER VALLEY SOILS SEMINAR 8

October 14, 1977
Louisville, Kentucky

PROCEEDINGS



EARTH DAMS AND EMBANKMENTS
DESIGN, CONSTRUCTION, PERFORMANCE

Proceedings of the Eighth
Ohio River Valley Soils Seminar on

EARTH DAMS AND EMBANKMENTS
DESIGN, CONSTRUCTION, PERFORMANCE

October 14, 1977
Executive Inn
Louisville, Kentucky

Sponsored by

KENTUCKY GEOTECHNICAL GROUP, ASCE

CINCINNATI-DAYTON GEOTECHNICAL GROUP, ASCE

UNIVERSITY OF LOUISVILLE
DEPARTMENT OF CIVIL ENGINEERING

UNIVERSITY OF KENTUCKY
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OFFICE OF CONTINUING EDUCATION
AND EXTENSION

UNIVERSITY OF CINCINNATI
DEPARTMENT OF CIVIL ENGINEERING

KENTUCKY DEPARTMENT
OF
TRANSPORTATION

PREFACE

The Kentucky Geotechnical Group and the Cincinnati-Dayton Geotechnical Group, in cooperation with the Civil Engineering Departments of the University of Louisville, University of Cincinnati, and University of Kentucky, continued in 1977, with the assistance of the Office of Continuing Education at the University of Kentucky and Kentucky Department of Transportation, the tradition of an annual Ohio River Valley Soils Seminar. These specialty seminars, begun in 1970, have been designed to provide an opportunity for interchange of ideas and information to practicing engineers and engineering students. The seminar topics are drawn from the fields of soil mechanics, foundation engineering, rock mechanics and engineering geology. Past seminar subjects are listed below:

1970	Building Foundation Design and Construction Lexington, Kentucky
1971	Earthwork Engineering, Start to Finish Louisville, Kentucky
1972	Lateral Earth Pressures Fort Mitchell, Kentucky
1973	Geotechnics in Transportation Lexington, Kentucky
1974	Rock Engineering Clarksville, Indiana
1975	Slope Stability and Landslides Fort Mitchell, Kentucky
1976	Shales and Mine Wastes: Geotechnical Properties, Design and Construction Lexington, Kentucky

A detailed listing of past seminar data, including paper titles, authors, and information on the availability of past proceedings is given in the Appendix.

The Eighth Ohio River Valley Soils Seminar was held on October 14, 1977, at the Executive Inn in Louisville, Kentucky. The seminar was organized principally by the Kentucky Geotechnical Group with the assistance of the organizations named above; that assistance is gratefully acknowledged.

In October 1976, a task committee was appointed to select a seminar theme and to organize the meeting. The members of that committee were:

C. R. Ullrich (Co-chairman)
D. J. Hagerty (Co-chairman)
(both) University of Louisville, Department of Civil Engineering
R. C. Deen, Kentucky Bureau of Highways
T. C. Hopkins, Kentucky Bureau of Highways
C. T. Gorman, Kentucky Bureau of Highways
G. Chisholm, U. S. Army Corps of Engineers
V. A. Gleason, Dames and Moore, Cincinnati
H. A. Mathis, Kentucky Bureau of Highways
V. P. Drnevich, University of Kentucky, Department of Civil Engineering

The theme for the 1977 Seminar was "Earth Dams and Embankments." The purpose of the meeting was to bring together a group of speakers who would describe the concepts and methods employed in the analysis, design, and construction of earth structures. The speakers were asked to illustrate the concepts they would describe through an extensive use of case histories.

Written texts of six presentations made at the Seminar were published in these Proceedings. The remaining presentations could not be published because of temporal and/or legal constraints binding the speakers. The task committee is sincerely grateful to the speakers who made the Seminar an event memorable to all attendees.

Geotechnical equipment and materials were exhibited during the Seminar. Members of the task committee appreciate the participation of the following patrons and exhibitors:

The Seaman Nuclear Corproration
3846 West Wisconsin Avenue
Milwaukee, Wisconsin 53208

The E. I. DuPont de Nemours Company
2265 Harrodsburg Road
Lexington, Kentucky 40503

Central Mine Equipment Company
6200 North Broadway
St. Louis, Missouri 63147

Karol Warner, Inc.
261 South Adelaide Avenue
Highland Park, New Jersey 08904

APA, Inc.
122 South 12th Street
Louisville, Kentucky 40203

The Reinforced Earth Company
1414 22nd Street, N. W.
Washington, C. C. 20037

National Ash Association
1819 H Street, N. W.
Washington, D. C. 20006

T. C. Hopkins
Chairman, Proceedings Subcommittee
D. J. Hagerty, C. R. Ullrich
Chairmen, Task Committee

SIGNIFICANT ENGINEERING-GEOLOGY FEATURES AT DAMSITES IN FLAT-LYING SEDIMENTARY ROCKS

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ABSTRACT. Geologic features significantly affecting design, construction, and performance of dams on flat-lying sedimentary rocks are: structural features related to tectonic stresses and valley downcarving (faults, joints, shears, bedding mylonites, etc.); lithologic features affecting either suitability of borrow materials (mineralogy, cementation compaction, etc.) or foundation characteristics (karstic limestone, rapid deterioration of argillaceous materials); weathering profiles; position of water table and transmissivity of geologic materials; state of stress; and spatial distribution of soils. These features are grouped according to principal causative process, agent or factor. It is emphasized that knowledge of origin of significant engineering-geology features leads to more effective exploration programs of sites and less chances of encountering unexpected conditions during construction. Significant engineering-geology features primarily related to structural geologic processes are discussed in detail. A case history is presented showing significance of origin of a soft seam in flat-lying Paleozoic beds. Features in karstic sites are also reviewed, and a case history of a site in weathered flat-lying Paleozoic limestone is presented.

INTRODUCTION

Perusal of the article on engineering geology of dams by Burwell and Moneymaker (1) shows that twenty years ago geologists had already found and described almost all the types of geologic features that are important at damsites in rock. The advances in engineering geology of damsites, in the last two decades, have occurred on two fronts. First, geologists have, through increased exposure to rock mechanics and other quantitative geodisciplines, gained a better understanding of the geologic processes responsible for these relevant geologic features. This better understanding has resulted in more effective exploration programs. Second, exploration tools and techniques have been refined considerably.

For all of his efforts to quantify geology, an engineering geologist remains a conceptual thinker; his task: problem-finding. Notwithstanding the fact that engineers--the problem solvers--cannot design dams based on concepts or even probabilities, the professional engineering geologist must resist the pressures that induce him to attach design figures to his observations and inferences. When an engineering geologist has the ability, training and experience to contribute directly to design, he is practicing engineering, not engineering geology.

More specifically, the most important and most difficult task of an engineering geologist at a damsite is not merely to show that the amount of exploration already performed has not disclosed unfavorable geologic features. By the proper choice of exploration techniques, he is to prove, to everyone's satisfaction, that certain adverse geologic features do not exist at a given site. His proper course of action at the different stages of the project depends on his ability to anticipate the geologic features that are likely to be present at the site, and his ability to understand how these features, if present, could affect the design, construction, and performance of the dam.

The purpose of this paper is two fold: to present an outline of the interrelationships between principal significant geologic features and causative geologic processes at damsites in some flat-lying sedimentary rocks; and to discuss in some detail two groups of geologic features (structural and lithological) which are important in the type of flat-lying Paleozoic sedimentary rocks that underlie the Ohio River Valley and many of its tributaries.

Table 1

Principal Causative Geologic, Factor, Process or Agent	Significant Engineering Geological Features at Damsites
1) Structural Processes	- Structural features related to tectonic stresses (faults, joints, shears) - Valley-downcarving features (stress relief joints, open joints, bedding mylonites, compression structures, and bedding separation on valley floors)
2) Lithology	- Features affecting suitability of borrow material for embankments and aggregate (mineralogy, cementation, compaction) - Solution and karstic features in limestone - Rapid deterioration of argillaceous rocks
3) Weathering	- Weathering profiles, irregular soil-rock contacts - Enhancement of features caused by lithologic and structural factors
4) Groundwater	- Position of water table and features controlling amount of seepage
5) State of Stress	- Enhancement of features caused by structural, lithologic and ground water factors
6) Recent Environments of Continental Deposition	- Spatial distribution of alluvial, colluvial glacial, eolian, etc. soils

SIGNIFICANT ENGINEERING GEOLOGY FEATURES

Varied geologic agents and processes create innumerable features in the earth materials at a damsite. Fortunately, only relatively few of these features affect the shear strength, permeability and deformability of the rock, or the spatial distribution of the different types of rocks and soils at or near the site. Such geologic features are called "significant engineering-geology features". More importantly, these relatively few features can be grouped into broad categories according to the principal geologic process agent or factor responsible for their existence.

Table 1 is an outline of the principal geologic processes, agents or factors and the resulting significant engineering-geology features at damsites in flat-lying layered rocks. This outline is partially based on a listing of features originally presented by Deere, who first introduced the concept of "significant engineering-geology features" (2). The present outline probably does not include all the geologic features that may be significant at the above damsites. However, experience and review of available case histories seem to indicate that the features included account for the overwhelming majority of geologic problems encountered during exploration, excavation, construction and operation.

An outline such as this is a useful framework of reference to bear in mind particularly during the exploration stage for a damsite. Because the outline is essentially a genetic one, i.e.: it groups the features according to probable origin or cause, it helps to anticipate the type and position of significant geologic features that are likely to be present in an area with certain known geologic conditions. The use of such a framework results in more effective and economical exploration programs. Exploration for bedding-plane mylonites is a good example of the advantages of the genetic grouping. Bedding-plane mylonites were originally observed and described as "soft seams" occurring usually at the top of flat-lying units overlain by sandstone or limestone, both at the bottom and in the flanks of river valleys. Most of the original observations were made after excavation because standard boring techniques failed to disclose their presence during exploration. Now it is generally accepted that these mylonites are the product of valley downcarving (3, 4, 5). Shear displacements occur along bedding planes, because or horizontal valley displacements (3), vertical valley rebound (4), or most probably a combination of the two types of displacement, and grind up the weaker rock at these bedding interfaces, producing the mylonite layers. Insofar as there cannot be a river valley without downcarving, the engineering geologist should assume that all sites in flat-lying sedimentary rocks, will contain mylonites, and should plan an exploration program, including special core-recovery and trenching techniques which will show that such features exist at the site or will prove that they are not present. Without this knowledge of the mechanics of mylonite formation he would have only incompletely related case-histories describing the presence of certain "soft seams" that came as a surprise during construction because they were not detected during exploration.

Obviously, the outline in Table 1 is not without limitations. For instance, it is very difficult to determine the main causative geologic factor in the formation of a specific geologic setting with a specific set of construction and performance problems. Thus, in dealing with karstic features it is difficult to decide which is the principal cause - lithology, weathering or groundwater - because these factors processes and agents act together to develop the karstic terrain. An outline somewhat similar to that in Table 1 has been proposed by Fookes (6) and criticized by Knill (7) because of such difficulties. More important, however, is the danger that such a genetic outline may prejudice the engineering geologist about some features disclosed by exploration. For example, he may recover a core containing a relatively soft seam which may actually be a depositional feature and which, although unlikely, may be unsheared in situ. Assuming that such a seam is a mylonite may penalize the economics of a project. Temporary structures such as cofferdams, that can be safely designed using shear strength values above residual, would be overdesigned because mylonites are assumed to have residual shear strengths. However, it is believed that possible prejudices can be minimized if the engineering geologist uses a genetic outline such as the one presented in Table 1, and keeps an open mind. A brief discussion of the various causative geologic factors, pocesses and agents, and the resulting features now follows.

Structural Processes

Included in this group (see Table 1) are those processes that lead to the shear or tensional fracturing of rock and the formation of roughly planar discontinuities in rock masses. These flaws or discontinuities in rock may be merely thin planes of separation (joints, fault planes, bedding planes) or may have widths varying from a fraction of an inch to tens of feet (open joints, fault zones, shear zones). The filling material in some of them is also quite varied and may display a wide spectrum of mechanical properties - from hard cementing materials such as calcite and quartz, to weak and permeable crushed rock (breccia), to weak and relatively impermeable soil-like materials (gouge). The areal extent of these discontinuities is just as variable; they range from a few square feet to dimensions many times greater than those of the damsite. The geometry of these planes and zones varies from undulating to straight. Finally the spacing varies from a few inches to several feet or tens of feet (joints), to several miles or tens of miles (faults). All of the above characteristics - openness and thickness, filling material, areal extent, geometry, and spacing - determine to what degree these discontinuities downgrade the mechanical properties of the rock masses that contain them.

For the purpose of this paper, structural processes can be subdivided into two types: Those processes which create discontinuities by virtue of stresses affecting the valley and areas beyond the valley (regional or tectonic stresses), and the process of valley downcarving which creates discontinuities only in the immediate valley area. Structural processes and features are examined in more detail in another section of this paper.

Lithology

The rock type near or at a damsite is significant to dam construction in at least three different ways.

First, the rock type may determine the suitability of the materials to be used as aggregate, rock fill, and random fill. The requirements for each use are relatively simple but the result of the tests available to establish the material's suitability are not always straightforward to interpret. It is relatively simple to determine the amount of argillaceous material, the density, and the strength of a limestone to be crused for aggregate, and it is also relatively simple to assess its future performance. On the other hand, if shale must be used as random fill or rock fill, perhaps because no other type of rock is available within reasonable distance from the site, the different tests for shale durability may lead to conflicting results. Unfortunately, there is yet not enough field data to positively assess the various tests and classification systems for shales (8). Geological observations regarding thickness of colluvial materials on shale slopes and behavior of old cuts can help somewhat in closing this gap.

Second, the type of rock may determine the excavation method. It is generally possible to rip soft shales and weakly cemented sandstones; harder rocks must be blasted. Whether ripping is an economic means of excavation can usually be determined only by actual field tests. Results from these tests can then be correlated with criteria based on seismic velocities and unconfined compressive strength of the intact materials (9).

Finally, there are certain types of sedimentary rocks which, because of characteristics inherent to the rock type, have well deserved reputation as "problem" foundation materials. Two types of sedimentary rock fit particularly well into this group; these are the so-called shales of compaction, and limestones.

Shales of compaction initially can be quite strong but they tend to deteriorate by slaking. Montmorillonitic shales slake mostly by differential swelling, kaolinitic shales slake mostly by "air breakage", and illitic shales slake due to a combination of both mechanisms (10). The Paleozoic shales in the region around the Ohio River Valley are predominantly illitic and some are known to slake fairly rapidly (11). It would appear that slaking by "air breakage" is more important in these shales than slaking by differential swelling. There is a notable scarcity of available data on swelling problems in dam foundations in the above region (shales with swelling characteristics do exist in certain parts of the general region but they owe their characteristics to their iron-sulfide content (12, 13)). Slaking by "air breakage" can occur even with the removal of very small amounts of moisture (10). Thus, damsite excavations in slaking shales can be protected by water sprinkling, or by impervious films. More commonly, a foot or so of the shale itself is left in place and excavated within 24 to 48 hours before pouring concrete.

Limestone, when unaffected by solution, is an ideal geotechnical material. However, weathered limestone masses are among some of the most difficult terrains for damsiting (14). Further discussion on the geologic aspects of damsites in weathered limestone is presented in a subsequent section.

Weathering

Weathering processes can also degrade the engineering quality of rock masses. Rocks develop characteristic weathering profiles which depend in part on the rock type, combinations of rock types, and the structural features at a particular site. Patton and Deere (15) have reviewed the stability of slopes underlain by residual soils and have emphasized the importance of the weathering profiles. Weathering of argillaceous materials produces a shallow zone of totally weathered material which characteristically has low permeability and low shear strength. This zone can vary in thickness from several inches to few tens of feet depending on whether the underlying shales are shales of cementation or shales of compaction respectively. Beneath this surficial zone is a zone of fractured, partially weathered rock which has high permeability and shear strength intermediate between those of the weathered material and the intact rock. The combination of the upper and middle zones may lead to excess water pressures in the slope and to instability. The fractured zone, of course, grades into the unweathered rock beneath. Some weathering profiles in limestone, on The other hand, do not present a gradation zone from the residual cover - usually red clays and silty clays of medium to high plasticity - to the fresh, unweathered rock. However, many profiles in limestone do exhibit an intermediate zone with limestone core-stones and chert fragments of varying size. The depth to unweathered rock in weathered limestone profiles may range from a few feet to a few hundred feet (depending in part of geologic factors to be discussed later). Finally sandstone units will weather in diverse modes which depend on the mineralogy of the grains and cementing agent, and on the degree of jointing. The depth of weathering thus can vary from a few inches to a few tens of feet. It is apparent, then, that intercalations of two or three of the types of rocks described above at a damsite will result not only in different depths of weathering zones, but also in weathering profiles with different engineering characteristics. Structural features at the site will make the weathering zones more irregular and complex.

Weathering may affect in different ways the mechanical properties of structural discontinuities in rock. Thus, weathering may heal open fractures by deposition, of strong chemical precipitates, it may have little or no effect on the strength of gouge zones in argillaceous rocks; or it may decrease the strength and permeability of joints, shears, and faults in sandstones and limestones by decomposition of the sound rock and deposition of clayey materials. In connection with this last effect, many faults may be quite permeable in a direction parallel to the fault plane, but may have very low permeability in a direction perpendicular to that plane.

Some of the engineering concerns related to weathering are the amount of each rock type to be excavated; the type and the amount of treatment of the material to be left in place; and in the case of embankment dams, the geometry of the excavation as well as the geometry and cross section of the embankment itself. The extent to which the solutions to these concerns differ before and after excavation is a function of the accuracy of the geologic model developed by the engineering geologist during exploration.

Groundwater

The groundwater conditions at a future damsite are a function of the regional flow system and the geologic details of the site. Important changes in the groundwater conditions are introduced by excavation for a dam, but potential problems can be averted if the factors that control the local groundwater regime are disclosed during exploration. Diversion and dewatering schemes are also controlled by the local geological and hydrological features.

One of the most significant contributions to understanding the groundwater conditions at some damsites was made by Hubbert (16). He used a simple, isotropic model to show that, in a regional groundwater system, valleys are in many cases discharge areas, and that under such conditions upward groundwater flow is to be expected under the valley bottom. This type of flow produces pressures in excess of hydrostatic pressure under the valley floor (assuming the river level as the hydrostatic level). Hubbert's concept can be extended to flat-lying layered rocks and its validity has been demonstrated analytically (17) and in the field (18). In fact, the presence of shales overlying rocks with high transmissivities such as sandstones and fractured limestones can produce very high water pressures in the more permeable beds. Without the concept of a regional flow system, these artesian conditions may not be expected in the flat-lying beds, particularly if one is relatively certain that there is no communication between the river and the deeper, more permeable beds. Thus, without provisions for proper dewatering the high pore water pressures in the beds could lead to a blow-up and perhaps flooding of the excavation.

Local geologic details, such as permeable soils, karstic limestone and other weathered rock above the foundation level, usually control the amount of seepage into the excavation at a given river level. But equally or more important than the amount of seepage is the amount of excess pore pressures developed in structural features such as bedding-plane mylonites, open bedding planes and faults that are in hydraulic communication with the river. Temporary or permanent structures founded on these features could easily slide if drainage is not provided.

State of Stress

The initial state of stress in a rock sequence has an important effect on the development of structural features associated with valley downcarving. Elastic theory shows that for a given initial vertical stress value the higher the initial horizontal stresses the larger the horizontal and vertical rebounds. The amount of rebound is believed to be responsible for the degree of development of those structural features (4). Thus, areas that have developed high horizontal stresses, because of previous deep burial or ice loading, can be expected to have better developed structural features than areas with lower initial horizontal stresses, provided the material properties of the rocks and valley depths are comparable in both areas.

During excavation, the existing state of stress in specific locations such as the bottom of a valley, may help induce buckling failure of the beds. Grouting in horizontally stressed layers may uplift the dam foundation. Of course, underground appurtenant structures will be more directly affected by the present state of stress near the valley walls. Engineering geologists can develop a feeling for the magnitude of horizontal stresses in an area from a knowledge of the geologic history of the area, and from observations of the deformation and present behavior of rock in existing quarries and tunnels.

Environments of Continental Deposition

When available, transported soils--principally alluvial and glacial soils--are main sources of borrow for embankment and aggregate material. Exploring for adequate volumes of a particular type of soil can be an extensive and expensive enterprise. Relatively recent developments in sedimentology and glacial geology (19, 20, 21) indicate that in some cases the geometry and spatial distributions of sediment bodies tend to conform to certain models. For instance, rivers with a high degree of meandering, tend to deposit coarse-grained materials in narrow zones across the flood basin; the rest of the basin will be filled with silts and clays. On the other hand, rivers with low sinuosity tend to distribute sediments of all sizes evenly throughout the flood basin. It is suggested that engineering geologists look more critically at these existing models as potential aids in planning exploration programs for these borrow materials.

FEATURES RELATED TO STRUCTURAL PROCESSES

Features Related to Tectonic Stresses

Although in general, faults, shears and joints formed by regional or tectonic stresses lower the quality of rock masses, when found at damsites, these structural features will affect the project in degrees than range from negligible to very critical.

Tectonic faults, at first thought, appear to be the most ominous and important features controlling sliding stability of dams. However, considering the flat-lying Paleozoic beds along the Ohio River Valley, these faults are probably less troublesome than some of the stress-relief features. First, large portions of the Ohio River Valley region are not faulted; when important faulting is present, it tends to be confined to narrow trends or zones, e.g. Rough Creek trend. Further, the majority of both major and minor faults tend to have high dips. This last characteristic is relevant to sliding stability of dams since the most unfavorable structural discontinuities of any type are those with low to very-low upstream dips (up to 20^{o}). This is not to say that high-angle faults, critically oriented in the abutments cannot intersect other discontinuities such as bedding-plane joints or mylonites to create potentially unstable blocks regarding sliding. Tectonic faults do tend to be associated with fracturing and brecciation which can considerably increase weathering. As mentioned earlier, brecciated material in fault zones may have very high transmissivity.

The terms "shear zone", "shear planes", or simply "shear" are used somewhat loosely and subjectively in engineering geology. In general, the term "shear" is commonly used to denote structural discontinuities that present some evidence of shearing activity (gouge, striations), but which either cannot be demonstrated to be faults (lack displaced elements), or whose demonstrable displacements seem too small. Thus, shears tend to be not as thick nor areally extensive as compared with faults. But they usually do possess enough planarity, and continuity, and low-strength fillings to be considered a serious problem with regard to sliding stability; in addition they are more prone to be missed during exploration.

Tectonic joints are ubiquitous discontinuities in rocks. Flat-lying rocks typically display two orthogonal, vertical-to-subvertical sets, and one horizontal-to-subhorizontal set. If tight and unweathered, tectonic joints usually do not pose any problems. They may transmit water pressures but their shear strength is normally more than adequate (22). Near the surface tectonic joints are affected by weathering and the valley downcarving processes, resulting in joints filled with soft materials or open joints. These joints, obviously, can introduce problems of leakage, piping, and sliding stability.

In general, the presence of faults, shears and joints does not necessarily imply that serious problems will be encountered at a damsite. Probably the most common problem in connection with these structural features is the high transmissivity of open and weathered joints. In limestone terrains, the open joints are not only responsible for the flows entering an excavation today but they are principally responsible for the degree of solution that these rocks have undergone in the past. Sliding stability is a problem only when these geologic features have particular orientations, i.e.: gentle upstream dips. In addition, they usually affect only concrete structures; embankments, with proper drainage, can usually be shown to be stable. Tectonic faults, shears and joints very seldom affect the deformability of the flat-lying rocks to be a problem in damsites for embankment dams.

The surface expression of some tectonic features at damsites can be detected during exploration on the basis of air photos, topographic and geologic maps, and field observations. If the site is within or adjacent to known regional structural trends, faults and shears should be expected and subsurface exploration should be planned considering their anticipated attitudes. Good drilling techniques are a necessity and special attention should be paid to poor core recovery, signs of weathering in the recovered material, water losses, drops of drill stems, plugged bits, changes in color of drilling fluid, etc.

Double or triple barrels and NX- or larger-sized cores should be used to insure acceptable recoveries. Special techniques including calyx-holes, downhole camera logging and integral sampling may be required to gain a better knowledge of zones of particularly poor quality rock.

Features Related to Valley Downcarving

In the last few decades, it has become apparent that a series of structural features observed adjacent to valleys are the product of stress relief associated with the formation of the valley itself. Many of these structural features are continuous over several hundred feet and include wide openings on the valley walls and horizontal layers with residual shear strengths. Thus, their influence can be potentially very deleterious to a dam if they are not properly explored, understood and allowed for in design.

Ferguson (3) first described the stress relief features in flat-lying Paleozoic beds at damsites in the Allegheny Plateau. These features were of two types. Those occurring in the walls above the rock floor of the valley were tension cracks and were explained in terms of the horizontal rebound of the valley walls responding to horizontal stress relief. The second type occurred under the valley floor and included: 1) buckled beds, 2) overthrusts, 3) vertical fractures, 4) open bedding planes and 5) bedding-plane mylonites. This last group of features was explained in terms of compressional failures in the valley floor created by the horizontal displacement of the valley walls. All the above features are shown in Figs. 1 and 2.

Ferguson described stress relief joints as vertical joints and observed that their spacing was a function of bed thickness and lithology. Whereas it is correct that these vertical joints are caused by river downcarving and therefore by stress relief, the writer believes it should be emphasized that these joints are also the direct result of the differential strains caused by the different mechanical behavior of the rocks in the sedimentary sequence. Because there is another type of joint, not necessarily vertical, caused solely by stress relief and because the two types can be found at different distances into the valley wall, it is well to discuss the two types separately and in some detail.

Stress relief joints have been described by others (23, 24, 25) in valleys and man-made excavations of uniform lithology (massive limestones, sandstones, layered volcanics and intrusives). These joints are roughly parallel to the surface of the valley or excavation and can be observed to occur into the walls for distances that are small compared to the depths of the valley or excavation. These joints are believed to be extension fractures caused when the ratio of tangential stresses (both in the direction of the slope and in the horizontal direction) to normal stresses (normal to the valley or excavation wall) reaches a critical value. Only those stress relief joints closest to the surface show a tendency to open. Fig. 3 is a photograph of typical stress relief joints in a layered sequence.

Vertical joints--of the stress relief type described by Ferguson--typically occur in flat-lying sequences of different lithologies. The contrast in mechanical properties of the different rock types seem to be the most important factor in the development of these joints. Sandstone-shale combinations appear to be the most critical for this type of vertical joints. The contrast in behavior between these two rock

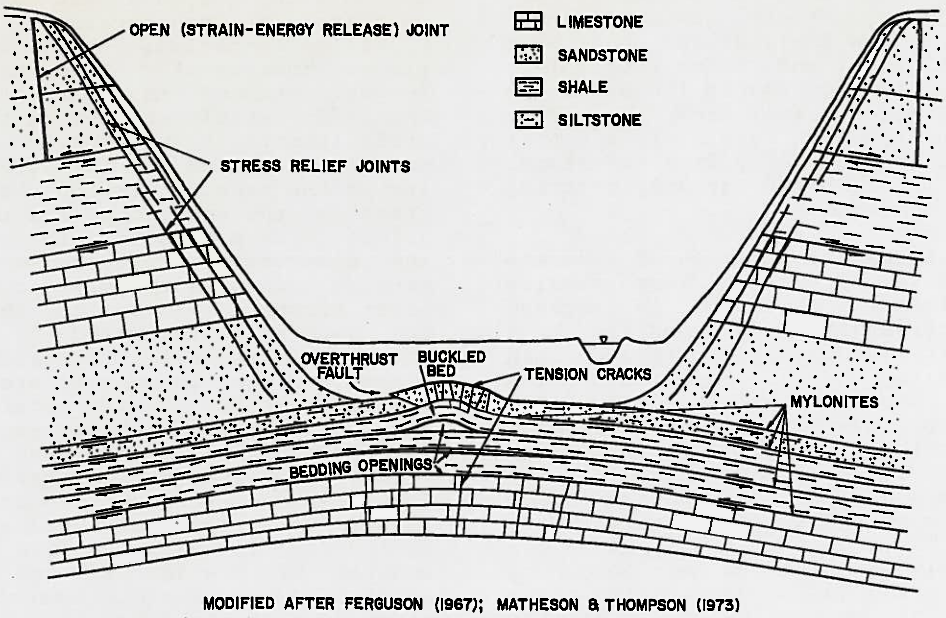

Fig. 1. Structural features in flat-lying sedimentary rocks created by valley downcarving.

types is enough not only to create joints in the sandstone but to open them to considerable widths.

Open or clay-filled vertical joints, parallel to the valley axis occur in sandstone beds underlain by or interbedded within shales (26). The writer has observed this type of joints in Paleozoic sandstone-shale sequences in south-central Ohio, southern Indiana, north-central Illinois, and south-central Wisconsin. The joints have a spacing approximately equal to the thickness of the sandstone beds and some can have widths of more than 1 ft. As with the joints formed solely by stress relief, they are difficult to detect from the surface because they strike parallel to the valley axis, and the upland surface above the walls is usually covered by soil and vegetation. In all cases observations of the joints have been made in key trenches or quarries. Because of their attitude and the irregularities of valley walls, the probability exists that they may daylight upstream and downstream from a dam. Thus, they are quite significant in considerations of leakage, abutment stability, and piping in embankments.

Fig. 2 depicts schematically the conditions found on the walls of a key trench for a 100-ft.-high earth embankment founded on Pennsylvanian rocks in north-central Illinois. Here, open vertical joints were present in a 6-ft.-thick massive sandstone interbed within a shale sequence; they had an average spacing of 6 ft. and the joint closest to the valley slope had opened about 5 in. A few thinner sandstone beds

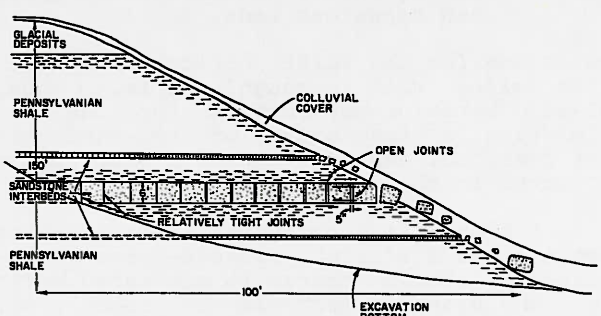

Fig. 2. Diagram showing downstream wall of abutment key for earth embankment in north-central Illinois.

Fig. 3. Typical stress relief jointing in slope in layered rock.

exhibited narrower open joints at closer spacings. The shale beds above and below the sandstone beds were conspicuously devoid of any joints; these shale beds contained only a few small cracks that were discontinuous, tight and randomly oriented. The vertical open joints in the sandstone closed progressively away from the valley slope and could be seen relatively tight at about 100 ft. horizontally from the slope. The open joints required gravity grouting from inclined boreholes.

Fig. 4 shows a photograph of a vertical, clay-filled crack in very massive basal Pennsylvanian sandstone in southern Indiana. The sandstone is underlain by a limestone unit which in turn is underlain by a shale unit. The excavation is for the abutment key of a 35-ft.-high earth dike. Except for the clay-filled joint, the sandstone unit does not contain any discernible joints (the parallel, subvertical features in the photograph are drill casts of previous blasting holes) until the gulley wall is reached. There, a few joints can be observed roughly parallel to and occurring some feet into the gulley wall. The clay-filled joint is nearly 100 ft. from the gulley wall measured horizontally along the

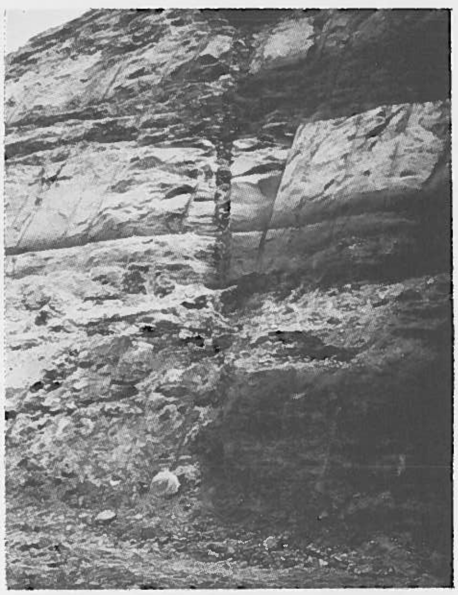

Fig. 4. **Clay-filled joint, approximately** 10 in. wide, in massive sandstone; abutment excavation for small dike in southern Indiana.

bottom of the excavation and has an average width of about 10 in. for most of its height.

The width of these open joints can hardly be explained in terms of frost-wedge action or creep along the valley slopes. These joints are too deep and extend too far into the valley walls to be associated with these slope processes. Ferguson (3) alluded to a dragging mechanism to explain the cracking of the more competent beds. A probable mechanism of similar nature for the development of the joints is described in the next paragraphs.

Horizontal tensile stresses and strains within a sandstone layer interbedded with, or underlain by shale can be generated if, upon horizontal stress relief by valley formation, the shale units displaces horizontally into the valley for longer distances than the sandstone beds. The differential strains at the sandstone-shale interfaces would create a drag effect on top and bottom of the sandstone resulting in the tensile strains that would first fracture the sandstone and then open the joints. This mechanism is consistent with the observation that the spacing of the tension joints is roughly equivalent or at least proportional to the thicknesses of the beds as interpreted by Price (27). However, when realistic values of initial stress and elastic modulii are assumed, the elastic tensile strains obtained are many times smaller than those required to account for width of the open joints in the field. A differential strain $\Delta E_h = 0.001$ is obtained if one assumes uniaxial conditions (most critical case); an initial horizontal stress $\sigma_h = 4000$ psi; elastic modulii of 5×10^6 psi and 10^6 psi for sandstone and shale, respectively; and elastic unloading from point A in Fig. 5 to negligible horizontal stresses. With a differential strain of 0.001 the total differential displacement between shale and

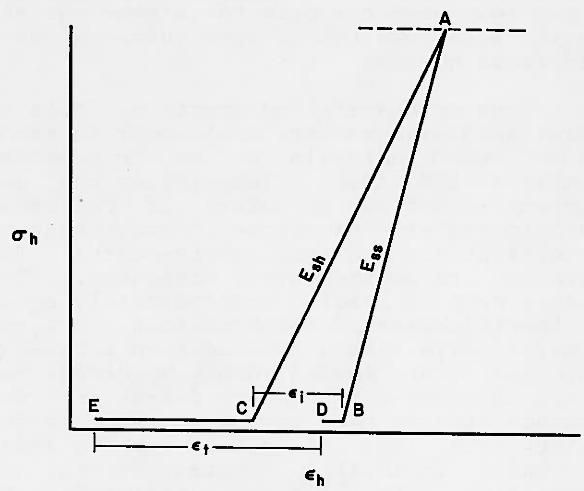

Fig. 5. Hypothetical elastic and time-dependent stress-strain relations during valley downcarving for shale and sandstone beds.

sandstone for the first horizontal 100 ft. into valley wall is roughly 1 in. Thus, elastic behavior may account for the initial tensile fracturing of the sandstone but does not explain the wide-open joints observed in the field.

A mechanism involving the different amounts and different release-rates of recoverable strain energy is suggested herein. The proposed mechanism is probably not the only one contributing to the opening of joints in this type of sandstone-shale sequences but it is likely to be very important considering the rocks involved. The concept of recoverable strain energy was first introduced by Bjerrum (28) to explain the progressive failure of slopes in over-consolidated clays and clay shales. The strain energy absorbed by many earth materials during long-term consolidation and burial is apparently not recoverable immediately upon stress relief because diagenetic bonds that developed during the long periods of load application prevent immediate strain-energy release. The rate of destruction of these diagenetic bonds determines the rate of strain-energy release. Nichols and Savage (29) have presented abundant field and experimental evidence of long-term, time-dependent, strain-energy release in many types of rock.

Consolidation tests show that at a given stress level and for a given stress increment, clays absorb more strain energy per unit volume than sands. It is reasonable, then, to infer that for a given depth of burial, shales have stored more strain energy per unit volume than sandstones. Further, because the rate of horizontal strain-energy release after horizontal stress-relief, depends on the strength of the diagenetic bonds, this rate should be much faster in the shales than in the sandstones under consideration. Mead (30) observed that shales which only have been consolidated during burial have much weaker diagenetic bonds than shales which have undergone cementation (calcareous, siliceous, ferruginous, etc.). Also, Lee and Lo (31), who have performed measurements of strain-energy release in rock cores from southern Ontario, indicate that dolomites will tend to have much lower rates of strain-energy release than shales. From these observations, one may conclude that diagenetic bonds resulting from the cementing action of chemical precipitates are stronger than diagenetic bonds resulting only from consolidation of clay particles. Since, the midwestern shales under discussion are all consolidation shales, the rate of strain-energy release should be considerably faster than that of the adjacent cemented sands (sandstones).

There are, then, reasons to believe that the wide vertical joints in the sandstone beds are caused by the greater and faster time-dependent strain-energy release of the shale beds. This is illustrated qualitatively in Fig. 5 which shows a greater time-dependent strain for shale (EC) than for sandstone (BD). Thus the total differential strain, $\Delta\varepsilon_t$, necessary to account for the open joints is much greater than the elastic differential strain $\Delta\varepsilon_i$. The writer is not aware of any in-situ long-term rebound measurements in the Paleozoic compaction shales of the Midwest that would allow checking of the present hypothesis. On the other hand, the relatively large and rapid time-dependent rebound of the upper Cretaceous compaction shales of central Canada, Montana, and the Dakotas is well documented. Fleming and others (32) report average annual rates of rebound of 0.04 in. in the Bearpaw shale at the Fort Peck Dam spillway (Montana) and a cumulative rebound between 1 and 2 ft. since 1937. Matheson and Thomson (3) have measured amounts of vertical rebound caused by valley downcarving for several valleys, up to 200 ft. deep, in Alberta, Saskatchewan, and Manitoba. The rebound at the center of the valley ranges between 3 and 10% of the valley depth, whereas man-made excavations in the same area have maximum vertical rebounds of less than 1% of the excavation depth. The difference is explained in terms of time-dependent rebound occurring during geologic time. The amount of vertical rebound can be as much as 20 ft. but typically ranges between 6 and 10 ft. The amount of maximum vertical rebound can be shown to be approximately equal to the maximum horizontal rebound if the original vertical and horizontal stresses are assumed equal. In situ stress measurements indicate that this last assumption is not unrealistic. Thus, maximum amounts of horizontal displacement of up to 20 ft. can be expected. These upper Cretaceous shales have different clay composition (montmorillonitic) and rebound characteristics from the midwestern shales (predominantly illitic). However, a small fraction of the long-term rebound measured in the Cretaceous shales could account for the strains necessary to explain the vertical open joints in the sandstone beds at the midwestern sites.

As mentioned above, the vertical open joints may be very significant at damsites in shale-sandstone sequences. However, they may be particularly difficult to detect in very massive sandstone interbeds within shales, because of their wide spacing, and more importantly, because they may occur at greater distances into the valley walls than are normally associated with the classical stress relief joints as shown in the example from southern Indiana.

The other stress relief features shown in Fig. 1 will be briefly discussed now. Mylonites in the valley walls and under the valley floor have been explained by Matheson and Thomson (4) as a result of the flexural slip along bedding planes. This slip is caused by the bulging of the beds which in turn is the result of vertical rebound. Numerical models used by these authors confirm qualitatively the origin of both the bulging and mylonites. Overthrust faults, buckled beds and open bedding planes are definitely compressional features and require the inward movement of the valley walls as postulated by Ferguson.

In summary, significant structural features seem to be inextricably associated with the formation of the valley itself. All these features can lead to adverse conditions during excavation, construction and operation of a dam. A knowledge of their probable origin is important in the exploration and design stages. Their presence can be disclosed only with the special procedures given in connection with tectonic structural features.

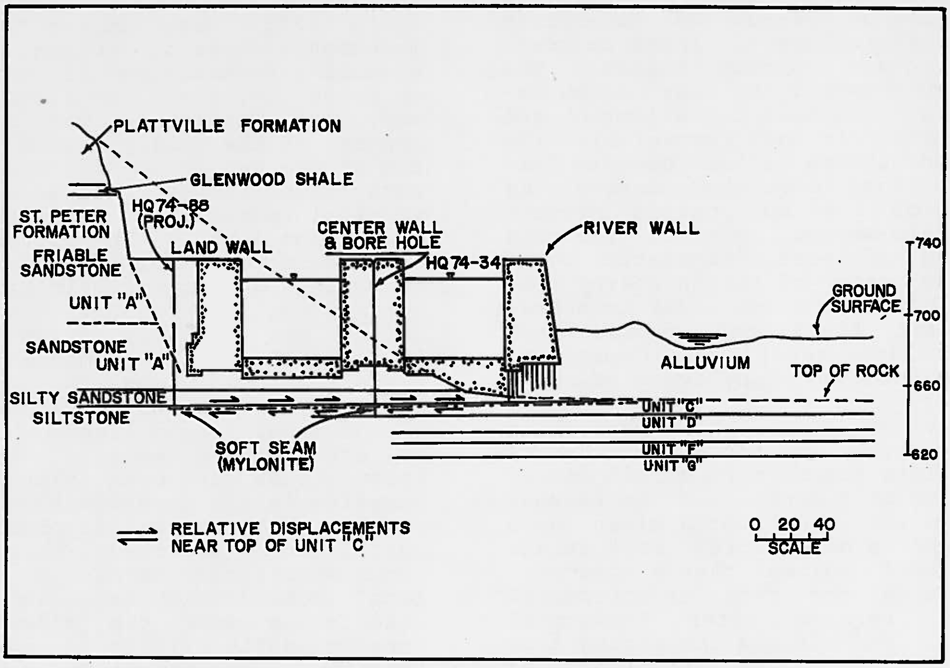

Fig. 6. Cross-section showing position of soft layer under locks in the Mississippi River, near Minneapolis.

Case History

The following example shows the significance of the knowledge of the geologic origin of a soft layer in rock. The soft seam in this example was found under two locks in the Mississippi River near Minneapolis, during the geotechnical exploration undertaken as part of a study to improve the existing locks. Careful drilling showed that the soft layer was continuous under the area of the locks and that the layer had an average thickness of 2 to 3 in. The soft layer was found at the level of the rock floor of the valley, within a sequence of flat-lying Paleozoic beds (St. Peter formation); it was overlain by a silty sandstone and underlain by a shaly siltstone layer. Fig. 6 shows a cross-section of the locks, the position of the soft layer, and two (HQ-74-88 and HQ-74-34) of the several exploration boreholes. The improvement works included replacing the river wall and this required removing the alluvial material on the river side, the river wall, and part of the river chamber. Since the land lockwall would remain operative during the improvement work, it was critical to examine the stability of the middle lockwall. This consideration led to investigate the shear strength properties of the soft layer.

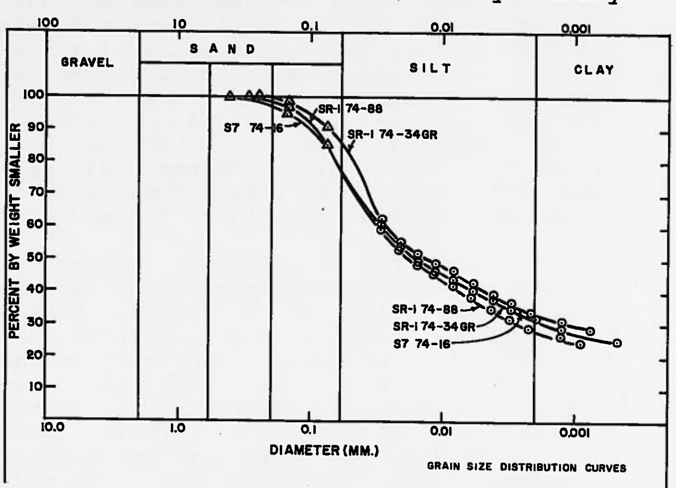

Fig. 7. Grain size curves for ground-up clayey siltstone (GR) and two samples of soft layer material.

Given the position of the soft seam, it was strongly suspected that such feature was a bedding-plane mylonite. However, soft layers that are depositional features,

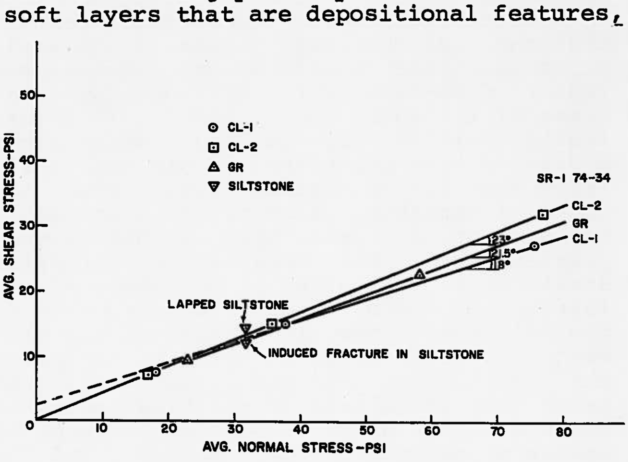

Fig. 8. Coulomb envelopes, residual conditions, for various types of samples from core SR-1-74-34.

e.g.: underclays, are also found in some midwestern Paleozoic sequences. It was the writer's opinion that if it could be demonstrated that the layer was a mylonite and not a depositional feature, residual strength values had to be assumed for calculations of short-term and long-term stability. If, on the other hand, it was not possible to show that the layer was a mylonite, the layer could be a depositional feature which, although not likely, could have some strength above residual. Residual strength then could be used for stability calculations of permanent structures, but a value above residual could be used for stability analysis of temporary structures associated with the improvement works. Of course, this latter design criterion would have required testing of large-diameter, undisturbed samples of the thin, soft layer, or even better, in situ testing. Direct shear tests could have been run on the HQ-sized cores containing the relatively undisturbed soft material. However, the absence of peak strength in those tests could have not been construed as definite evidence for a mylonite because of the uncertainty about amount of disturbance of the samples during coring and specimen preparation. Therefore, a testing program was designed to ascertain, by different means, if the soft layer was indeed a mylonite. Three HQ-sized cores (SR-1-74-34, S-7-74-16 and SR-1-74-88) containing the soft layer were used for the study. Core SR-1-74-34 was separated into three portions. The first portion was made up of the more plastic zones of the soft layer; it was remolded and designated CL-1. The second portion consisted of the siltier material; it also was remolded and was designated C-2. The third portion was the clayey silt underlying the soft layer; this portion was ground up only using mortar and pestle, was sieved through a No. 70 sieve and labeled GR. These three portions and the remolded soft layers from the other cores were characterized by means of Atterberg limits, drained direct shear tests with shear displacements of up to 2 in., grain size analyses, and x-ray diffraction. Additionally, direct shear tests were performed on induced fractures and lapped surfaces in the clayey siltstone.

Fig. 7 shows the grain-size curves for the ground-up clayey siltstone (SR-1-74-34-GR) and the soft material from the other two cores. The curves show essentially the same grain-size distribution for the two types of materials; in fact, the variation between the soft material from the two different cores is greater than the variation between the ground-up clayey siltstone and the soft material samples. Fig. 8 is a Coulomb diagram at residual conditions for the different types of samples prepared from core SR-1-74-34. The friction angles vary between 18° and 23°; the cohesion intercept can be neglected. The residual strength for all the types of materials including the rock-on-rock tests is essentially the same. Again there seem to be a greater difference between samples CL-1 and CL-2 than between either one of these and the ground-up material, GR. X-ray data showed that both the soft seam and the ground-up clayey silt had clay-sized fractions composed of kaolinite, illite and quartz. The Atterberg limits of the ground-up siltstone were w_L = 40% and PI = 19%--a few percent lower than the average but within the range of corresponding values for the soft layer samples. From all the above results and the position of the soft layer in the valley wall it was concluded that such feature had been formed by the grinding of the clayey siltstone layer. The soft layer was quite conclusively a mylonite; testing of undisturbed samples was not necessary.

FEATURES PRIMARILY RELATED TO LITHOLOGY: KARSTIC LIMESTONES

Limestone[(1)] terrains having undergone extensive solution are called karstic terrains; they present some of the most serious problems during every stage of a dam project. Very carefully planned exploration programs may often fail to uncover important adverse features at these sites. During excavation, costly and slow methods may be required to remove residual limestone blocks (see Section 2) adjacent to soft plastic clays. Foundation elevations and abutment profiles may vary considerably over short distances because of differences in the depth of solution. Remedial work--usually removal of residual clay and boulders and replacement with concrete--is again time-consuming and expensive. Trenches and drifts used for this remedial work may reveal the solution features to be more extensive than anticipated, and difficult decisions have to be made regarding how far the remedial work should continue. Excessive (mostly in terms of dewatering cost) seepage into the excavation may occur. After the dam is built and the reservoir begins to fill, excessive leakage, not near the dam but around the reservoir may render the project less economically attractive or an economic failure. In a few occasions, severe leakage may prevent filling of the reservoir. Finally, and most importantly, piping of the residual silts and clays may occur over several years next to the dam creating flows that may impair the stability of the slopes just downstream from the dam; or if the dam is of the embankment type, these high flow may eventually erode the imperious core.

After the preceeding litany of difficulties, it should be apparent that, if it is imperative to build a dam on solutioned limestone, the amount and quality of exploration should be far greater than at other sites. Engineering design should be flexible to allow for the inevitable unforeseen conditions. Londe (14) and Deere and Peck (33) have reviewed the engineering

(1) Dolomitic-limestone and dolomite terrains also undergo solutioning and develop karstic features. However, for clarity of discussion, it is assumed that the sites are predominantly limestone and that dolomite beds occur as minor intercalations.

problems of dams founded in karstic limestone. The following discussion emphasizes the causes of these engineering problems: the geologic controls that determine the severity of solution. It is through increased knowledge of these controls and their interaction, along with better exploration tools, that chances of unexpected conditions will be reduced.

The principal geologic controls affecting the complexity of sites in karstic terrains are: structural features; lithologic variations within the limestone sequence and lithologic field relations; and surface and subsurface water conditions, both past and present. Factors such as climate and geologic time are not significant in causing differences in weathering in areas of limited extent such as a dam site.

Fractures of any type (faults, shears, joints, and bedding joints) will be easily enlarged in a soluble limestone sequence. Thus, solution will proceed deeper in highly fractured, thinly bedded sequences than in sparcely jointed, massive ones. Tectonic faults are particularly affected but so are stress-relief features under the valley floor and in the valley walls. The latter features may be partially responsible for some of the deep, narrow and vertical solution channels under valley floors, the extensive solution along thrust faults also under valley floors, and the open caves parallel to the valley walls described by Moneymaker (1, 34). Joint intersections are notably prone to extensive solution. Regarding structural controls it should be remembered that very long geologic time periods may elapse between two flat-lying limestone formations. During that time the lower unit may have developed intense tectonic fracturing which did not affect the unit deposited subsequently. This difference in fracturing may result in a highly solutioned unit underlying a unit virtually free of solution features. Fig. 9 shows two of a series of clay-filled, vertical solution features exposed by a deep excavation in limestone. The solution features stop abruptly, at the formational contact; the upper formation is conspicuously free of solution activity.

Subtle compositional variations in the limestone units can influence the degree of solution and amount of overlying residual soil. Pure, coarsely crystalline limestones are more readily soluble than dolomitic, argillaceous, and cherty limestones, and far more soluble than dolomites. Impure limestones tend to have thicker residual soil covers than pure limestones (35). An intervening layer of lower solubility and permeability, can be effective in slowing or stopping solution of underlying units. Shale covers are, of course, quite effective in this respect whereas sandstone covers seem to accelerate solution of under-lying limestones (36).

Quite often the most intense solution activity occurs near the water table and decreases both upwards and downwards (37). The increase in solution at the water table is the result of faster flows and chemical reactions that occur when vadose and ground water mix (38). Again, it must be born in mind that, in the geologic past, the water table may have affected a limestone unit at different elevations above the present water table. The possibility also exists that depressed water tables during Pleistocene times may have induced solutioning below the present water table. Another complicating factor may be the upward regional flow under valleys described in an earlier section. This last factor may be partially responsible for some deep (100-200 ft.) solution features below valley floors (1). Finally, sharp bends in entrenched rivers may cause the water to flow primarily into the valley wall forming the outside bend and may produce solutioning in this wall while leaving the other wall relatively free of solution features (36, 39).

Exploration of karstic damsites is a difficult undertaking. Some unexpected features will almost invariably be encountered only during excavation. The engineering geologist, however, should not be overwhelmed by the number and complexity of the geologic factors influencing karst development and should not abandon the use of geologic models as aid in exploration. Excessive amounts of conventional drilling will not provide a complete picture of the weathering profile and solution features. Interpretations based on information obtained by conventional drilling--even if angle holes and the best sampling techniques are used--tend to minimize the amount and size of residual blocks and fragments contained in the overlying residual soil and to "smooth out" the relief of the bedrock. Calyx holes allow direct observation of the transition zone, if one is present, but they are still limited in **size. Trenches, shafts, and drifts are the**

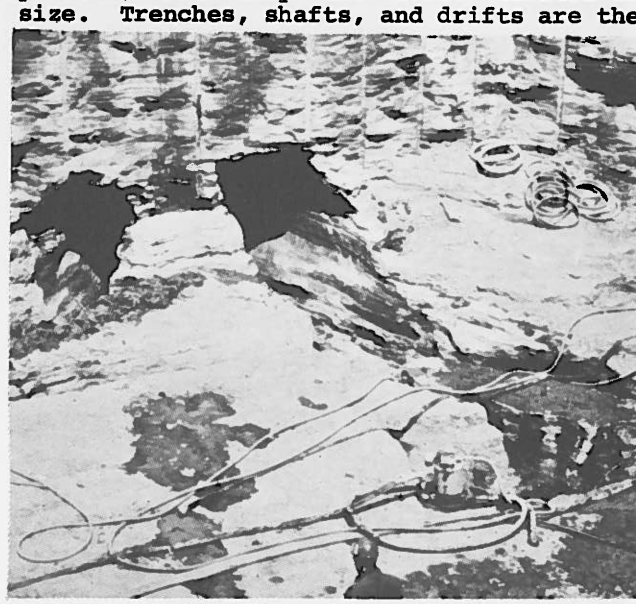

Fig. 9. Differential solutioning controlled by fracturing of underlying formation. Overlying formation is unfractured and unweathered.

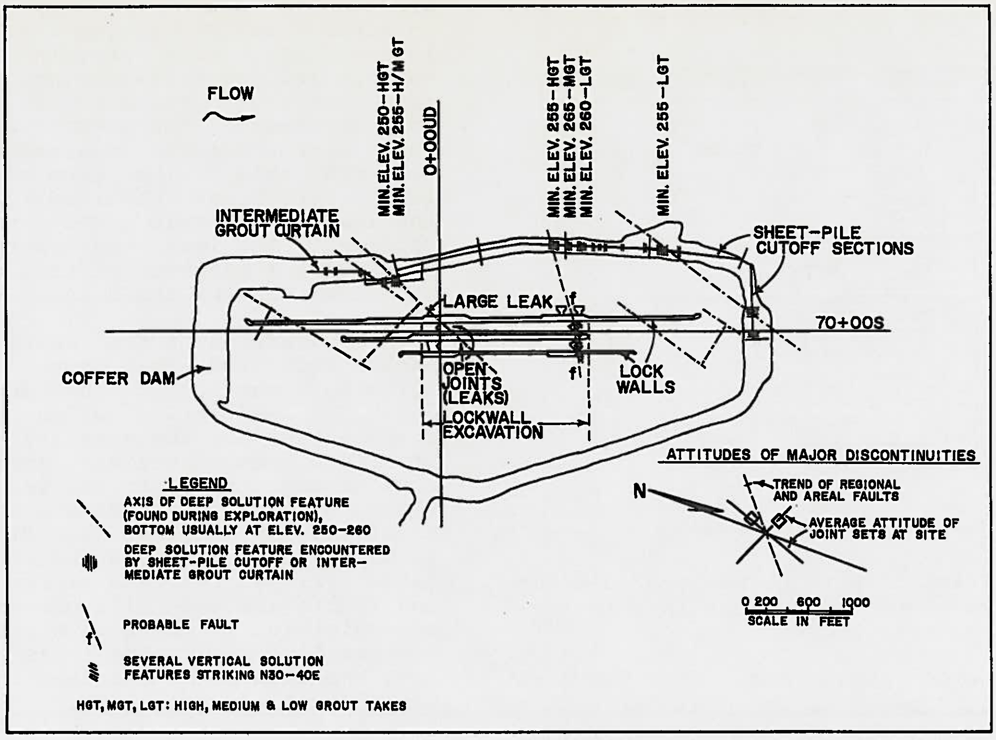

Fig. 10. Plan view of cofferdam, excavation and significant engineering geology features at locks in Ohio River, near Paducah, Kentucky.

most effective means of exploring karstic sites (33) but also the most expensive and sometimes unfeasible. Electrical resistivity appears to be one of the more successful of the geophysical methods commonly used in geotechnical exploration in karstic terrains. The electrical resistivity methods can be used to outline areally some larger features which can then be explored in detail by other means (40, 41). Seismic refraction information, although more accurate than electrical resistivity in predicting the depth to flat soil-rock interfaces, can given erroneous results in the high-relief interfaces typical of karstic terrains. However, improved seismic methods (42) along with holographic techniques (43) may prove to be useful in future exploration programs.

Case History

This example illustrates some of the points made above regarding geologic aspects of damsites in karstic terrains. The case concerns an excavation for two locks along the Ohio River near Paducah, Kentucky. Extensive drilling and sounding revealed that the site was underlain by alluvial materials, and flat-lying Paleozoic limestones (St. Genevieve formation). Between those two materials, however, a well developed residual cover and weathered rock were encountered. The residuum had an average thickness of about 20 ft.; the underlying weathered rock averaged 15 ft. in thickness. "Sound", unweathered bedrock was found in the borings at variable depths; however, for the most part, no major weathering was encountered below elevations 250-260 ft. Boring information indicated that deeper weathering seemed to occur along two sets of elongated, parallel zones with strikes of N15-25°E and N70-75°W. These zones are outlined in Fig. 10 which also shows a plan view of the locks, the earth-dike cofferdam for the excavation, and the extent of the first stage of excavation for the locks. The design of the cofferdam dike included a sheet-pile cutoff wall and a grout curtain to bedrock. The sheet-pile cutoff wall and grout curtain were placed only along part of the dike as shown in Fig. 10. The zones with lower sheet-pile refusal elevations and associated grout takes are also given in Fig. 10. In all cases refusal occurred above elevation 250. Some of these zones of deeper sheet-pile refusal fell in the extensions of the elongated zones found during drilling.

Excavation of the alluvium proceeded without problems. By contrast, excavation of the residuum and weathered rock met with difficulties. Large residual cherty limestone slabs, occurring sometimes 20 ft. above foundation level, could not be handled by conventional earth-moving equipment, and required blasting. Deeper in the section, the weathered rock contained large knobs separated by zones of residuum of varying widths and depths. This combination of materials required the removal of the residuum by clamshells and orange-peel buckets and removal of the knobs by blasting.

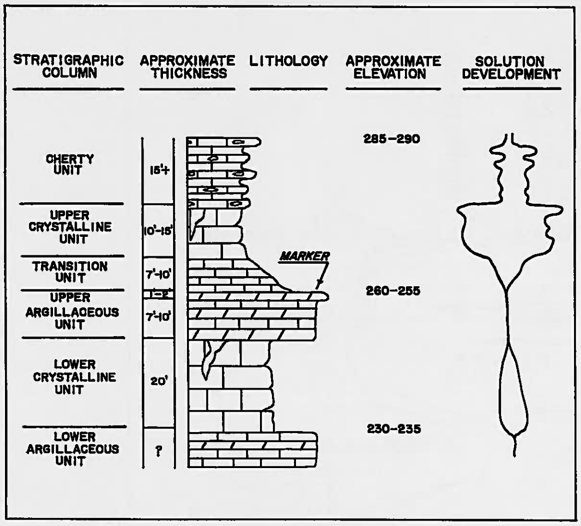

Fig. 11. Stratigraphic column and solution development at locks in Ohio River, near Paducah, Ky.

Foundation levels were, for the most part, as anticipated by exploration, with a couple of exceptions. One of these exceptions occurred near the lower miter sill where a deep solution feature, striking approximately N70°E, required excavation some 30 ft. deeper than anticipated, to elevation 230-235.

During excavation a leak with a flow of about 6000 gpm developed on the river side, at the upstream end of the excavation; the flow occurred at elevation 255. This leak as well as smaller flows at the bottom of the excavation for some monoliths of the middle and land lackwalls were on

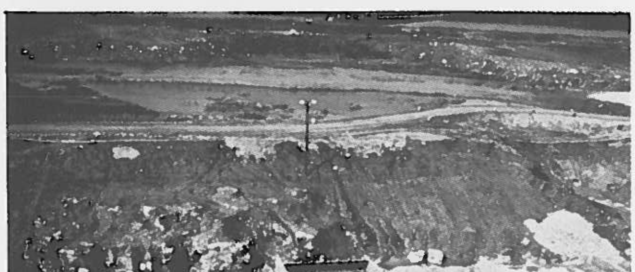

Fig. 12. View of landslide of excavation for locks, near Paducah, Ky. Marker is thin, light colored bed at top of pre-split face of excavation for monolith (A). Material above is a knob of upper crystalline and intermediate units, and residuum. Excavation for monolith (B) is ready for pouring concrete.

the axis of one of the major deep weathered zones disclosed during exploration. An intermediate grout curtain (see Fig. 10) was drilled and fast-setting, polyurethane-type grout was injected in an attempt to stop the leak. The attempt was unsuccessful; when grout was injected at the points near the axis of the deep weathered zone, liquid grout was observed flowing out of the leak area within a short time after injection. The leak was eventually stopped by placing a blanket of sand and gravel on the river side of the dike.

Examination of the walls of the excavation indicated that both structure and lithology controlled the degree and the depth of weathering which in turn were responsible for the excavation and leakage problems. Two vertical joint sets were present and their strikes were the same as those of the elongated deep solution zones disclosed by drilling (see Fig. 10). The very deep solution zone near the downstream miter sill had the same strike as NE-trending faults of the Illinois-Indiana fluorspar district in which this site was located (see Fig. 10). The leaks developed along the most prominent deep solution zone. Lithological controls were just as important as structural discontinuities. Fig. 11 shows a stratigraphic column of the rocks exposed by excavation, their relative susceptibility to solution as observed around the excavation, and their thicknesses and elevations. The topmost unit is a thin- to medium-bedded cherty limestone; the large cherty slabs described earlier were residual elements within this unit. The next unit was a pure, coarsely crystalline, massive limestone which had undergone very extensive solutioning. This unit presented only isolated blocks of residual limestone separated by wide zones of residuum. The next note-worthy unit is the upper argillaceous unit, whose top occurred at elevations 260-265, and which apparently had not been affected by weathering. In fact, the top of this unit was a yellowish argillaceous and dolomitic bed that could be traced as a marker bed around most of the excavation. This layer generally corresponded with the bottom of the deep solution zones discovered by drilling. Fig. 12 is a photograph of the landside of the excavation showing the marker bed and the contrast in weathering above a below this bed. Another pure, coarsely crystalline unit was present beneath the protective cover of the upper argillaceous unit, and extended down to elevations 230-235. The units below the marker bed, and particularly the lower crystalline unit, showed solution cavities only where they intersected probable faults as in the downstream miter sill area. Thus the interaction between structural features and subtle changes in lithology can be used to explain all of the characteristics and problems at this site.

Finally, Fig. 13 shows the results of laboratory solution-rate tests on samples from the marker bed, the upper and lower crystalline units, and the upper argillaceous unit. Also included are the test results of two samples from an argillaceous limestone obtained at a damsite in central Tennessee; this site was found to be notably free of any solution features except for one narrow zone associated with a fault. The tests consisted in placing 5-gram samples in an organic acid solution (0.3 N. chloroacetic acid) in separate containers, and weighing the samples at several time intervals to determine the relative rates of solution. The test results indicate that the crystalline limestones had initial solution rates up to twice those of the samples that displayed localized solution associated with faulting in the field (marker bed, upper argillaceous unit, and samples from Tennessee). The rate of solution appears to decrease faster for the crystalline limestone samples but this decrease is caused by a faster decrease in specific area with time.

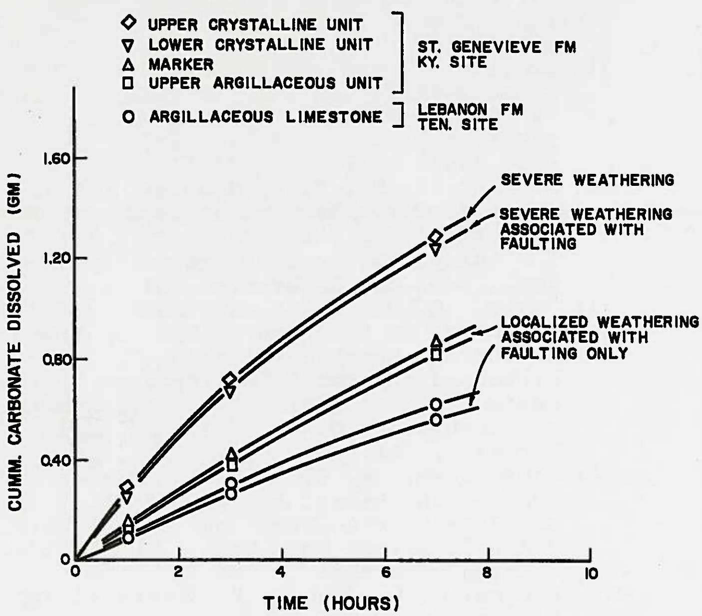

Fig. 13. Laboratory solution-rate tests for limestone samples from sites near Paducah, Ky., and central Tenn.

The content of insolubles by weight was 4% for the crystalline limestones and ranged between 15 and 20% for the argillaceous limestones. The writer does not suggest that such tests could be used to predict karstic features at a particular site. However, he would suggest that results of similar tests and field observations should be stored in data banks and be available to the profession. These data could be used in conjunction with other known geologic controls, such as joint spacing and bedding thickness, to assess the general karstic characteristics of limestone formations at different sites with comparable topographic and climatic conditions (same physiographic region).

CONCLUSIONS

The effectiveness of exploration programs at damsites depends on the engineering geologist's ability to: 1) anticipate, given some known geologic conditions, the nature and position of geologic features that may have a significant influence on the construction and performance of the project, and 2) plan an exploration program that will disclose the presence or prove the absence of such features. To these ends, he requires a clear understanding of the specific geologic processes, factors, or agents that are principally responsible for the individual significant geologic features.

Structural features caused by valley downcarving in flat-lying layered rocks are a demonstrable example of the value of the knowledge of origin of these features. Such knowledge can not only help in planning exploration programs, but at times, can be used to determine design criteria (case history of mylonite under locks).

Exploration of damsites in karstic limestone is often a very complex task. However, some of the complexities can be resolved if an effort is made to understand the interaction of the geologic factors controlling karstic development (case history of locks of limestone).

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SMALL DAMS - PARTICULAR PROBLEMS AND CONSIDERATIONS

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SYNOPSIS. Geotechnical considerations and other criteria for evaluating the feasibility and desirability of small dam projects are described. Interrelationships between the hydrology and hydraulics and the geotechnical aspects are emphasized. Primary spillways and the importance of emergency provisions are discussed.

Introduction

In the past, many more small earth dams than large ones have been built. This ratio of small to large dams will undoubtedly increase in the future, even though the rate of total construction will probably decrease. On the other hand, much more has been written about large than small dams for at least two reasons: (1) the grand scale of a project tends to capture headlines, and (2) on larger dams, more funds are usually allocated to investigations, design, and for instrumentation. However, the complexity of the engineering for dams is not necessarily related to the size of the project. Indeed, the demands for expertise may be greater on small than on large dams.

Although the success of a dam rests heavily on the geotechnical engineer, the overall planning, design and construction of even a small project usually requires the integrated efforts of various disciplines. In some cases, the geotechnical engineer may be the prime professional, but more often than not he is a member of a design team. In any case, the geotechnical engineer is better qualified if he possesses an understanding of the various other important aspects of dam and reservoir projects. He should even have some working knowledge of biology and environmental impacts in this day and age.

The results of a careful investigation and study of the geotechnical and hydrologic conditions for a proposed small dam and lake will be discussed in this paper. Other aspects and criteria for the evaluation of the feasibility, desirability and project cost of the same project will be mentioned. Advantages and disadvantages of two sites less than one mile apart on the same stream will be compared. The project is located on Jubilee Creek near Jubilee College State Park in Peoria County, Illinois. The park is under the jurisdiction of the Illinois State Department of Conservation.

Watershed and Reservoir Characteristics

Fig. 1 shows the two dam sites that were investigated and the extent of the reservoirs associated with each. The drainage area for Dam No. 1 is approximately 36 sq. mi. and for Dam No. 2 about one square mile less.

Approximately 20 to 30 percent of the watershed is moderately to heavily wooded. The remaining 70 to 80 percent is under cultivation with some farmsteads and pasture lands. The general slope of the watershed between the basin divide and the dam locations is between 0.2 and 0.5 percent. The surface soils, which have developed from several feet of loess, are predominantly silts and clays.

Boring and well log data obtained from the Illinois Geological Survey substantiate geological information that the bedrock surface in upland areas upstream of Dam No. 2 exists between elevations 600 and 650. Moreover, geological information indicates that the tributary bedrock valley more or less conforms with the outline of the present valley and the proposed lake. Considering the proposed normal pool levels (575 and 578), this information appears favorable regarding the ability of the basin to hold water. The only problem in this regard appears to be in the abutments of Dam No. 1 discussed later.

Aerial reconnaissance of the lake area detected no sand and gravel pits, although there could be some small abandoned pits which were not detected from the air due to brush overgrowth.

Coal mining activity in the watershed has been insignificant. What mining has taken place was abandoned more than 30 years ago. All past activity consisted of shallow hillside operations above the proposed normal lake level. No evidence of pollution was found.

The surface area of the reservoir impounded by Dam No. 1 is about 595 acres, and the storage volume is 10,670 acre-feet. The maximum water depth is 50 ft, and the average depth is about 18 ft. Over 40 percent of the lake area, the average water depth is 30 ft or greater, which exceeds the Illinois Division of Fisheries criterion of 15 ft minimum average depth (1). The Shoreline Development Index of Lake No. 1 (S_d = length of shoreline divided by the circumference of a circle that would encompass the same lake area) is a favorable 5.6, well above the range of 1.5 to 2.5 for most lakes.

Dam No. 2 would create a lake area of approximately 550 acres and a volume of 7850 acre-feet. Maximum water depth at the dam is 43 ft and the average is approximately 14.3 ft. Over 40 percent of the lake area, the average water depth is 25 ft or greater. The Shoreline Development Index for Lake No. 2 is 4.6.

It is worthwhile noting here that the area of Lake No. 2 is only 8 percent less than No. 1, but the volume is reduced 26 percent. Also, the maximum water depth, the average depth over all the lake, and the average over 40 percent of the area are reduced 14, 21, and 20 percent, respectively. Therefore, even though the reduction in lake area may be small, large changes may occur in other factors used in judging the desirability of the lake. This is not to say, however, that Lake No. 2 does not satisfy all the common criteria for area, volume, depth and shoreline.

Reference to the topography shown on Fig. 1 also indicates that an emergency spillway for Dam No. 1 could probably be located on either abutment; but for Dam No. 2, the feasible location appears to be on the west abutment. More will be said about spillway elevations, lake levels and flood routing in subsequent sections of this paper. These values and calculations, together with the natural topographic and geologic characteristics, establish the height of dam.

Fig. 1 shows 2 existing county bridges across the upper reaches of the proposed lake. It was determined that the elevations of the bridge bearings would be more than 2 ft above the lake level during a 100 yr. flood. Therefore, no adjustments in elevations of the bridges appear to be needed for the proposed heights of the dams. Some riprapping of the approach roadway embankments may be desirable, and this matter should be explored further during final design.

General Geology and Subsurface Conditions

Figs. 2 and 3 indicate the general subsurface profiles along the centerlines of Dams No. 1 and No. 2, respectively, based upon widely spaced preliminary borings and tests.

The upland soil materials are Pleistocene deposits which generally consist of a post-glacial windblown soil underlain by glacial deposits. The glacial materials are Illinoian while the deposits of loess are associated with both the Illinoian and Wisconsinan glaciers. Recent deposits of alluvium overlie the Pleistocene deposits in the valleys.

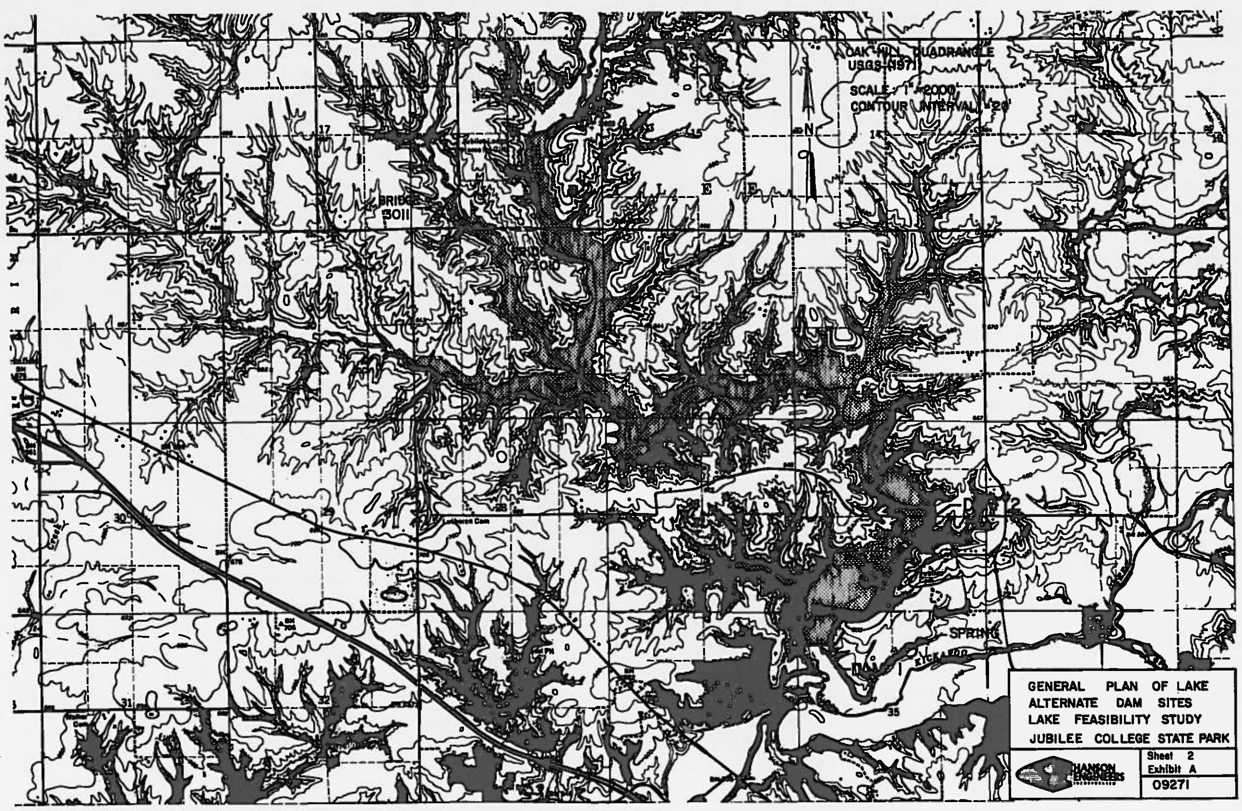


Figure 1. General plan of lake and alternate Dam sites

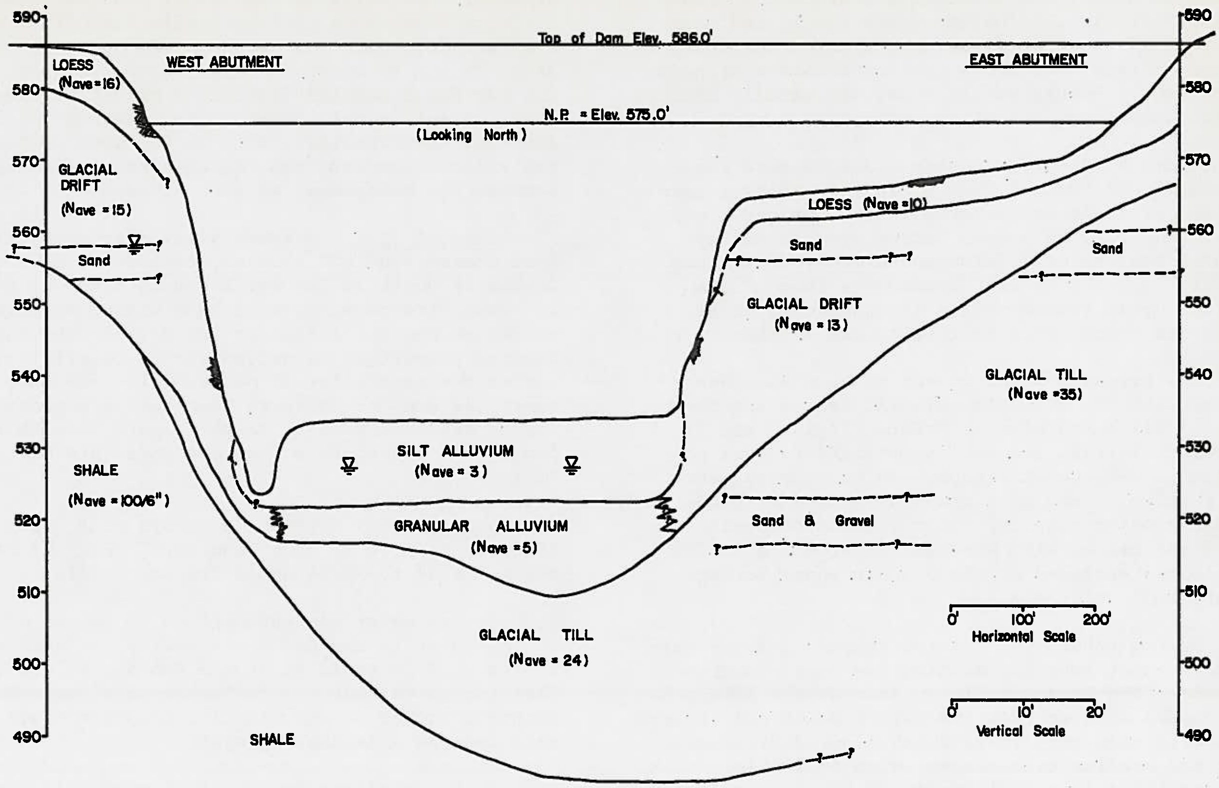


Figure 2. Generalized subsurface profile along centerline of Dam No. 1

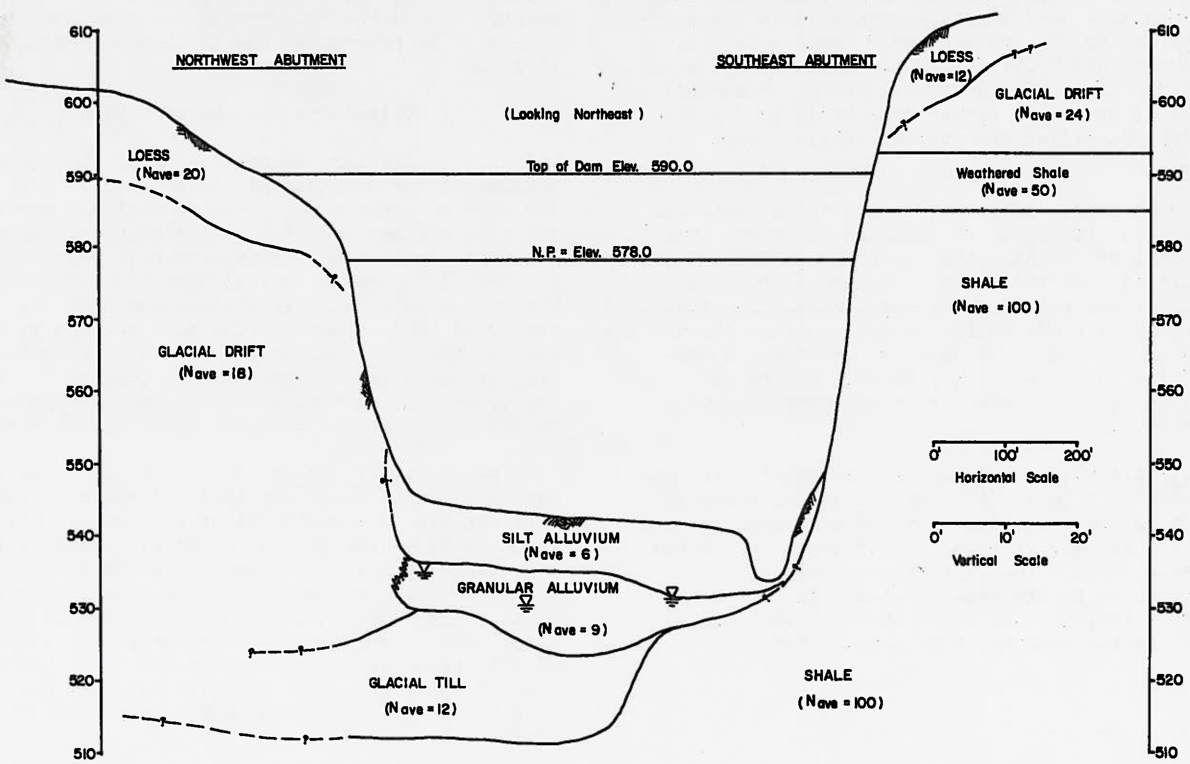


Figure 3. Generalized subsurface profile along centerline of Dam No. 2

Because of high bedrock and relatively thin Illinoian drift, the underlying bedrock topography is reflected in the Jubilee Creek basin, and rock outcrops are present in several areas. The bedrock strata in this area are a part of a series of beds deposited by Pennsylvanian seas, and usually include shale, limestone, sandstone and coal.

Bedrock topographic maps indicate that the Jubilee Creek basin formed a bedrock valley (upper surface of shale encountered in the borings), which was a tributary to a much larger bedrock valley running east to west (Kickapoo Creek valley shown on Fig. 1) prior to the Pleistocene Epoch. The Jubilee Creek stream valley is apparently in at least its third cycle of filling and erosion.

The deposits which appear to have been associated with the Illinoian Glacial Period are the Glacial Tills and Glacial Drifts (Figs. 2 and 3). The Glacial Tills are typically stiff to hard cohesive heterogeneous mixtures of silt, clay, sand and pebbles, containing granular lenses or pockets. Stream erosion occurred after the bedrock valley was first filled with Glacial Tills, and a valley was formed outlined by the present upper surface of the till materials.

During subsequent glacial retreats, large volumes of water from the melting ice were concentrated along the larger valleys, such as the Kickapoo. Overloaded with debris, the waters deposited valley-trains of sand and gravel which blocked drainage from the smaller tributaries, such as Jubilee Creek. The resulting lake environment in the Jubilee Creek basin somewhat explains the layered character of the materials designated as Glacial Drift on Figs. 2 and 3. The Glacial Drift consists mainly of silts and clays with some sand layers encountered in the abutments of Dam No. 1. An erosional cycle recurred, and another valley was formed, outlined by the present upper surface of the Glacial Drift. Apparently, the Glacial Drift was almost completely eroded from what might be called the present Jubilee Creek valley.

The present valley then became filled with the existing deposits of unconsolidated Alluvium. One main source of the Alluvium was probably erosion and transportation of the Wisconsinan Loess deposits which cover the upland areas surrounding the site. The subsurface profiles in the valley areas shown in Figs. 2 and 3 at both dams are similar and consist of four distinguishable strata, described below as they occur from the ground surface downward.

Silt Alluvium. This upper stratum of silts and fine sands is 5 to 10 ft thick. Standard Penetration values and unconfined compressive strengths are fairly low in this stratum, particularly at the Dam No. 1 site. While this material is quite low in shear strength in its natural state, it will consolidate fairly quickly under load and would gain strength as the embankment is constructed.

Granular Alluvium. A coarse sand and gravel stratum, containing varying small smounts of silt and clay, underlies the upper alluvium. This stratum extends to maximum depths of 25 ft at Dam No. 1 (Fig. 2) and to about 20 ft at Dam No. 2 (Fig. 3). The Dam No. 2 samples indicate a higher silt and clay content in this Stratum than the Dam No. 1 samples. Nevertheless, this is the most critical of the valley deposits from the standpoint of seepage beneath the embankment at both sites.

Glacial Till. A sandy silty clay containing some coarse sand and pebbles, extends to maximum depths of 50 ft at Dam No. 1 and to 30 ft at Dam No. 2. This stratum appears to have higher strength values at Dam No. 1 than at Dam No. 2. The engineering properties do not appear to be critical from either the standpoint of permeability (seepage beneath the dam) or strength (foundation support). Cutoff trenches used to block seepage beneath the dam should penetrate a few feet into this Glacial Till.

Shale. Very hard shale occurs at a depth of approximately 50 ft under Dam No. 1 (Fig. 2) and at depths of 12 to 30 ft under Dam No. 2 (Fig. 3).

Ground water was encountered at depths of 6.0 ft to 7.5 ft in the Dam No. 1 valley borings and at depths of 8 ft to 12 ft at the Dam No. 2 location. This indicates that the construction of conventional cutoff trenches in the Granular Alluvium would probably require a dewatering system.

It is important to note that permeable sand layers were found in the abutment borings for Dam No. 1 (Fig. 2). These layers, some 20 ft below normal pool level, represent potential routes for seepage around the embankment. No such sand layers appear to be present in the abutments of Dam No. 2 (Fig. 3).

Preliminary Embankment Design

The preliminary cross section of the proposed embankment for both dam sites is shown in Fig. 4. The type of embankment, internal drainage features, and proposed methods of controlling underseepage are identical for both dams; the only differences are the elevations of normal pool and top of dam. The hydraulic structures are not shown on Fig. 4; they will be discussed in the next section of this paper. Nevertheless, it is important to realize that at least certain preliminary judgements regarding flood routing must have been made before a tentative cross section such as Fig. 4 can be sketched.

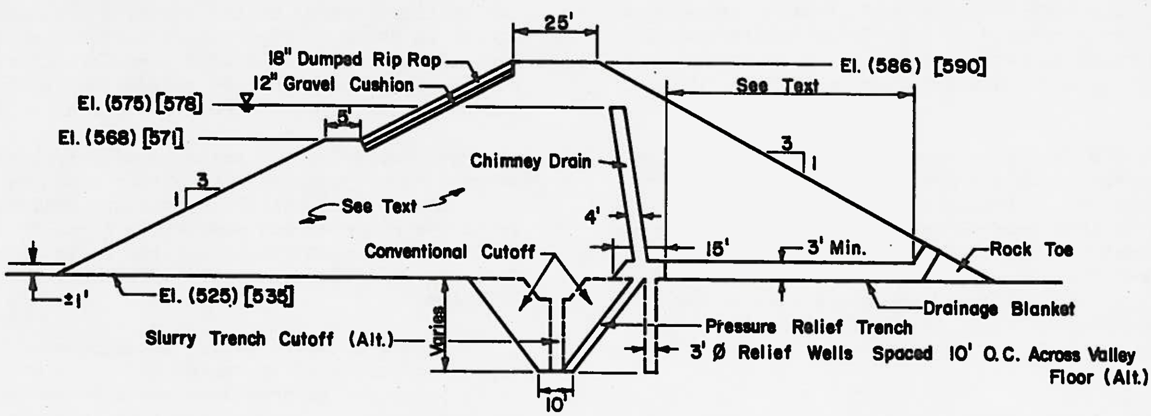
Availability and Use of Material. It is evident that sufficient quantities of suitable embankment material are available at both sites, most of which would be the glacial material excavated from the emergency spillway areas. The clayey soils should be used in those areas requiring the most pliable and impermeable embankment, such as the cutoff trench and base of abutment slope, and otherwise in the upstream portion of the dam. Coarse-grained loess and other granular material can be used but should be placed in the downstream portion of the dam.

Stability. Preliminary stability analyses were performed for Dam No. 1 using a 60 ft high embankment with 3:1 side slopes. In addition to the routine soils tests, one series of consolidated-undrained triaxial tests was performed to evaluate the strength, including the increase due to consolidation of the soft alluvial soils under embankment loads applied rather slowly during construction. The end-of-construction and the long-term design cases were analyzed using circular arc and spreading type failure surfaces. Factors of safety of 1.2 (spreading analysis) and 1.5 (circular arc analysis) were computed for the end-of-construction design case. Factors of safety in excess of 1.5 were computed for the long-term design case.

Although triaxial tests and stability analyses were not performed for Dam No. 2, it was concluded by inspection that the factor of safety would be equal to or greater than those computed for Dam No. 1 (Dam No. 2 embankment height = 50 ft; subsoils are as good or better based on N-values and routine laboratory tests). Based on these preliminary analyses, it was concluded that the alluvial soils beneath embankment areas will not have to be removed and replaced, and that a 50 ft to 60 ft high dam would be stable on 3:1 side slopes at either alternate embankment location.

Settlement. Without benefit of consolidation tests, it is estimated that approximately 6 in. to 1 ft of settlement could occur due to consolidation of valley soils and that another 6 in. to 1 ft of settlement could be experienced from consolidation of the embankment itself. This would result in a total of 1 ft to 2 ft of settlement, of which approximately 60 to 70 percent would probably occur within the construction period.

While the estimated settlement is considered acceptable, its importance as related to the performance of the structure should not be overlooked. Since the abutments at each dam location are either relatively stiff soils or bedrock, which will not settle appreciably, significant differential settlement could occur. Consideration of these factors will require close control of the placement of the embankment. The embankment must be placed at a high enough moisture content so that a sufficiently plastic fill results. Pliability is necessary to minimize the cracking that could occur as embankment strains reflect settlement. These considerations appear to justify the chimney drain as an added safety measure against potential piping damage from settlement cracking of the embankment.



Section at Stream
Point of Max. Cross Section

Legend:
Elevations in parenthesis (575) refers to Dam No. 1
Elevations in brackets [578] refers to Dam No. 2
Dashed lines ---- represents alternate underseepage control

Figure 4. Preliminary typical cross sections of Dams

Underseepage Control. Preliminary estimates of seepage quantities beneath both dams indicate that the seepage from the reservoir through the valley Alluvium beneath the embankment would be tolerable when considered only from a quantitative standpoint. However, when seepage through granular layers that are fed directly from a reservoir through the streambed involves significant (though not necessarily critical) quantities of water, subsurface erosion and pressure relief problems may develop. Considering these factors, measures should be taken to cut off the seepage from the reservoir through the valley Alluvium at either alternate dam site. Two alternate types of cutoffs are indicated on Fig. 4.

An acceptable method of controlling the quantity of underseepage would involve the use of an impermeable clayey cutoff trench extending downward from beneath the embankment into the underlying valley Glacial Till. While such a cutoff trench through the valley Alluvium will effectively block the quantity of seepage, there will still be a need, though less critical, for pressure relief and piping protection.

It is realized that the construction of a clayey cutoff trench may require the use of a well-point dewatering system and stage construction. The final investigation should include backhoe cuts in the valley areas (whichever dam site is finally chosen) to determine the seriousness of dewatering problems associated with a conventional cutoff trench. Based on the preliminary borings, it appears that the site for Dam No. 2 is more favorable since the borings indicated a lower water table at this site and a higher silt and clay content in the alluvial soils.

The clayey cutoff should be carried up the abutments in the form of a key trench which extends 6 ft into a soil abutment or 3 ft into a rock abutment. The extension of the clayey cutoff trench should reach up each abutment to at least the normal pool level, and preferably to the maximum lake level.

Special consideration should be given to the sand layers mentioned previously in the abutments at the Dam No. 1 location. The final investigation should include backhoe cuts in the valley walls and additional borings to determine (1) thickness of impermeable cover, if any, over sand in the abutments and (2) thickness and horizontal extent of sand materials. Also, it should be realized that this could represent a significant cost item for Dam No. 1.

The slurry trench alternate cutoff shown on Fig. 4 would appear to be economically competitive to the conventional cutoff method, especially for Dam No. 1. The conventional slurry trench method of constructing a seepage cutoff consists of excavating a vertical wall trench (usually 3 ft to 5 ft wide) supported by a bentonite slurry, and backfilling with a select impervious material.

As the trench excavation progresses, bentonite slurry is added so that the trench always remains filled. The slurry is a colloidal suspension of sodium bentonite and water. The slurry, with a minimum density of 64.5 pcf, exerts sufficient hydrostatic pressure to support the walls of the trench and to prevent the inflow of ground water.

After a sufficient length of the trench has been excavated, the backfilling can begin. The backfill is first placed with a clamshell bucket to insure that slurry is not trapped in the backfill material. After enough backfill is placed to reach the top of the trench, then the backfill is pushed in with a bulldozer.

The backfill material is generally a well graded clayey gravel to protect against piping and to prevent excessive consolidation. On the Jubilee project, this can probably be accomplished by mixing the excavated Alluvium with select borrow to obtain the correct gradation.

Water Pressure Control. The construction of the cutoff trench will effectively reduce the quantity of underseepage. However, during and after construction, the relief of pore water pressures in the valley is of prime importance. This relief of pore water pressure is necessary if the underlying soils are to develop their maximum shear strength in the prevention of base failures.

A second function of a pressure relief system is to relieve the uplift pressures which tend to develop beneath the toe of the embankment as reflections of the water level in the reservoir. If such pressures are unrelieved, boils or quick conditions could result in serious piping problems.

Positive relief of potential pore water pressures downstream of the conventional cutoff trench can be accomplished by the use of a pressure relief trench as shown on Fig. 4. This trench should consist of a clean, well graded granular material and should extend continuously across the valley and up the abutments to normal pool elevation.

Another method of pressure relief which could be used in conjunction with either a slurry trench or a conventional cutoff is pressure relief wells. Tentative diameter and spacing of pressure relief wells are shown on Fig. 4. These wells should penetrate at least 5 ft into the underlying Granular Alluvium.

Whether pressure relief downstream of the cutoff be accomplished by trench or by the well method, it is important to provide a positive connection to the horizontal drainage blanket as indicated in Fig. 4.

Internal Drainage. The internal drainage system shown in Fig. 4 consists of a chimney drain, a drainage blanket and a rock toe.

The purpose of the chimney drain is to intercept horizontal seepage through the embankment caused by either the layering effect of constructing the embankment in horizontal lifts or by cracks in the embankment induced by differential settlement. The interception of this seepage will guard against both a piping failure and the saturation of the downstream slope, which might possibly result in instability. Ideally, the chimney drain material should be a fairly clean granular material which is compatible in terms of filter requirements with the surrounding cohesive embankment materials. The chimney drain should extend continuously between the normal pool elevations on the abutments.

The primary function of the drainage blanket is to discharge water intercepted by the chimney drain and the pressure relief system. The portion of the drainage blanket between the 15 ft wide strip under the chimney drain and the downstream toe (see Fig. 4) can be constructed in intermittent strips running transverse to the centerline of the dam. The minimum width of the granular strips should be 25 ft, and the maximum width of separation should be 50 ft, (one-third coverage when viewed in plan between about elevation 540 contours for Dam No. 1 and elevation 550 contours for Dam No. 2). The interior 15 ft wide strip should be continuous across the valley between normal pool elevation contours at the abutments.

As mentioned above, the primary function of the drainage blanket for this design is to discharge water. Final design estimates of seepage quantities may indicate that a somewhat modified approach could be used. Instead of using the conventional drainage "blanket," it may be possible to use a series of water "conductors" which emanate from the 15 ft wide granular strip (into which the chimney drain and relief system discharge) and discharge into the rock toe. These "conductors" might be 5 or 10 ft wide and would consist of a 2 ft layer of pea gravel sandwiched between 1 ft layers of concrete sand. Depending on the number of "conductors" required across the valley, a significant savings in granular material might be realized. Moreover, this type of design would appear to be equally functional.

Slope Protection. The upstream slope should be protected against wave erosion and ice damage with an 18 in. layer of dumped stone riprap underlain by a 12 in. layer of finer granular material. This protection should extend from the top of the dam down to the tentative elevations shown on Fig. 4.

Hydrologic and Hydraulic Considerations

No geotechnical engineer should expect to be the prime project manager of even a small dam project without knowledge and experience in the water resources area of civil engineering practice.

The general characteristics of the watersheds, surface soils and other factors involved in the hydrological studies for the Jubilee Project have been covered in previous sections of this paper. The purpose of this section is to discuss in a general manner the preliminary hydrologic and hydraulic considerations. Some of this work was accomplished before any geotechnical investigations were undertaken. Indeed, at least a rough watershed yield computation must be made before any dam project can be deemed worth even the minimum of further study.

Yield Analyses. Yield analyses for a 25 year frequency drought based on the method presented in Bulletin 51, Illinois State Water Survey, were made for both dams. These analyses indicated that reduced reservoir capacity due to sedimentation would be negligible. Seepage loss beneath the dams will not be significant, particularly with the cutoff measures recommended previously. Finally, taking into account evaporation, the yield analyses indicate that a sufficient quantity of water will be available for a recreation lake.

Calculations were made of the mean annual runoff and filling times for the reservoirs impounded by both dams. The estimated filling periods of 6 months or less are relatively short when compared with many conservation lakes, due primarily to the ratios of drainage area to lake volume.

Flood Estimates, Routing, Spillway Design. Practically no flow records were available for Jubilee Creek. A one hour unit hydrograph was developed from a two hour hydrograph previously developed by the Illinois Division of Waterways in a pre-preliminary study. Fifty year flood hydrographs were then developed for 6 and 8 hr storm durations using this unit hydrograph and also a synthetic unit hydrograph computed from Mitchell's Standard Form 80.10. Peak discharges for 50 and 100 year floods were computed from five other empirical methods. These calculated discharges varied from 23 percent below the mean discharge to 27 percent above.

After due consideration of the discharge values and the associated assumptions of hydrologic parameters, it was decided to base the preliminary design of the primary spillway on an 8 hr - 100 year flood, computed according to Mitchell's Standard Form 80.10. Accordingly, Fig. 5 shows (for both dams) the inflow hydrograph, spillway discharge and outflow curves, and the "live storage" curve.

Several quantities should be noted on Fig. 5. The peak stage is the same 4.7 ft for both spillways, even though the peak inflow for Dam No. 1 is greater than Dam No. 2 and the spillway discharge for Dam No. 1 is less than Dam No. 2. These values are all interrelated to reservoir storage and crest length of the spillways. That is to say, the reservoir storage of Dam No. 1 is enough greater so that a smaller spillway will handle a larger inflow at the same stage.

On Fig. 4, discussed previously, the freeboard from normal pool to top of dam was 11 ft for Dam No. 1 and 12 ft for Dam No. 2. Since the primary spillway capacities have been adjusted so that design flood stages are equal, it is apparent that the top of Dam No. 2 would also be 1 ft higher above the 100 year flood level (7.3 ft compared to 6.3 ft). This difference in flood freeboard is reasonable in view of the relative storage capacities and is consistent with emergency safety considerations (discussed later in this paper) for floods having a greater recurrence interval.

Type of Primary Spillway. Various types of primary spillways are used for small dams. The most economical type of spillway for both dams in the Jubilee study appears to be a 3-sided weir on a riser structure of reinforced concrete connected to either a prestressed concrete pipe or a reinforced concrete conduit which passes through the bottom of the dam. The conduit would terminate in a reinforced concrete divergent chute and stilling basin. A typical spillway of this type is shown in Fig. 6 which also shows a failure of a small dam. The probable causes of the failure shown in the figure have been discussed in a previous paper (2).

Preliminary hydraulic computations indicate that the crest length for Dam No. 1 would be about 42 ft and for Dam No. 2 approximately 48 ft, thus accounting for the larger outflow discharge previously mentioned for Dam No. 2. The computations also indicate that if pipe conduits are used, they would be in the range of 8 to 12 ft in diameter, depending upon the method of aeration.

Much more than space in this paper will permit could be said about the theory and design of this type of spillway. The hydraulics are not simple. The design should avoid siphonic action and cavitation by providing for ample aeration. This can be accomplished either by selecting conduits large enough (with respect to the control of capacity in the riser) so they do not flow full or by providing air vents to insure an adequate supply of air in the conduit (3).

The riser-conduit type of spillway presents some geotechnical concerns. Anti-seep collars must be provided to reduce the seepage and piping potential adjacent to the conduit. In addition, special attention must be given to water content and compaction of the material surrounding the conduit. Moreover, at the location of the conduit, adequate transitional continuity must be provided in the internal pressure relief, filter, and drainage provisions of the embankment.

Emergency Spillway Design. Crest elevations of emergency spillways are usually set at the flood stage used for the design of the primary spillway. Thus, floods in excess of 100 year frequency (at the Jubilee sites) would produce outflows from both spillways. The proportionate sharing of the flood routing between the two spillways during extreme floods depends on the dimensions of the emergency facility as well as the type of primary spillway. It is preferable from the standpoint of safety to locate an emergency spillway a considerable distance from the dam itself.

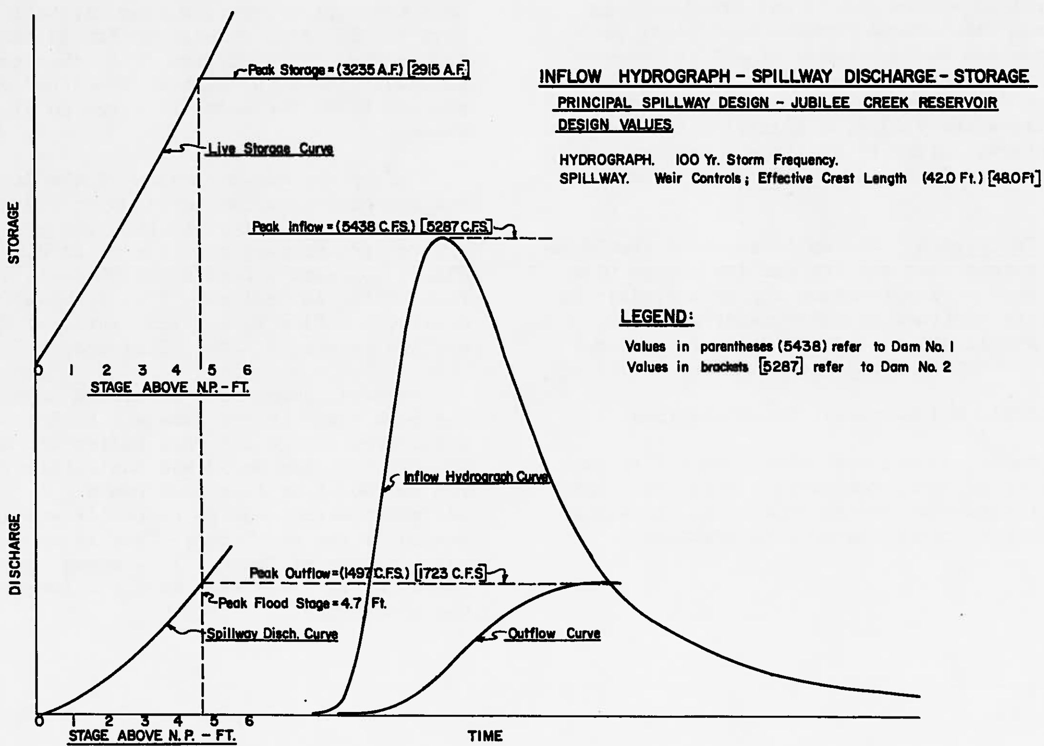


Figure 5. Inflow hydrograph, spillway discharge, storage

For the Jubilee dams, emergency spillway widths and top-of-dam elevations were obtained using routing procedures similar to those developed by the Soil Conservation Service (4). The freeboard hydrograph analysis establishes the minimum elevation of the top of the dam. The emergency spillway hydrograph analysis is used to establish the minimum dimensions of the emergency spillway based on exit velocities determined by erosion considerations.

The freeboard hydrograph (probable maximum storm) was routed through both reservoirs. The probable maximum six hour precipitation for this area is 25.5 in. This value was reduced according to (1) the size of the watershed because maximum probable rainfall will not occur uniformly over a large watershed, and (2) the hazard potential if failure should occur. A reduced net design value of 13.6 in. was used in the routing procedure. For Dam No. 1, a top-of-dam elevation of 586 was obtained and for Dam No. 2, an elevation of 590 was indicated by the calculations.

The emergency spillway hydrograph analyses (according to SCS procedures) indicated that crest widths on the order of 500 and 550 ft would be satisfactory. In final design, and perhaps even during construction, these dimensions may be increased in accordance with the suitability and need for the material excavated from the emergency spillway area.

Comments on Final Design and Construction

It is apparent that additional geotechnical investigations and engineering design in other areas must be accomplished during final design and preparation of contract documents for the Jubilee project. The actual scope of additional work cannot be given here because space has not permitted the inclusion of all data and analysis from the actual preliminary report on the two sites.

Actually, the design of even small dams is not completed until the dam is built and the reservoir is in successful operation. Indeed, changes during construction are almost always necessary to obtain the best end results. Greater skill, experience and judgement may be required of the engineer in charge of construction than of the designer (5). Most importantly, the designers and constructors should never organize into separate camps with antagonistic attitudes. Such separation sometimes results in complete failure of the project.

Conclusions

In the authors' judgement, both of the dam sites studied in this report are technically feasible. However, several factors point to the site of Dam No. 2 as the more favorable of the two sites:

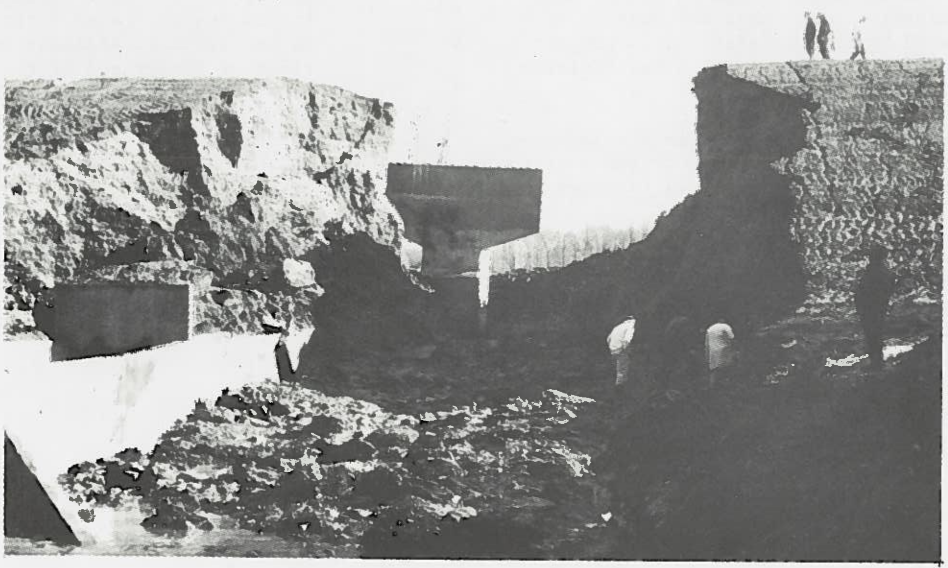


Figure 6. Typical riser-conduit spillway exposed in breach of small dam

1. Permeable layers of alluvium in the valley should be cut off by either conventional or slurry trench methods. The cutoff required for Dam No. 2 would be shallower and shorter. It also appears that dewatering problems associated with a conventional cutoff would be significantly less at the Dam No. 2 site.

2. Sand and gravel layers were encountered in the borings made in the abutment areas of Dam No. 1. No such layers were encountered in the abutments of Dam No. 2. The treatment of the abutments to prevent seepage and piping could be a significant cost item (over and above our estimate) if Dam No. 1 is selected.

3. The location of Dam No. 2 appears favorable for a possible extension of the existing road from the northwest over the dam and into the main park area.

4. As is evident from the estimates of cost ($1,982,000 for Dam No. 1 and $1,475,000 for Dam No. 2), the cost per acre is less for the lake impounded by Dam No. 2.

Lakes impounded by a dam at either site would appear favorable in terms of the Illinois Division of Fisheries criteria, except for the size of the lake. However, the lakes impounded by either dam would be only slightly larger than the maximum of 500 acres suggested by the Division. This project appears satisfactory in terms of all other Division criteria:

1. The maximum depths at the dam are less than 50 ft, and the average depths over 40 percent of the areas are well in excess of 15 ft.

2. The shorelines are quite irregular with high S_d values.

3. The shoreline to bottom slopes are generally favorable with a minimum amount of shoal areas.

4. The slopes of the uplands above the lake surfaces are sufficiently flat in most areas to allow safe access to the shore.

5. Watershed to pool area ratios are significantly greater than 15:1.

6. Reduced reservoir capacity due to siltation will be insignificant, and pollution is not expected to be a problem.

The above conclusions pertain to the Jubilee project. There are other conclusions pertinent to this paper and to the scope of the Seminar for which the paper has been prepared:

1. The investigation, planning, design and construction of small dams is interesting work which often presents many challenging engineering problems.

2. The interrelationships which exist between the various components of dam design must be understood. Therefore, the design of even small dams must be a team operation with each member of the team cognizant and tolerant of the special concerns of other members.

3. However, the geotechnical engineer can take solace in the fact that the states of art in the other disciplines involved in dam design are no further advanced than in geotechnical engineering. The limitations of theory and of the scientific method are present also. For example, there are just as many procedures for calculating flood magnitudes as for computing the factor of safety of slopes. Moreover, reasonable assumptions of hydrologic and hydraulic parameters are even more difficult to make than assumptions of the shearing strength of soils.

Acknowledgments

A considerable portion of this paper is based on a preliminary feasibility report entitled Proposed Dam and Lake, Jubilee College State Park, Peoria County, Illinois, prepared by Hanson Engineers Incorporated for the Illinois Capital Development Board and the Illinois Department of Conservation, dated December 22, 1976. The assistance and cooperation of the following individuals are gratefully acknowledged:

Mr. Carl Thunman, Sr., Chief Engineer Emeritus, Illinois Department of Conservation.

Mr. Edward Donohue, Chief Engineer, Illinois Department of Conservation.

Mr. Carl Anderson, Project Engineer, Illinois Department of Conservation.

Mr. Marc Hillier, Project Manager, Illinois Capital Development Board.

Mr. John M. Healy, Vice President, Hanson Engineers, Inc.

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3. Design of Small Dams. Bureau of Reclamation, U. S. Department of Interior. Second Edition, Revised Printing, 1974.

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FOUNDATION SEEPAGE PROBLEMS AT WOLF CREEK DAM

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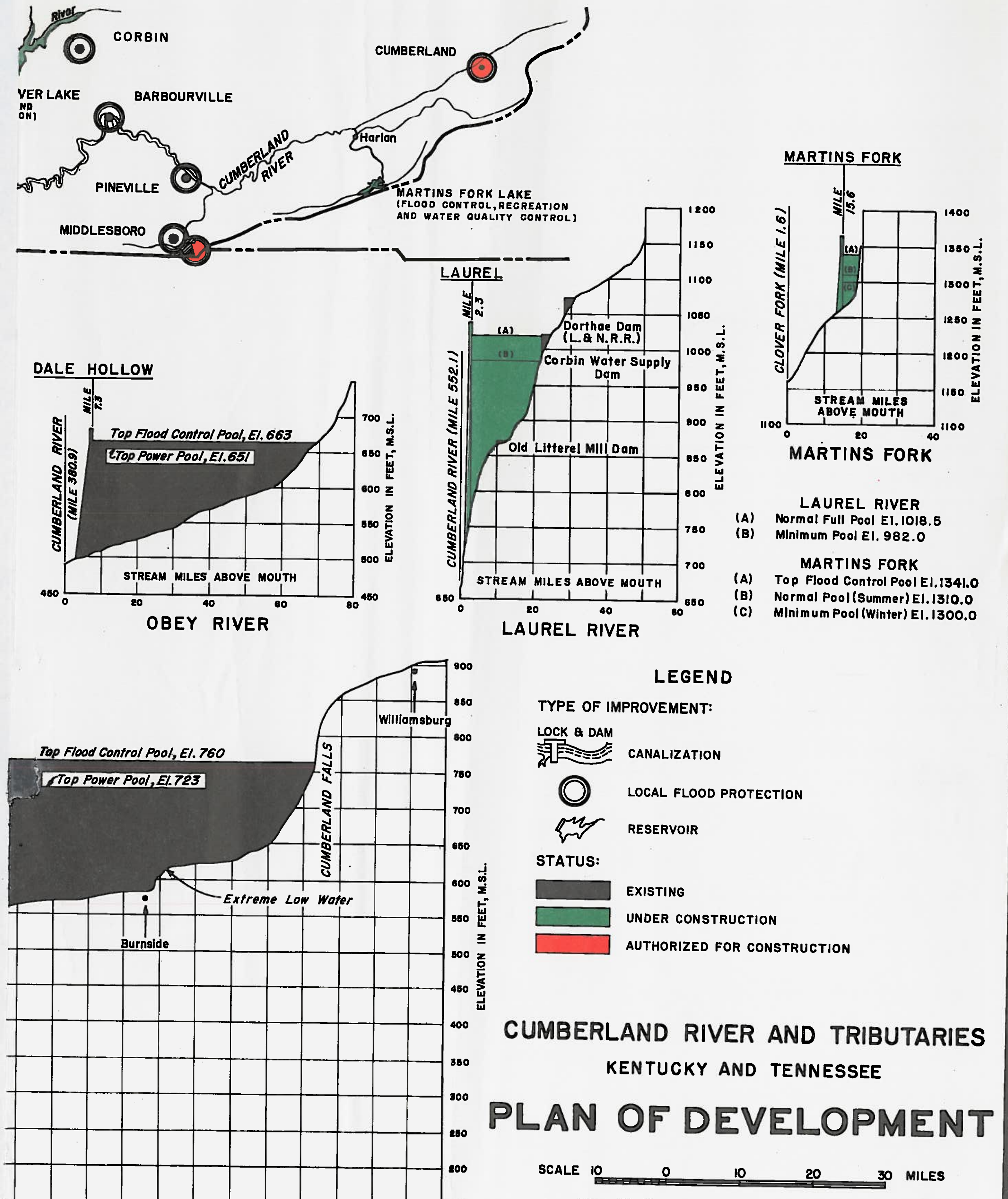
Nashville District, U. S. Corps of Engineers

ABSTRACT. Located at the head of the Cumberland River, Wolf Creek Dam was designed prior to World War II, but not completed until 1951. In 1968, muddy flows in the river and sinkholes in the toe of the dam were found to be linked to the reservoir by a solution system in the limestone foundation rock. The pre-war design of the dam was not adequate to prevent this long term piping development. Although emergency grouting probably averted a major failure at the time, with the aid of outside consultants, the Corps decided to build a concrete diaphragm wall in the dam as a permanent protection against future seepage. As the twice successful bidder on the two separate contracts for the project, ICOS Corporation of America is responsible for the installation of the wall. Now well underway, the ICOS wall consists of a line of interlocking concrete elements. A typical wall section consists of two primary elements (26 inch diameter concrete-filled steel casings) joined by a bi-concave secondary element. Upon completion, the continuous cutoff wall will have a cost of just under $96.5 million.

General Background and Description of Project

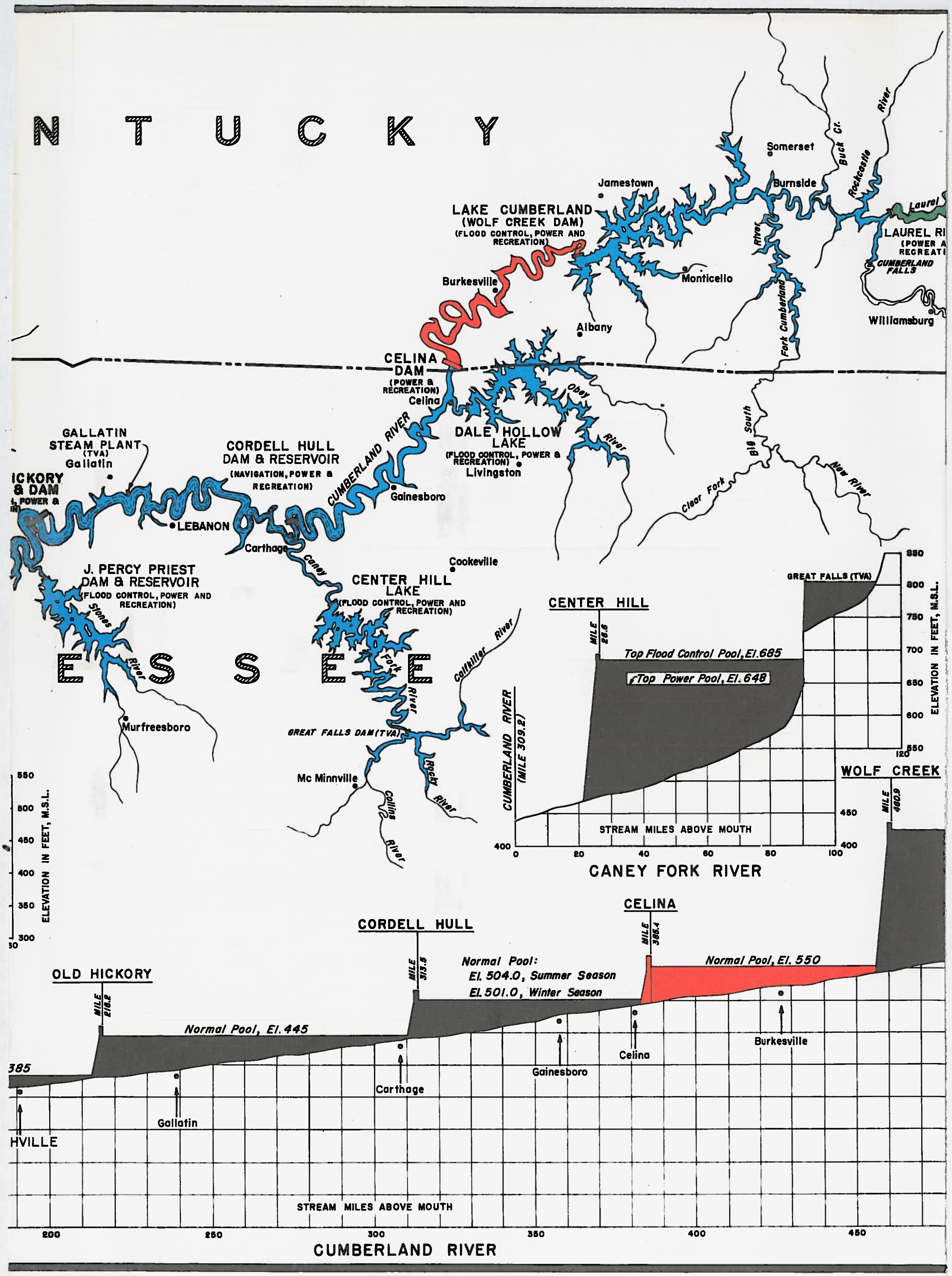
Wolf Creek Dam is a composite earth and concrete dam located in south central Kentucky at the head of the Cumberland River. As shown on Figure 1, the Cumberland River traverses much of middle Tennessee and parts of Kentucky before entering the Ohio River at Smithland, Kentucky. The 258 foot high dam impounds up to six million acre-feet of water at maximum flood storage. The six unit powerhouse at Wolf Creek generates 270 megawatts of hydroelectric power, and over six and a quarter million visitors enjoy the project's recreational facilities each year.

The general arrangement of the project is shown on the photograph in Figure 2. The main feature under consideration here is the 3940 foot long earth embankment that extends across the flood plain to connect the concrete section of the dam to the right abutment. This embankment is a homogenous rolled earth-fill structure constructed chiefly of clays, sandy clays, and clayey sands from the valley alluvium upstream and downstream of the damsite. The embankment, except for a small portion adjacent to the concrete structure, is underlain by up to 50 feet of alluvium and terrace materials which are, in turn, underlain by karstic Ordovician limestones. As seen in the dam cross section in Figure 3, the pre-World War II design and con-



River
CORBIN
CUMBERLAND
VER LAKE
ND
ON)
BARBOURVILLE
Harlan
CUMBERLAND RIVER
PINEVILLE
MARTINS FORK LAKE
(FLOOD CONTROL, RECREATION AND WATER QUALITY CONTROL)
MIDDLESBORO
DALE HOLLOW
CUMBERLAND RIVER (MILE 380.9)
MILE 7.3
Top Flood Control Pool, El. 663
Top Power Pool, El. 651
STREAM MILES ABOVE MOUTH
ELEVATION IN FEET, M.S.L.
OBEY RIVER
LAUREL
MILE 2.3
CUMBERLAND RIVER (MILE 552.1)
(A)
(B)
Dorthae Dam (L.& N.R.R.)
Corbin Water Supply Dam
Old Litterel Mill Dam
STREAM MILES ABOVE MOUTH
ELEVATION IN FEET, M.S.L.
LAUREL RIVER
MARTINS FORK
CLOVER FORK (MILE 1.6)
MILE 15.6
STREAM MILES ABOVE MOUTH
ELEVATION IN FEET, M.S.L.
MARTINS FORK
LAUREL RIVER
(A) Normal Full Pool El. 1018.5
(B) Minimum Pool El. 982.0
MARTINS FORK
(A) Top Flood Control Pool El. 1341.0
(B) Normal Pool (Summer) El. 1310.0
(C) Minimum Pool (Winter) El. 1300.0
Top Flood Control Pool, El. 760
Top Power Pool, El. 723
Williamsburg
CUMBERLAND FALLS
Extreme Low Water
Burnside
ELEVATION IN FEET, M.S.L.
LEGEND
TYPE OF IMPROVEMENT:
LOCK & DAM
CANALIZATION
LOCAL FLOOD PROTECTION
RESERVOIR
STATUS:
EXISTING
UNDER CONSTRUCTION
AUTHORIZED FOR CONSTRUCTION
CUMBERLAND RIVER AND TRIBUTARIES
KENTUCKY AND TENNESSEE
PLAN OF DEVELOPMENT
SCALE 10 0 10 20 30 MILES
U.S. ARMY ENGINEER DISTRICT, NASHVILLE, TENNESSEE
JANUARY 1974
DWG. NO. QO - 5/5.13

FIGURE 1



NTUCKY
ESSEE
LAKE CUMBERLAND
(WOLF CREEK DAM)
(FLOOD CONTROL, POWER AND RECREATION)
Jamestown
Somerset
Burnside
Buck Cr.
Rockcastle
River
Laurel
LAUREL RI
(POWER A
RECREATI
CUMBERLAND FALLS
Williamsburg
Monticello
Burkesville
Albany
Fork Cumberland
River
CELINA DAM
(POWER & RECREATION)
Celina
DALE HOLLOW LAKE
(FLOOD CONTROL, POWER & RECREATION)
Livingston
Obey
River
Big South
Clear Fork
New River
GALLATIN STEAM PLANT
(TVA)
Gallatin
ICKORY
& DAM
, POWER &
CORDELL HULL DAM & RESERVOIR
(NAVIGATION, POWER & RECREATION)
CUMBERLAND RIVER
Gainesboro
LEBANON
Carthage
Caney
Cookeville
J. PERCY PRIEST DAM & RESERVOIR
(FLOOD CONTROL, POWER AND RECREATION)
Stones
River
Murfreesboro
CENTER HILL LAKE
(FLOOD CONTROL, POWER AND RECREATION)
Fork
River
Calfkiller
River
GREAT FALLS DAM (TVA)
Rocky
River
Mc Minnville
Collins
River
CENTER HILL
MILE 26.6
Top Flood Control Pool, El. 685
Top Power Pool, El. 648
GREAT FALLS (TVA)
CUMBERLAND RIVER (MILE 309.2)
STREAM MILES ABOVE MOUTH
ELEVATION IN FEET, M.S.L.
400
450
550
600
650
700
750
800
850
0
20
40
60
80
100
120
CANEY FORK RIVER
WOLF CREEK
MILE 460.9
CELINA
MILE 385.4
Normal Pool, El. 550
CORDELL HULL
MILE 313.5
Normal Pool:
El. 504.0, Summer Season
El. 501.0, Winter Season
OLD HICKORY
MILE 216.2
Normal Pool, El. 445
385
ELEVATION IN FEET, M.S.L.
300
350
400
450
500
550
HVILLE
Gallatin
Carthage
Gainesboro
Celina
Burkesville
STREAM MILES ABOVE MOUTH
200
250
300
350
400
450
CUMBERLAND RIVER

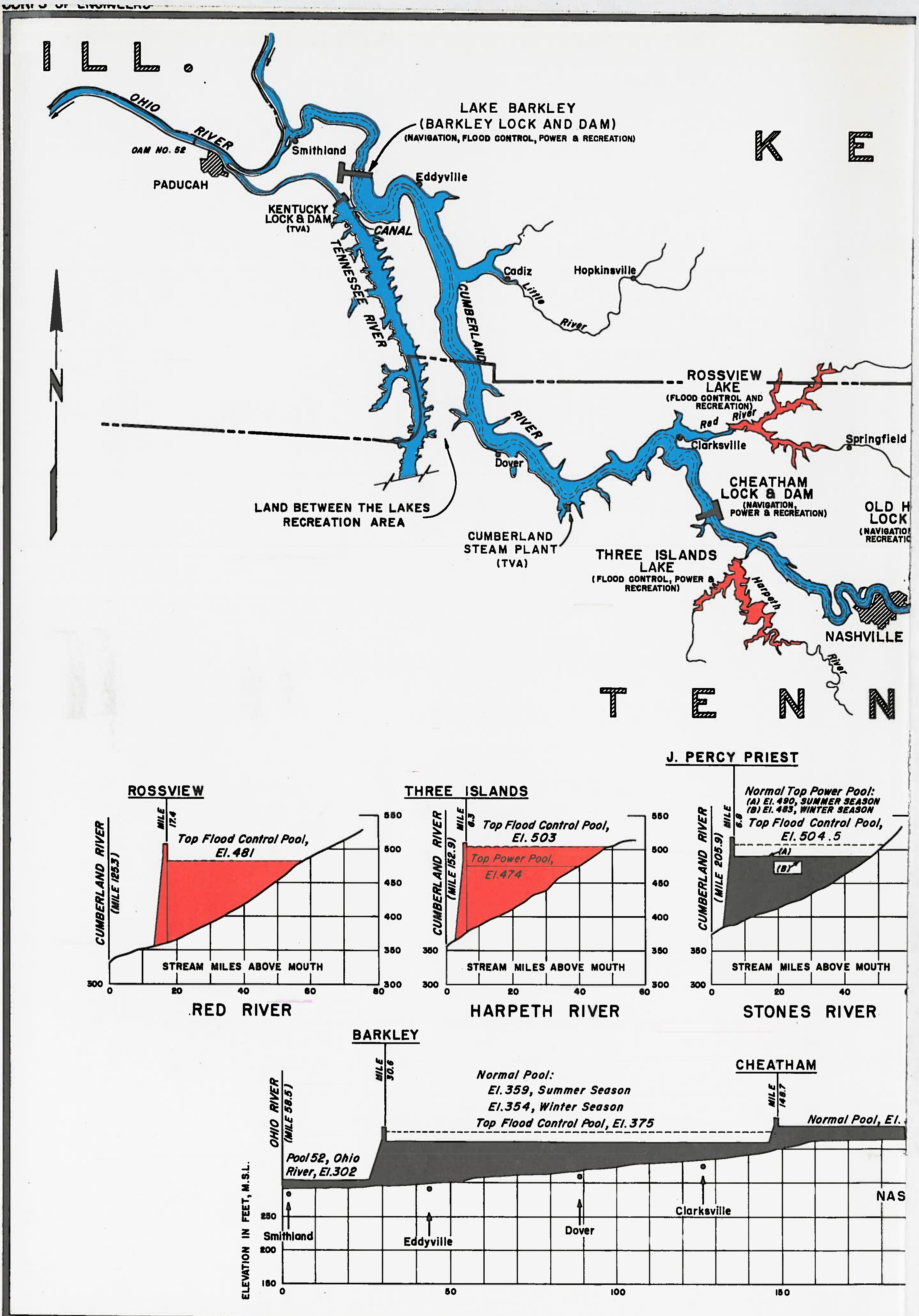

ILL.
OHIO RIVER
DAM NO. 52
PADUCAH
Smithland
LAKE BARKLEY
(BARKLEY LOCK AND DAM)
(NAVIGATION, FLOOD CONTROL, POWER & RECREATION)
K E
Eddyville
KENTUCKY LOCK & DAM (TVA)
CANAL
TENNESSEE RIVER
Cadiz
Hopkinsville
Little River
CUMBERLAND RIVER
N
ROSSVIEW LAKE
(FLOOD CONTROL AND RECREATION)
Red River
Springfield
Clarksville
Dover
CHEATHAM LOCK & DAM
(NAVIGATION, POWER & RECREATION)
LAND BETWEEN THE LAKES RECREATION AREA
CUMBERLAND STEAM PLANT (TVA)
THREE ISLANDS LAKE
(FLOOD CONTROL, POWER & RECREATION)
Harpeth River
NASHVILLE
T E N N
ROSSVIEW
CUMBERLAND RIVER (MILE 125.3)
MILE 17.4
Top Flood Control Pool, El. 481
STREAM MILES ABOVE MOUTH
RED RIVER
THREE ISLANDS
CUMBERLAND RIVER (MILE 152.9)
MILE 6.3
Top Flood Control Pool, El. 503
Top Power Pool, El. 474
STREAM MILES ABOVE MOUTH
HARPETH RIVER
J. PERCY PRIEST
CUMBERLAND RIVER (MILE 205.9)
MILE 6.8
Normal Top Power Pool:
(A) El. 490, SUMMER SEASON
(B) El. 483, WINTER SEASON
Top Flood Control Pool, El. 504.5
(A)
(B)
STREAM MILES ABOVE MOUTH
STONES RIVER
BARKLEY
OHIO RIVER (MILE 58.5)
MILE 30.6
Normal Pool:
El. 359, Summer Season
El. 354, Winter Season
Top Flood Control Pool, El. 375
CHEATHAM
MILE 148.7
Normal Pool, El.
Pool 52, Ohio River, El. 302
Smithland
Eddyville
Dover
Clarksville
NAS
ELEVATION IN FEET, M.S.L.

FIGURE 2 - GENERAL ARRANGEMENT OF PROJECT.

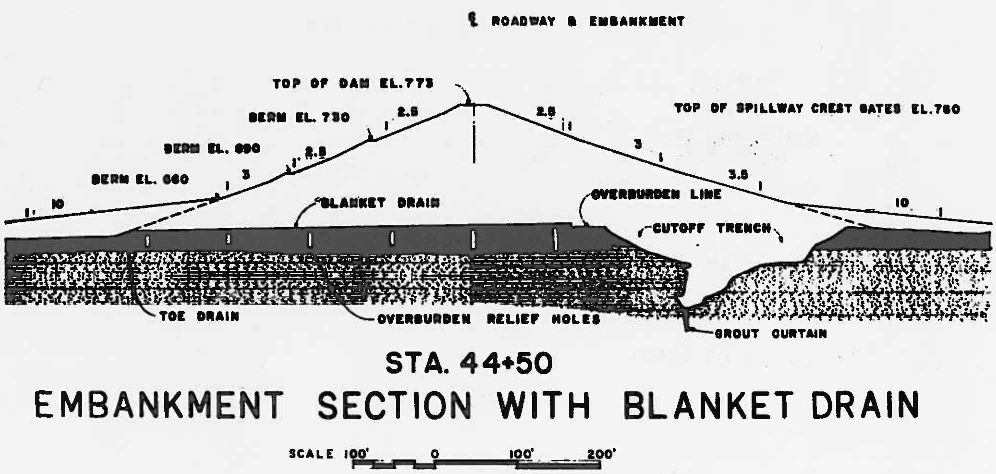


FIGURE 3. EMBANKMENT SECTION.

struction of the dam provided a clay-filled cutoff trench through much of the karstic limestones; however, this construction proved to be inadequate in preventing foundation piping of the filling of the karstic features upstream and downstream of the cutoff. In late 1967 and early 1968, after 17 years of apparently satisfacory operation, signs of distress appeared in the form of muddy flows in the tailrace just downstream of the powerhouse and sinkholes in the downstream toe of the dam. Emergency grouting probably averted a failure of the dam, and in 1972, a decision was made to provide a permanent positive cutoff by means of a concrete diaphragm wall that is currently under construction.

Geology

Wolf Creek is located in the Cumberland Plateau Province just west of the structurally affected Valley and Ridge Province. The site is underlain by Ordovician limestones designated the Leipers and the Catheys. As shown in the generalized geologic section of Figure 4, younger rocks outcrop on the abutments. Cycles of river downcutting and subsequent deposition during the Pleistocene and recent times have resulted in a system of karstic developments which have played an important role in the problems experienced at Wolf Creek. The karsts, which are more prominent toward the present location of the river at the site, include horizontal and near-vertical solution channels along joint systems as well as well-developed partially-filled cave systems with a vertical extent of 40 feet or more. Figure 5 shows a profile along the axis of the earth embankment and some of the karstic features encountered in explorations. Kellberg & Simmons (1) examined the geology of the site in considerable detail and show examples of the features uncovered in the excavation for the cutoff trench.

Design and Construction

The dam was designed just prior to World War II with construction beginning before the war and ending in 1951 after a termination of work between 1943 and 1946. Most of the foundation work was accomplished prior to the war. The explorations encountered several of the karst features and a design was provided which, at that time, was considered adequate to cutoff all seepage through the foundation of the dam. Figure 3 shows a typical cross section of the homogenous rolled-clay dam with cutoff trench into rock near the upstream toe. A single grout line was also provided. After excavation and grouting, a construction treatment of the cutoff trench generally consisted of careful preparation of a 10 foot wide strip centered on the middle of the trench floor. Concrete plugs were provided in karst features on the side slopes of the trench only where the 10 foot wide strip included part of the slope. Evidence of this treatment is shown in Figures 6 and 7. Other weaknesses were built into the

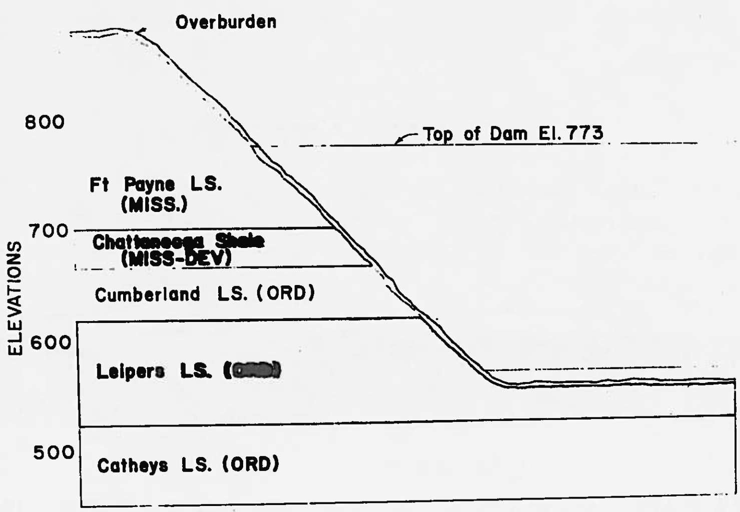
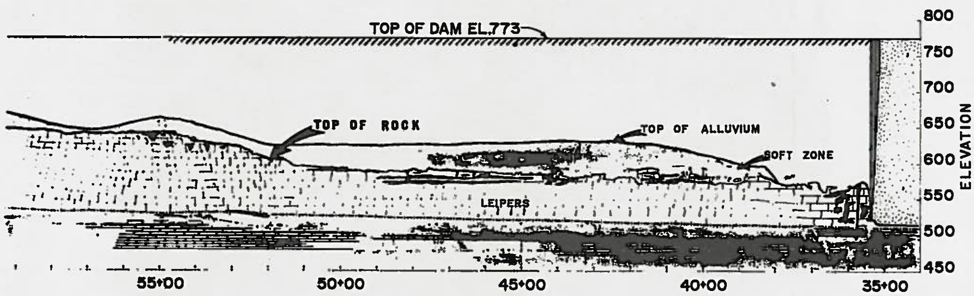


FIGURE 4. GEOLOGIC SECTION.

(1) This report by John M. Kellberg and Marvin D. Simmons has been submitted for publication to the Bulletin of the Association of Engineering Geologists under the title "Geology of the Cumberland River Basin and the Wolf Creek Damsite, Kentucky".



PROFILE ALONG AXIS OF DAM
STATION 0+00 A ℄ B

FIGURE 5. PROFILE ALONG AXIS OF EARTH EMBANKMENT.

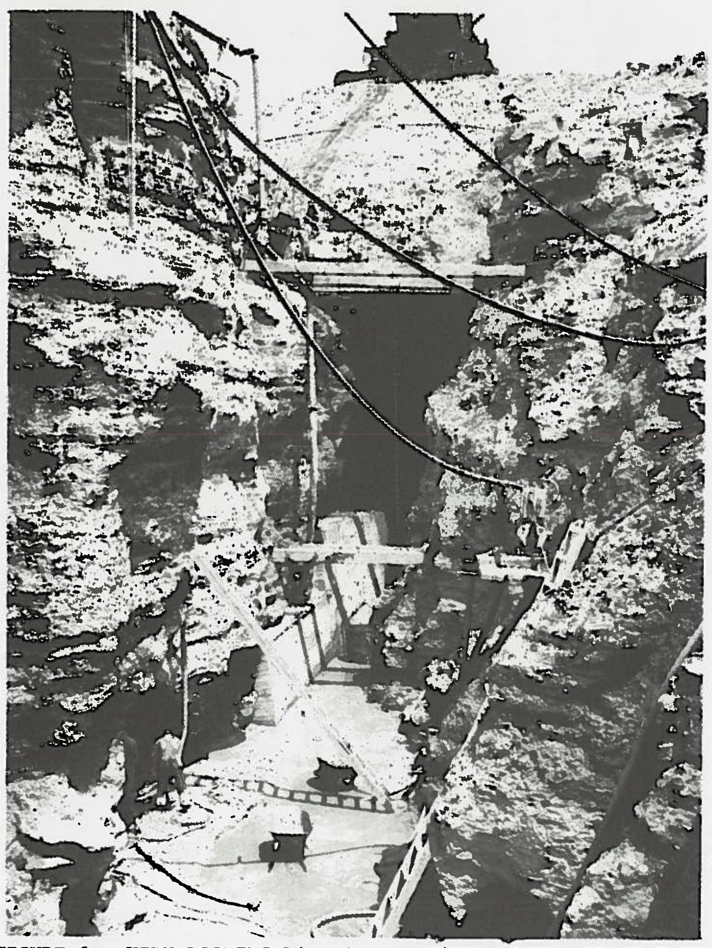


FIGURE 6. VIEW LOOKING LANDWARD. CUTOFF TRENCH DROPS FROM UPPER LEVEL INTO SOLUTION CHANNEL. NOTE CONCRETE PLUGS ON DOWNSTREAM SIDE.

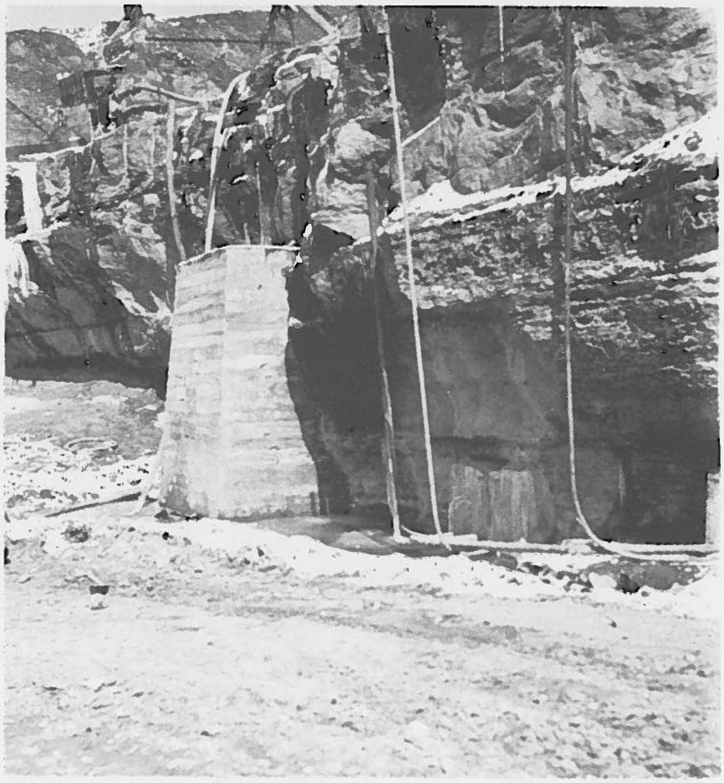


FIGURE 7. VIEW LOOKING DOWNSTREAM OF CONCRETE PLUG COVERING CAVITY IN CUTOFF TRENCH.

cutoff trench at its riverward juncture with the concrete section of the dam. Figures 8 and 9 show the narrow, steep-sided configuration of the trench in this location. Another dubious feature is the single line of grout holes located mainly below the horizon of major karstic development.

Development of Problems

Late in 1967, muddy flows were observed in the powerplant tailrace during periods of no generation. These flows were subsequently explained as piping of filling material from karstic features in the limestone. The piping was caused by reservoir seepage and drawdown in the channel resulting from 20 foot tailwater fluctuations. In March and April 1968, two sinkholes developed in the downstream toe of the earth embankment at the location shown on Figure 10. Explorations revealed a solution cavity system that linked the sinkhole area with the muddy flow area. The surface expression of the first sinkhole at its ultimate development is shown in Figure 11. Piezometer studies indicated a link with the reservoir through a near-perpendicular solution system that had apparently breached the cutoff trench in some way. The interconnected system is shown in Figure 12. The theorized mechanics for breaching of the cutoff, along with a similar mechanism for the 1968 sinkhole development and a mechanism for potential failure of the dam, are schematically shown in Figure 13.

Emergency Treatment

Three grout lines across the main solution system, as shown in Figure 14, were installed on an expedited basis. Indications are that this work greatly reduced the reservoir influence in the downstream area of the dam and probably averted a failure of the structure. Subsequent grouting, also shown in Figure 14, was completed in 1970 to fill known solution features and to consolidate the area of the cutoff trench tie-in to the concrete dam. Three hundred thousand cubic feet of grout solids were injected in this program, the details of which will be covered in an upcoming paper.

Long Term Evaluation

In 1972, after observing the instrumentation system of the dam and studying the foundation geology and design, the Corps decided that a review should be made by highly qualified non-Corps experts to appraise the safety of the dam and make recommendations for any treatments necessary to provide long term protection against a piping failure of the embankment into the karstic limestone foundation. These experts concluded that grouting could not be relied upon for long-term protection and that a positive cutoff should be installed through the dam and the karstic foundation features. After careful study of various means of providing this cutoff, the Corps decided on

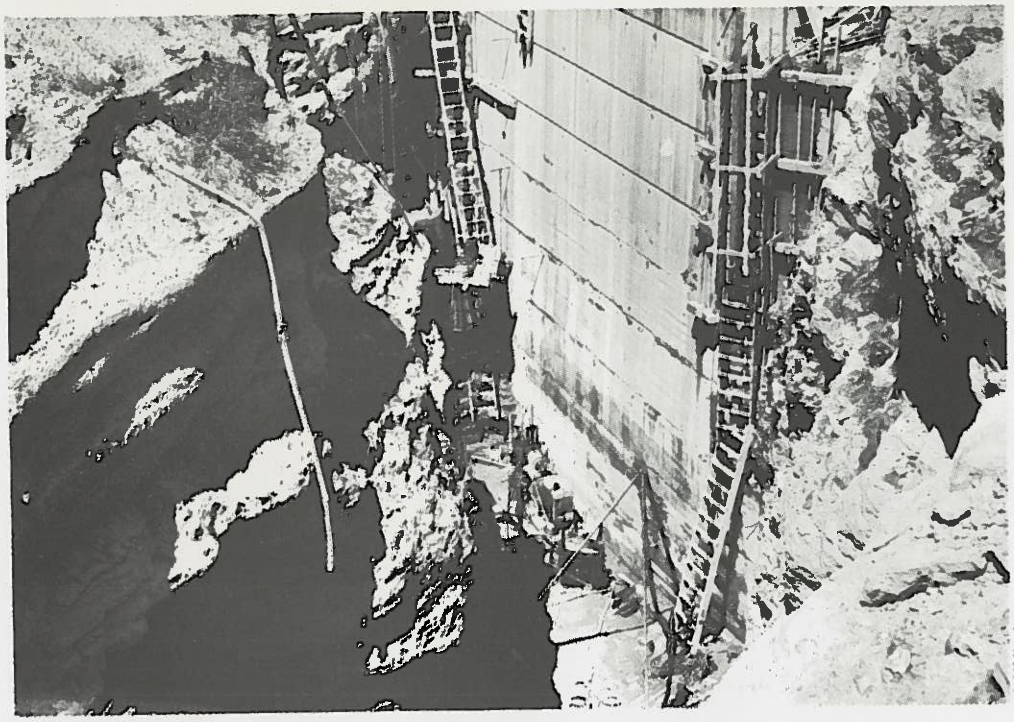


FIGURE 8. CUTOFF TRENCH AT TIE-IN TO CONCRETE STRUCTURE.

FIGURE 9. CUTOFF TRENCH AT CONCRETE STRUCTURE LOOKING LANDWARD.

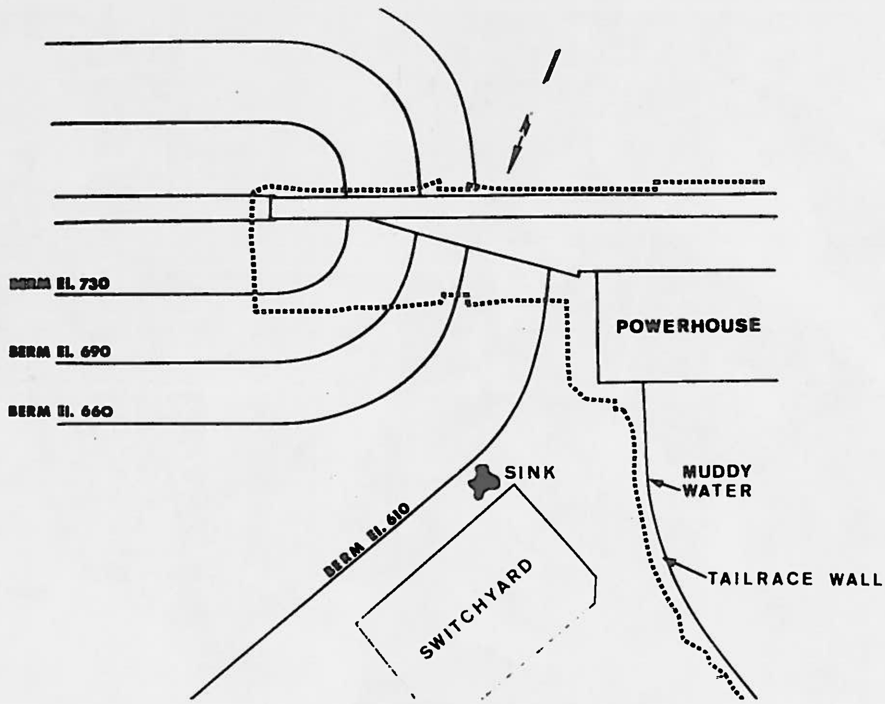


FIGURE 10. LOCATION OF SINKHOLES.

FIGURE 11. FIRST SINKHOLE AT TOE OF EMBANKMENT.

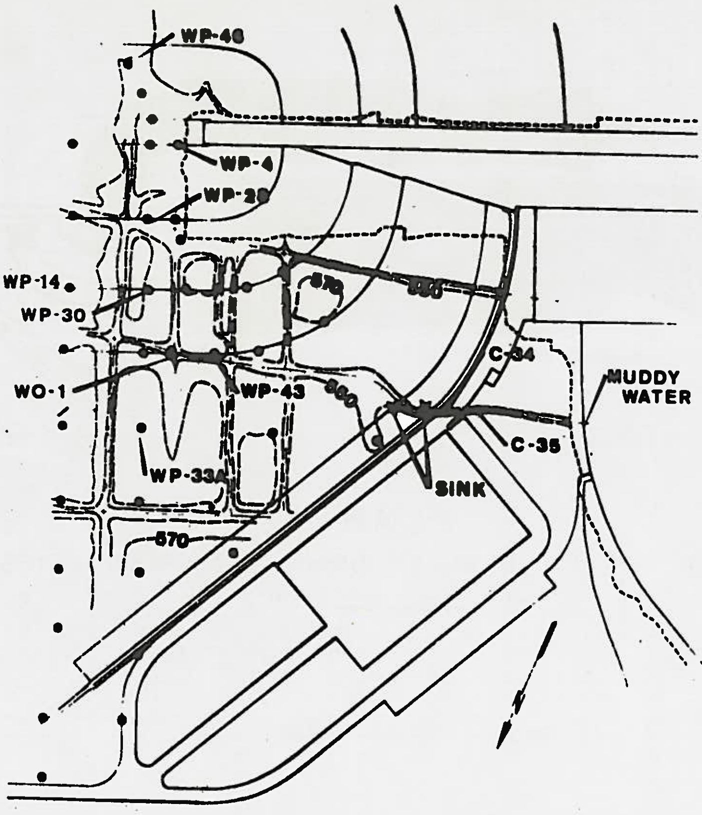


FIGURE 12. GEOLOGIC INTERPRETATION OF TOP OF ROCK IN APRIL 1968.

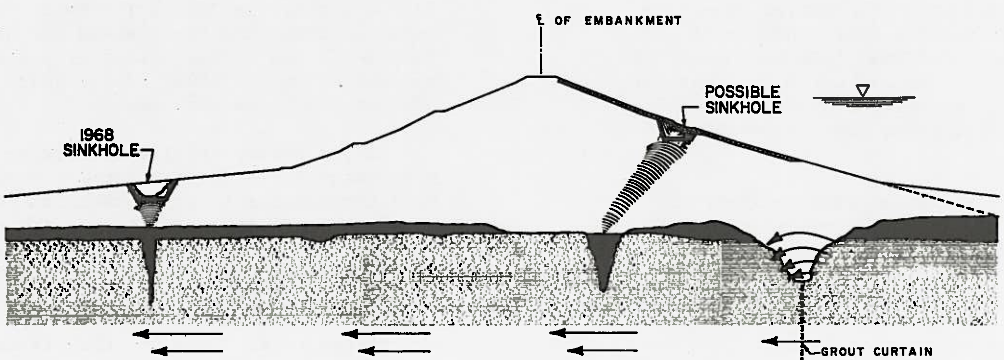
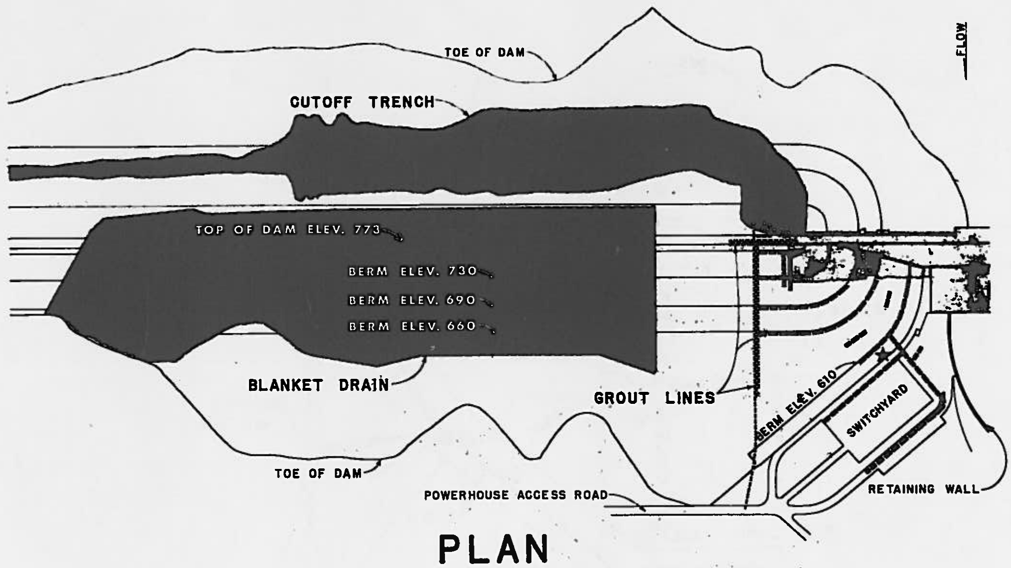


FIGURE 13. DEVELOPMENT OF PIPING AND SINKHOLES IN AN EARTH DAM ON LIMESTONE FOUNDATION.



PLAN

CUTOFF TRENCH, BLANKET DRAIN AND GROUT LINES

SCALE 200 0 200 400 FEET

FIGURE 14. LOCATION OF GROUT LINES.

a two foot thick concrete diaphragm wall extending some 2000 feet along the upstream crest of the embankment through the dam and foundation as shown in section in Figure 15. An artist's conception of the wall is shown in Figure 16.

Cutoff Wall Construction

Because of the vast scope of this project, it was decided to procure the wall installation under two separate contracts with methods of installation approved prior to taking bids. The two step contracting procedure was utilized to make this procurement. The first and most critical 1000 feet adjacent to the concrete dam was put under contract in 1974. The successful bidder at just under $50 million was ICOS Corporation of America.

The ICOS diaphragm wall consists of interlocking primary and secondary piles placed as shown in the plan of Figure 17. The procedure for construction of these interlocking piles begins with the excavation in the dry of a 51 inch outside diameter hole approximately 75 feet into the compacted clay of the embankment. After this hole is filled with a thin bentonite mud, a temporary one-piece casing--47 inch outside diameter and 80 feet long--is inserted into a hydraulic casing driver and positioned. Held at the top by the casing driver, this temporary casing requires no special grout. Then, with additional casing linked by special mechanical joints, this 47 inch casing is forced down by the casing driver while a clamshell continues the excavation to a depth of 140 feet. Throughout this operation, verticality is regularly checked by a direct plumb bob method.

A crane now suspends 140 feet of 41 1/4 inch temporary casing inside the 47 inch casing. The inner temporary casing is then advanced downward to continue the excavation past the alluvium (about 150 feet) to the bedrock (about 200 feet) where the oscillation performed by the casing driver along with the weight of the steel casing seats the notched shoe into the rock.

Next a rotary drill with reverse circulation excavates a 36 inch outside diameter hole through the rock. Meanwhile, to keep within the tolerance limits for the permanent casing, verticality is checked every 10 or 20 feet. When the bottom elevation of the wall is reached, exploratory drilling is carried out in order to test the underlying rock. Once the rock is determined sound and tight by pressure testing, the exploratory hole is grouted and the bottom of the hole is cleaned to remove grout and rock cuttings. The 47 inch casing is now withdrawn and the 41 1/4 inch casing freed. A 26 inch outside diameter permanent casing is weighted with

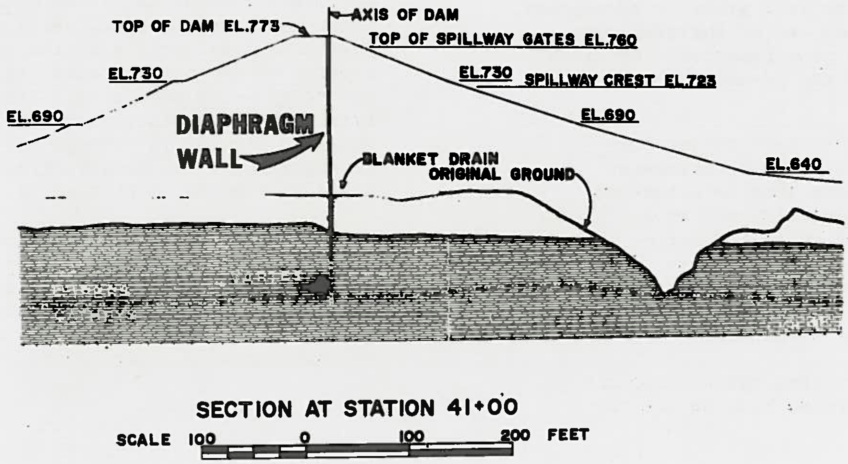


FIGURE 15. SECTION OF EMBANKMENT SHOWING CONCRETE DIAPHRAGM WALL.

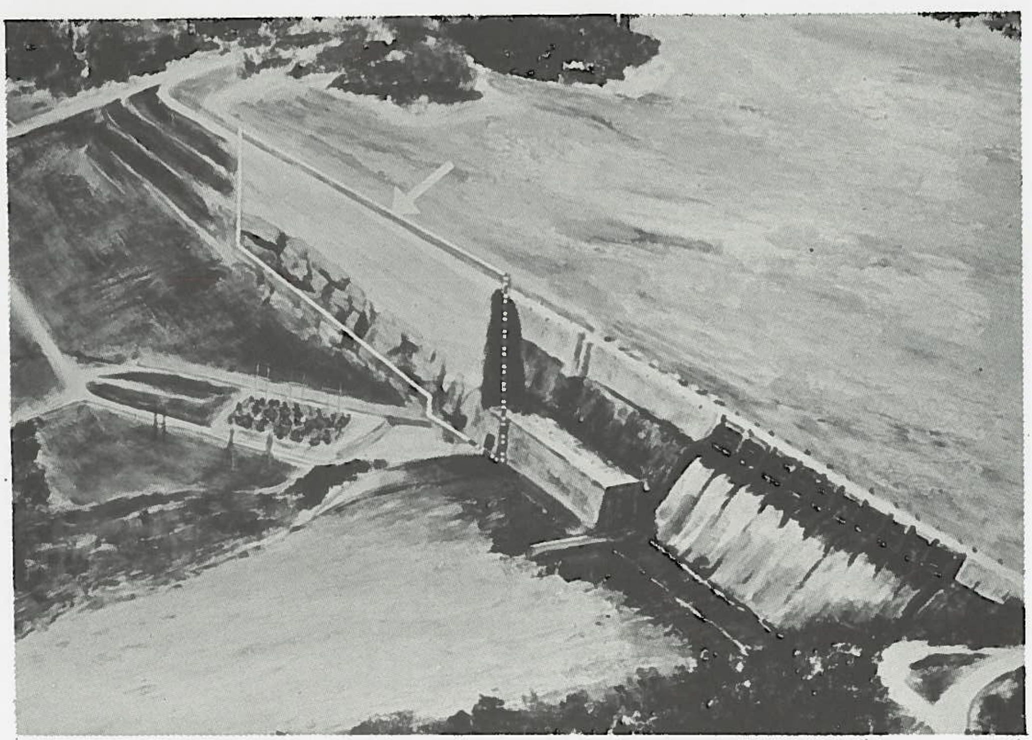


FIGURE 16. ARTIST'S CONCEPTION OF DIAPHRAGM WALL.

ballast and lowered into position. With the permanent casing in place, a weak grout is tremied into the annular space as the 41 1/4 inch casing is withdrawn. After waiting a minimal 24 hours for the weak grout to strengthen, the ballast is lifted out of the permanent casing and a tremie pipe inserted. Concrete is now tremied into the permanent casing.

Once two permanent casings (primary elements) are completed, the embankment and overburden between them is excavated under a very thin bentonite mud by a "Wolf Creek" rig and chisel bucket -- the latter is a specially designed clamshell that has a small set of jaws and bi-concave chisels that ride the outsides of the two permanent casings.

The excavation having reached top of rock, the rock remaining between the two permanent casings is broken out by a star chisel. After these cuttings are removed by the alternate use of a special clamshell and bailer, the entire excavation for the secondary element is filled with concrete using the tremie method. With this last operation, one section of the wall -- consisting of two primary elements and one secondary -- is completed. Steps illustrating the construction of the wall are shown in Figures 18 through 25, with photographs of the construction activity shown in Figures 26 through 32.

On July 5, 1977, a second contract was awarded to the ICOS Corporation of America to complete the wall installation at a contract price of $46.5 million. The evaluation of the performance of the wall will be the subject of a later paper.

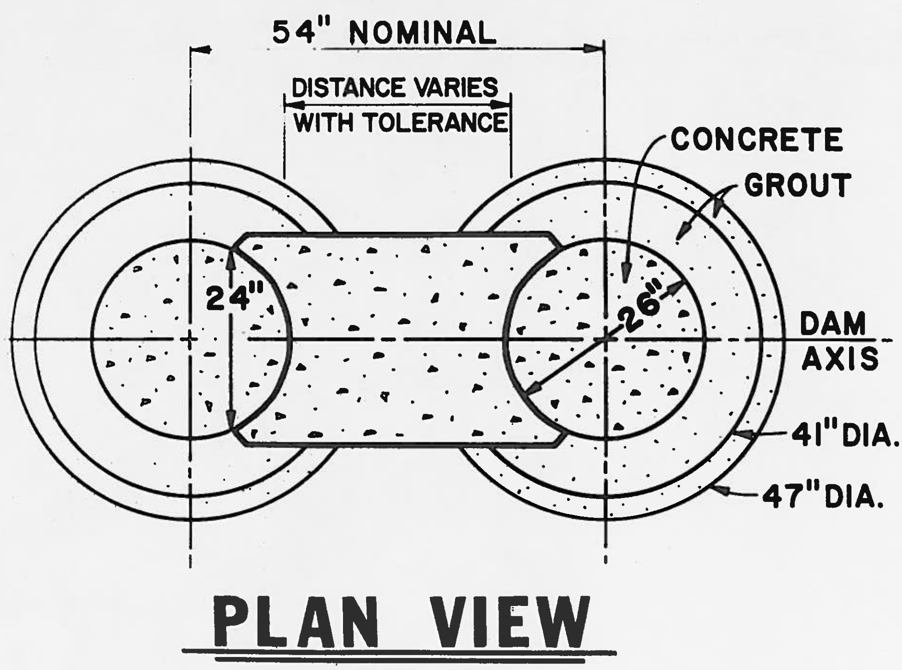


FIGURE 17. PLAN OF INTERLOCKING PRIMARY AND SECONDARY ELEMENTS.

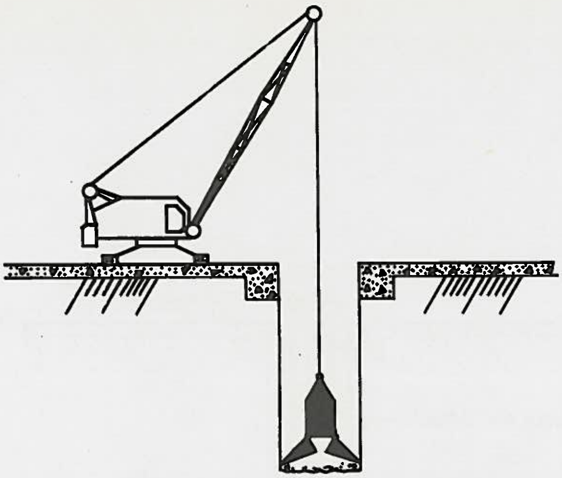


FIGURE 18. 50-INCH DIAMETER UNCASED EXCAVATION TO 75 FEET.

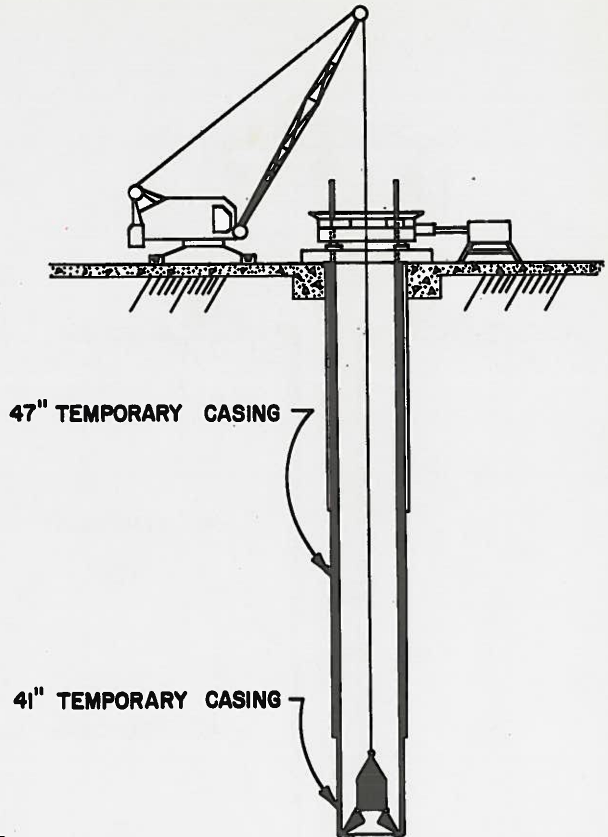


FIGURE 20. CASING DRIVER FORCING 41-INCH CASING AS EXCAVATION CONTINUES TO BEDROCK

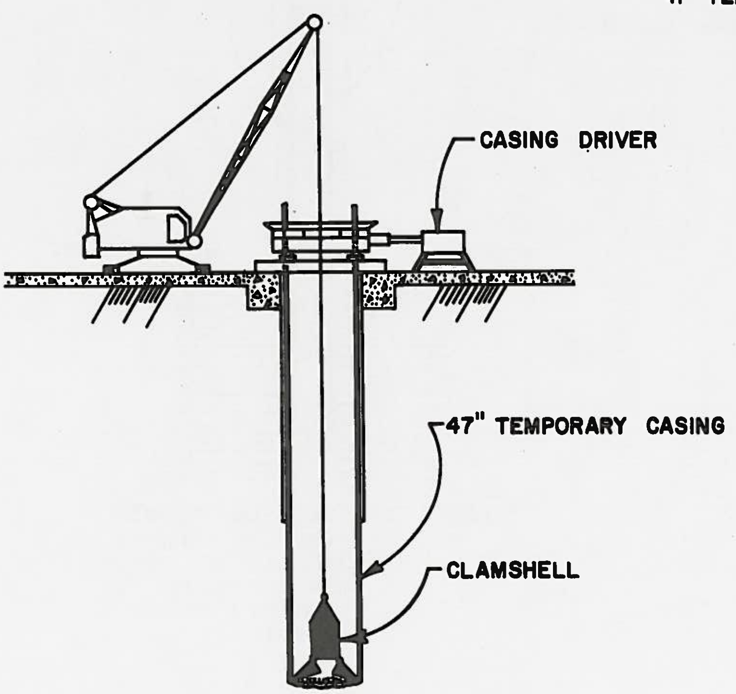


FIGURE 19. CASING DRIVER WITH 47-INCH CASING AS EXCAVATION CONTINUES TO 140 FEET.

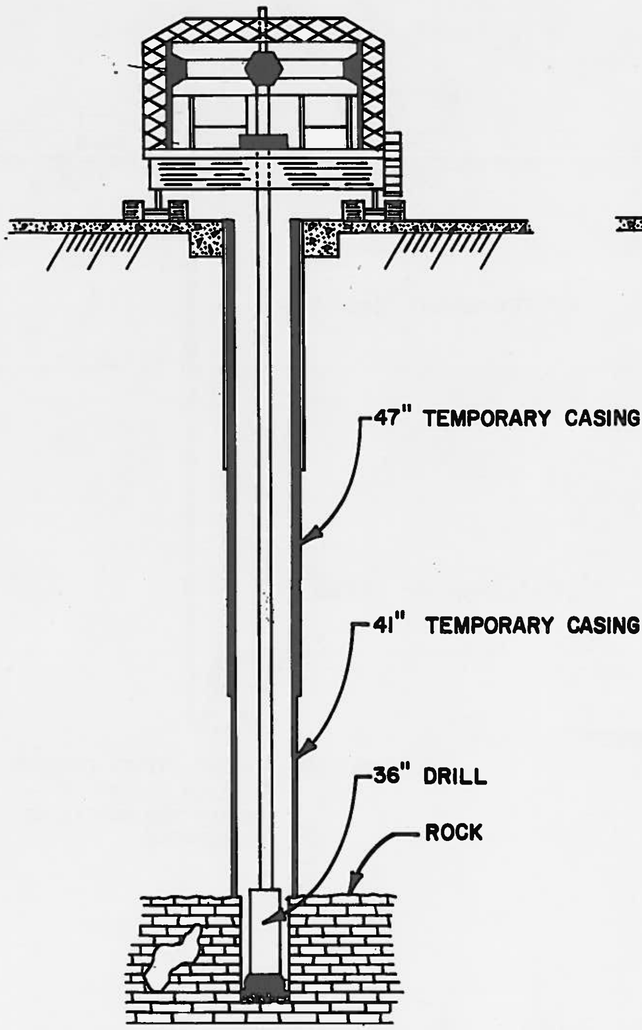


FIGURE 21. THE TEMPORARY CASING IS CARRIED TO BEDROCK AND THE ROCK DRILLED TO THE DESIGN ELEVATION.

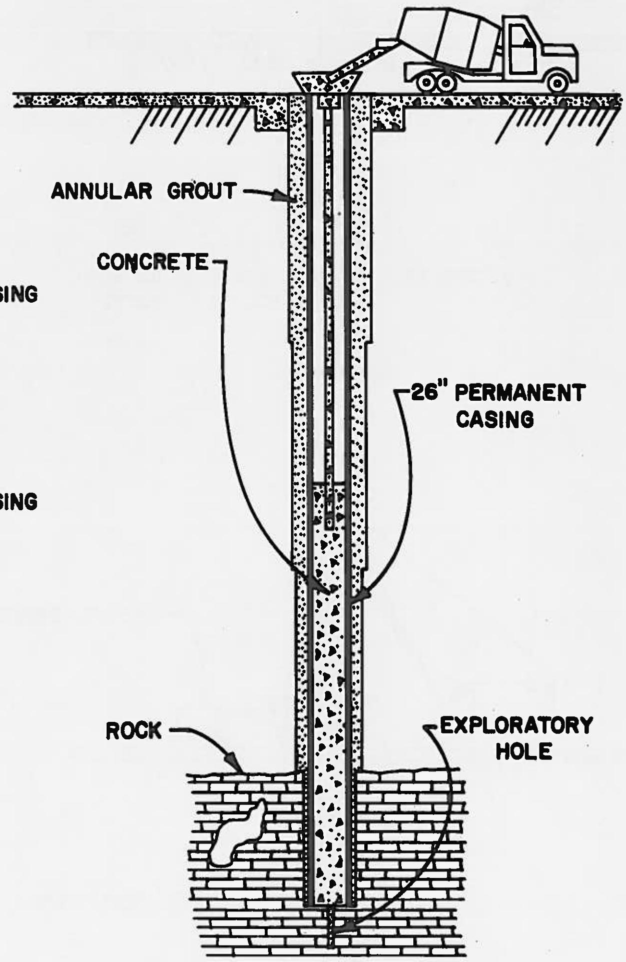


FIGURE 22. THE 26-INCH DIAMETER PERMANENT CASING IS INSTALLED AND TREMIE CONCRETE PLACED. TO COMPLETE A PRIMARY ELEMENT.

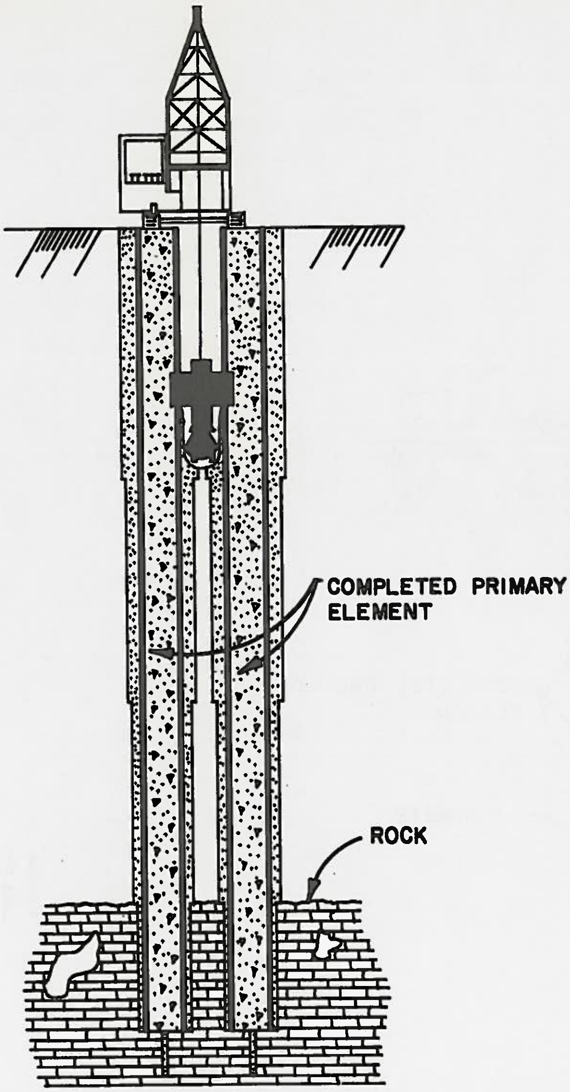


FIGURE 23. THE SOIL AND GROUT BETWEEN PRIMARY CASING IS REMOVED WITH WOLF CREEK RIG AND CHISEL BUCKET.

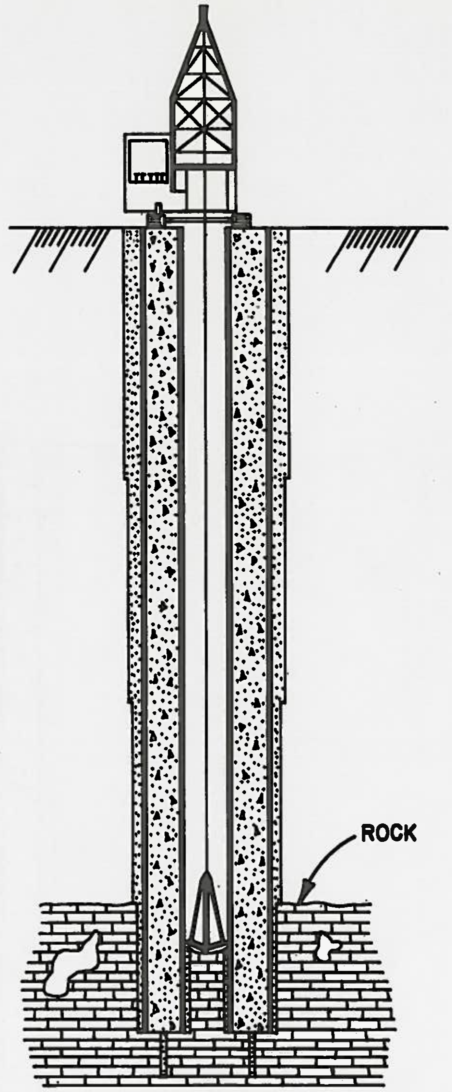


FIGURE 24. ROCK AND ANNULAR GROUT REMOVED BY ALTERNATE USE OF CHISEL AND CLAM-SHELL.

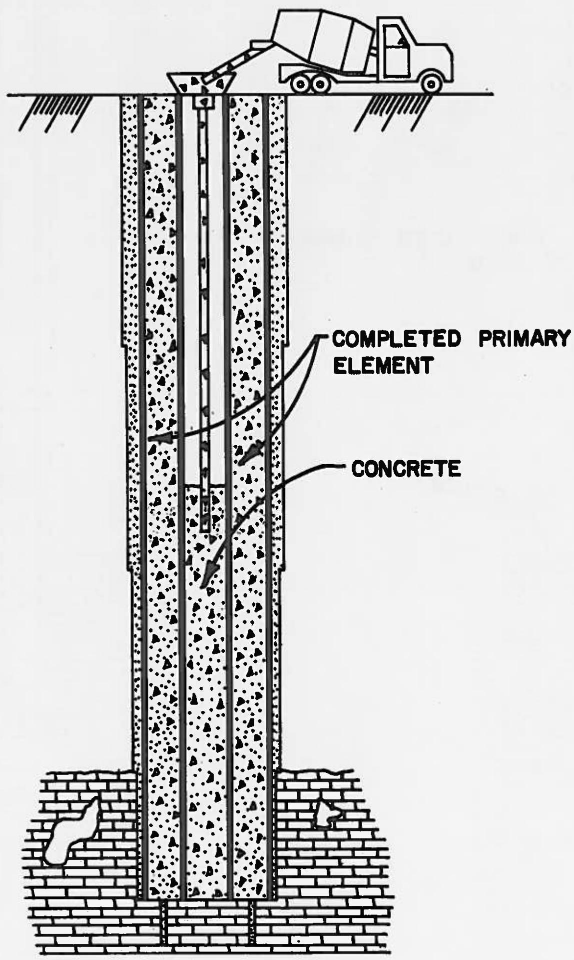


FIGURE 25. THE EXCAVATION IS CLEANED AND TREMIE CONCRETE PLACED TO COMPLETE A SECONDARY ELEMENT.

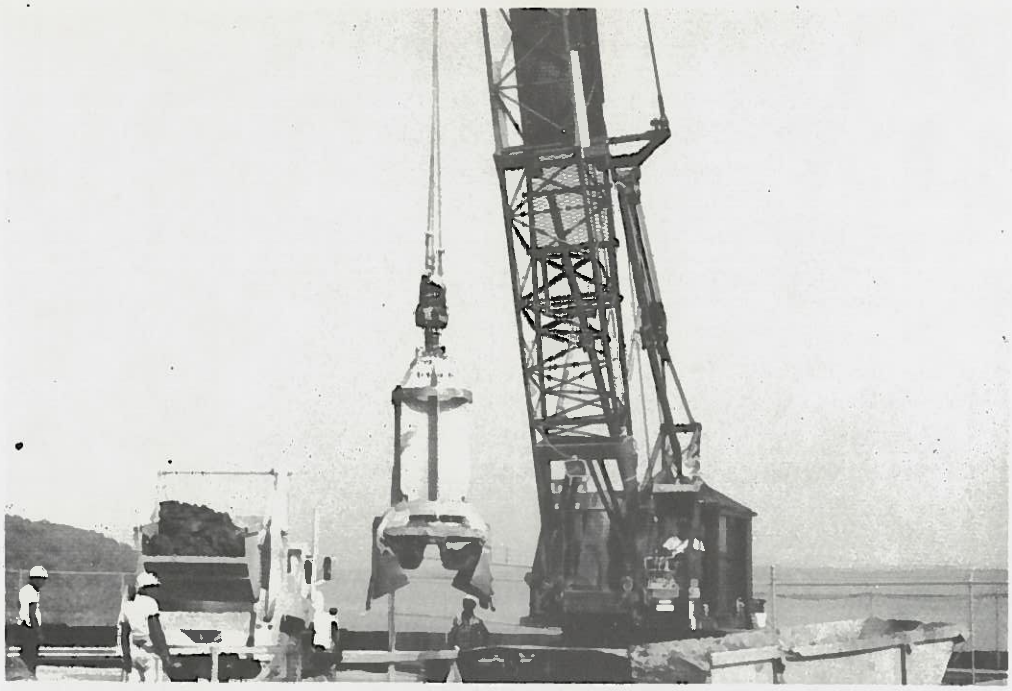


FIGURE 26. EXCAVATION OF 50 INCH DIAMETER HOLE WITH CLAMSHELL.

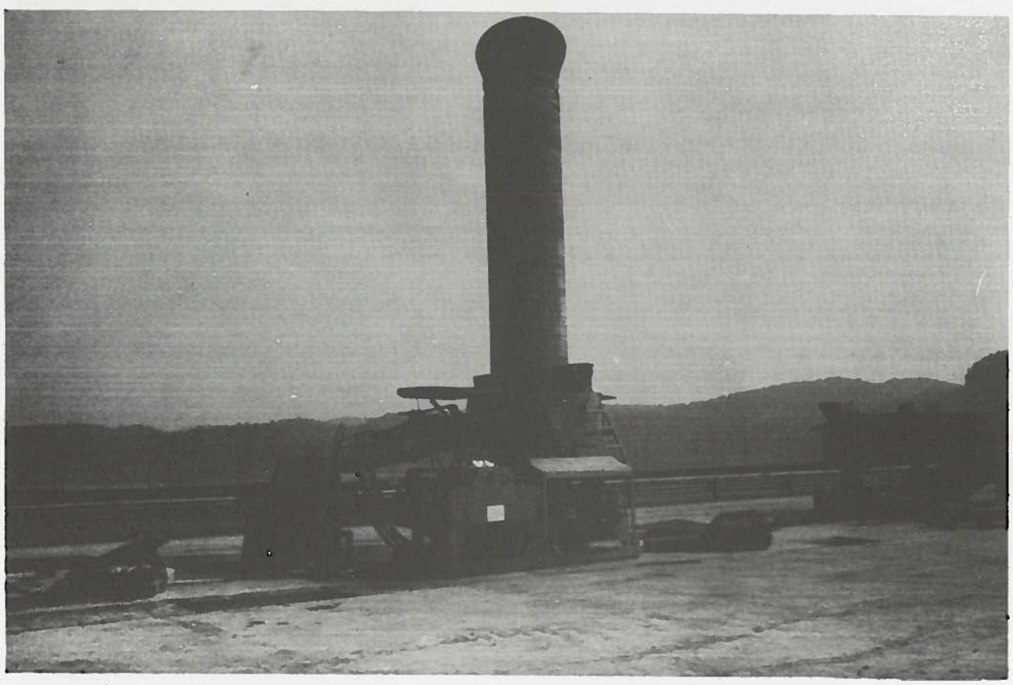


FIGURE 27. VIEW OF CASING DRIVER WITH TEMPORARY CASING.

FIGURE 28. EXCVATION WITH CLAMSHELL THROUGH TEMPORARY CASING. NOTE CASING DRIVER.

FIGURE 29. 36-INCH DIAMETER ROTARY DRILL WITH REVERSE CIRCULATION.

FIGURE 30. "WOLF CREEK" DRILL EXCAVATING SECONDARY ELEMENTS.

FIGURE 31. BI-CONCAVE CHISEL WITH SMALL CLAMSHELL FOR EXCAVATING SECONDARY ELEMENTS.

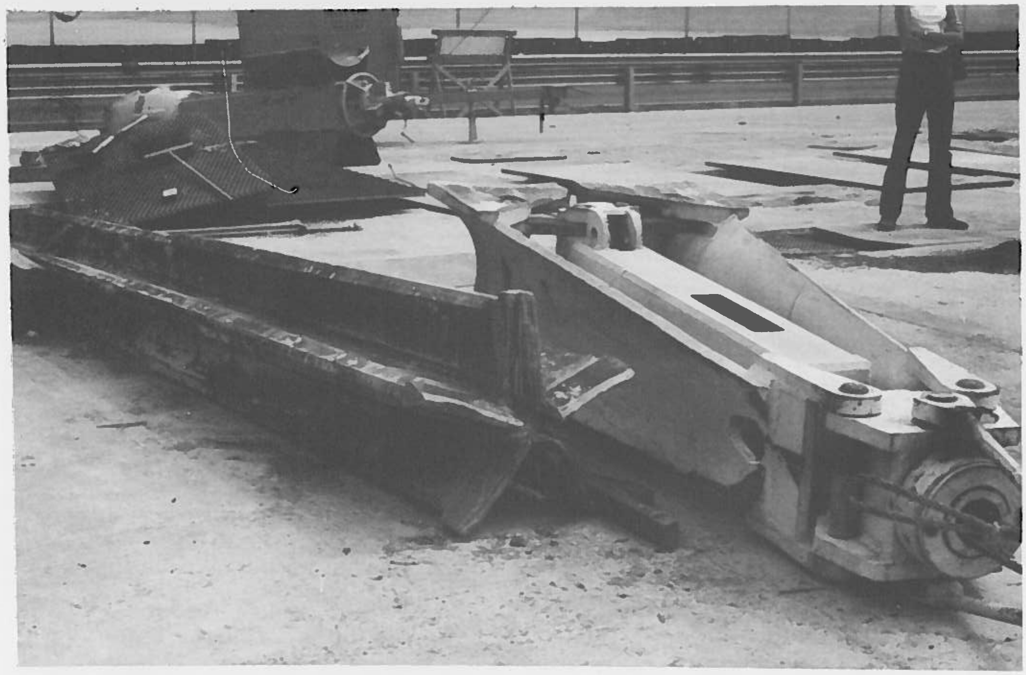


FIGURE 32. CHISELS USED IN SECONDARY EXCAVATIONS.

ACKNOWLEDGMENT

The writer is grateful to Albert J. Dunn for helpful comments on the paper and organizing the figures. The editing of the manuscript by Elizabeth Sarcone is greatly appreciated.

DESIGN AND CONSTRUCTION OF HIGHWAY
EMBANKMENTS IN NEW YORK STATE

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ABSTRACT. This paper summarizes some of the experiences and knowledge gained from thirty years of design and construction of highway embankment foundations in New York State. An adequate and economical solution for any foundation problem requires thorough attention to the details of investigation, design, and construction. Shortcomings in any one of these three phases can lead to an unsuccessful end result. Case histories are included to demonstrate the various types of foundation treatments that may be selected, the consequences of mistakes during investigation and design, the advantages gained from using new technology where applicable, and the difficulty in predicting the behavior of complex soil deposits under embankment loads.

Many embankment foundation problems in New York are associated with lacustrine varved silt and clay deposits. The strength and consolidation design parameters are influenced by the past geologic stress history. Special considerations used for investigation, testing, and design are summarized.

INTRODUCTION

Prior to World War II most of our highways were designed to fit into the existing terrain without major grading except in mountainous areas. However, modern highway designs are governed by geometric standards for sight distance, grade, curvature and safety. As a result, extensive embankments and cuts of various heights are a major component of our modern highway systems.

A failure in a highway system can be considered as any roughness or damage to the highway that affects the safety of the traveling public. Most pavement distress is caused by traffic loads and the environmental forces of temperature variations and water acting upon the pavement system. However, there are instances when the pavement failure is caused by deficiencies in the support provided by the embankment or embankment foundation soils. An embankment foundation failure may be a spectacular slide resulting from a shear failure in the underlying foundation soils or a slow continuing distortion caused by plastic deformation or consolidation of compressible fine grained deposits. This paper relates only to failures concerning embankment foundation soils.

In New York many embankment foundation problems have been encountered during the expansion and improvement of its highway system over the last 30 years. The large number of foundation problems result from an extensive Statewide highway system and the location of major transportation routes in the valleys where critical soil deposits occur.

As the last of the Wisconsin Age glaciers receded northward across the State many of the major river valleys contained temporary lakes. Varved silts and clays up to 300 feet in thickness now fill these former lake sites. In western New York the ancient shorelines of Lake Ontario and Lake Erie extended far inland leaving extensive deposits of silts and clays. The St. Lawrence Lowlands and Champlain Lowlands on the northern boundary of the State contain deposits of sensitive marine clays. The lowland areas containing lacustrine silt and clay deposits are shown on Fig. 1. All over the State, including the upland areas, small post-glacial lakes or ponds existed which have been subsequently filled with highly compressible peat and muck. Tidal marshes containing deep deposits of organic silt and clays are found in the lower Hudson Valley and western Long Island.

In the late 1940's, New York State embarked on a major highway construction program and experienced a number of costly foundation failures due to lack of adequate investigation and analyses. The State Budget Director, concerned with the costly overruns, established a Soil Mechanics organization in what is now the Department of Transportation. The organization mission was to apply the new technology of soil mechanics to adequately investigate and design embankment foundations Statewide. This Soils organization expanded with the increased highway program in the 1950's and 1960's and applied the developing knowledge of geotechnology not only to embankment foundations but to many other areas involving soils, rock and water problems such as structure foundations, cut slopes, pavements, subdrainage, soil stabilization, and soil erosion.

Each year the Soil Mechanics Bureau investigates over a hundred major embankment foundation problems. The general approach for investigation and design has been outlined in the previous

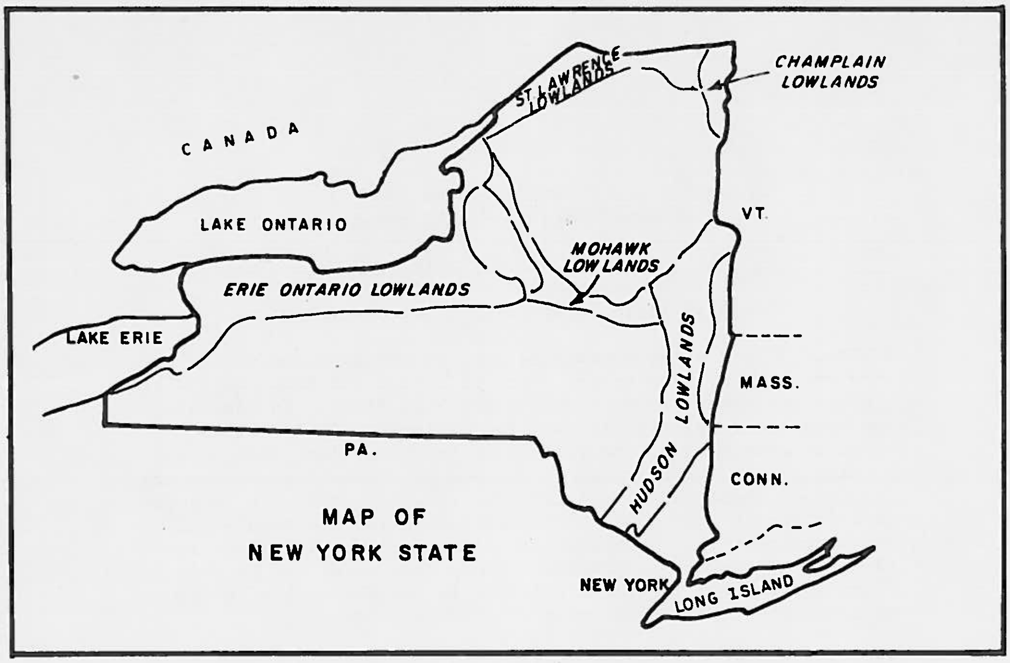


Figure 1. Map of principal lowland areas in New York State

section. In the last 30 years, there have been many advances in geotechnology which have been incorporated into the investigation, testing, and design activities to improve the adequacy of the treatment. However, due to unusual soil conditions, procedural oversights or lack of construction knowledge, there have been a small number of projects where failures of various degrees have been experienced.

In retrospect, most of the technical developments in soils over the past 30 years have been in analytical procedures. Implementation of these improved procedures have contributed to an improved record of successful projects. However, the Geotechnical Engineer should be aware that the analytical design is probably one of the easier problems to solve and that success depends on careful assessment and investigation of the subsurface conditions, appropriate testing, selection of applicable design parameters, and adequate control of construction operations. This paper describes the procedures that have been developed to investigate and design stable highway embankment foundations, the methods of treatment used, and includes several case histories of several failure experiences to illustrate the gap between theory and successful practice.

DESIGN AND CONSTRUCTION PROCESS

The approach to an adequate and economical solution for any embankment foundation problem has three principal phases -- investigation, design, and construction. A successful end result depends upon adequate attention to each one of these three phases. Mistakes or oversights in any one phase can result in failure.

SOILS INVESTIGATION. It is essential that the soils investigation program is conducted early in the project development in order that major foundation problems may be identified, investigated, and evaluated before final alignment and grade is established. The initial investigation during the location studies consist of a terrain reconnaissance survey. A typical highway project on new location may traverse many different soil deposits and may be located on highly variable terrain. Terrain reconnaissance is based on the concept that deposits formed by identical geologic processes will have similar engineering properties. Agricultural Soil Survey maps, air photo interpretation, and field inspections are utilized to identify and define the boundaries of major soil deposits. A report is prepared for each project, including a soil map identifying the deposits and an evaluation of the potential soils engineering problems for each deposit. This report is used to determine where major soil problems will occur and is used as a guide for planning subsurface investigations.(1)

Preliminary soils investigations are performed in areas where major embankment foundation problems are expected. Sufficient subsurface explorations and testing are conducted to develop the soil profile and the strength and compressibility characteristics of critical deposits. Analyses are conducted to determine the general types of treatment required and preliminary cost analyses are prepared. If this preliminary analysis indicates that major foundation problems exist, then alignment shifts or gradeline changes may be made that will eliminate or sharply decrease the cost of foundation treatment. Changes of this nature should be made in the early stages of project development in order not to disrupt the design process.

DESIGN. After the final highway location is established, additional explorations, testing and analysis are conducted to determine in detail the most economical and adequate solution, and limits of the foundation treatment. Adequate drawings, special notes, and specifications are required for the contract documents to explain the desired foundation treatment. It is important that the foundation problem and general treatment are adequately described in order that the Contractor and project engineer understand the purpose, desired results, and method of payment for the treatment.(2)

CONSTRUCTION. A common weakness in many engineering projects is the communication gap that often exists between the designers and the people involved in construction. For major foundation designs, a preconstruction conference is held between the Soils engineers, the Contractor, and the project engineer. This establishes a thorough understanding of the desired treatment required and an opportunity for the project people to obtain full knowledge of the problem.

On major projects, construction control instrumentation such as settlement platforms, piezometers and inclinometers are included to monitor the behavior of the subsoil during the construction operation. This provides the ability to compare foundation soil performance with the design assumptions; to prevent failures if stabilization does not progress as anticipated; and often to decrease the required construction time when stabilization is more rapid than anticipated.

METHODS OF TREATMENT

The method of foundation treatment selected for any problem is determined by the prime considerations of adequate end results and economics. Often the economics of the treatment is influenced by many factors such as highway alignment, grade requirements, available time for construction, right of way cost and limitations, land usage, available construction materials, and design class of highway. Principal considerations for an adequate pavement involve stability of the foundation soil and the post-construction settlement of the pavement. In many cases, the treatment method selected solves both of these considerations. A summary of possible treatments is shown in Table 1.

Table 1. Summary of Embankment Foundation Treatments

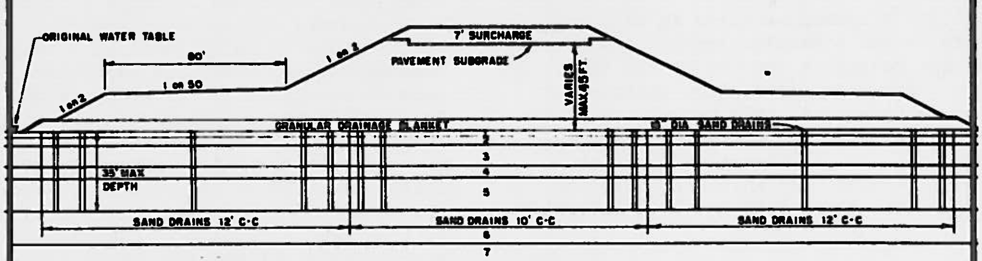
	Settlement	Stability
Waiting Period Before Paving	X	
Surcharge and Waiting Period	X	
Stage Construction	X	X
Counterweight Berms		X
Removal and Replacement	X	X
Lightweight Material	X	X
Vertical Sand Drains	X	X
Horizontal Drains		X
Stone Columns	X	X
Pile-Supported Structure	X	X

CASE HISTORIES

The first two case histories have been selected to show the successful application of embankment foundation treatment procedures. Two other case histories describe unforeseen problems that occurred during construction. The description of the projects is followed by a discussion of the lessons learned from each experience. More detailed data on several of these projects may be obtained from publications included in the bibliography.

OSWEGO BOULEVARD, SYRACUSE, NEW YORK. This first case history illustrates an extreme problem in stability and foundation settlements where a number of treatment methods were combined to achieve successful stabilization. This project, constructed in 1957, is located in central New York near Syracuse. The problem involved a 45 foot high embankment for a highway structure over a railroad embankment. The subsurface soils were deposited in a postglacial lake and consisted of very soft deposits of peat, marl and sensitive silty clay as shown in Fig. 2. The foundation design required counterweight berms to prevent shear failures and sand drain installations to increase the rates of settlement and gain in shear strength. A surcharge and waiting period was also employed to decrease post-construction settlements. The rate of embankment construction was determined by piezometer and settlement platform records that were used to determine the soil strength increase with consolidation. Typical settlement platform records and piezometer records are indicated on Fig. 3 and Fig. 4. Post-construction investigations were conducted to determine the moisture content changes due to consolidation and the soil boundary elevations after settlement. Close agreement was found between the measured field settlement, theoretical design settlement determined from the consolidation tests, and the calculated settlement determined from change in moisture content, as shown on Fig. 5. Total settlements at the bridge abutment achieved during construction were between 8 and 9 feet. Post-construction settlements adjacent to the pile supported abutment have been in the order of 3 to 4 inches in the last 20 years. This is an example of a successful foundation treatment project for a high embankment on very soft soils.(3)

The sand drains on this project were installed by the closed end mandrel method which displaces and remolds the soil structure and in some soils may decrease the initial shear strength and increase to total settlement. This was not a problem on this project. Later in the 1950's, a similar displacement type sand drain installation was made in tidewater organic clay deposits in the Borough of Queens, New York. The weak sensitive clay structure was remolded and measured settlements were considerably greater than those anticipated in design. Post-construction analysis indicated that this additional settlement was caused by the volumetric displacement of the mandrel remolding the soils structure.(3) In recognition of this problem, most sand drain installations today are made by minimum displacement procedures such as the hollow shaft flight auger method and the jetting method developed in Holland.



NOTE Shown are the original characteristics of the layer materials as determined from lab tests for design.

TYPICAL SECTION
SCALE 1" = 40'

LAYER	THICKNESS	SOIL	AVE MOISTURE CONTENT %	LIMITS			AVE. SHEAR STRENGTH		AVE. CONSOLIDATION CHARACTERISTICS		
				LL	PL	PI	CONSOLIDATED UNDRAINED C-LBS/FT^2	UNCONFINED STRENGTH LBS/FT^2	COMPRESSION INDEX	COEFFICIENT OF CONSOLIDATION C_v - FT^2/DAY	INITIAL VOID RATIO
1	3'	SOLVAY SLUDGE	280	150	128	22	$\phi = 0^\circ$ C = 350	300	4.00	3.0	7.50
2	2'	BLACK PEAT	200	-	-	-	$\phi = 0^\circ$ C = 350	300	1.50	0.1	4.00
3	10'	GRAY MARL	70	48	35	13	$\phi = 20^\circ$ C = 200	180	0.54	1.0	1.85
4	5'	SILT, SAND AND MARL	30	26	20	6	$\phi = 15^\circ$ C = 200	250	0.1 - 0.2	0.5-1.0	0.75-1.00
5	15'	SOFT SILTY CLAY	38	32	18	14	$\phi = 10^\circ$ C = 200	180	0.35	HORZ 0.12 VERT 0.07	1.05
6	15'	FIRM SILT	25	NPL			$\phi = 25^\circ$ C = 300	400	0.02	2.0-3.0	0.69
7	-	FIRM SAND	20	NPL			-	-	-	-	0.56

Figure 2. Subsurface conditions and design parameters Oswego Boulevard, Syracuse, New York

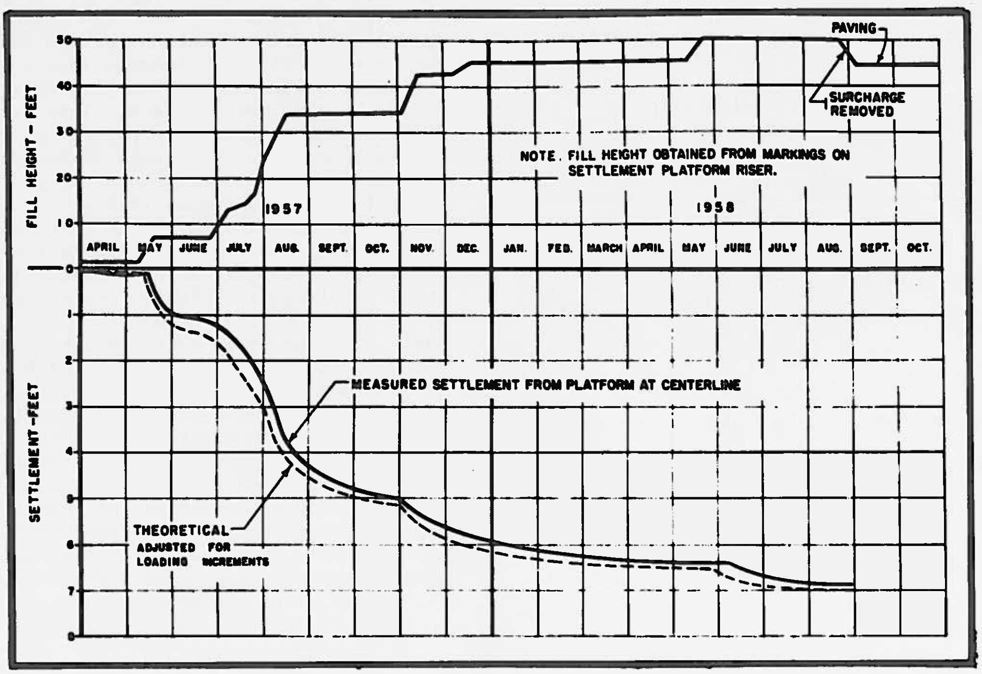


Figure 3. Typical Time - Settlement Data Oswego Boulevard, Syracuse, New York

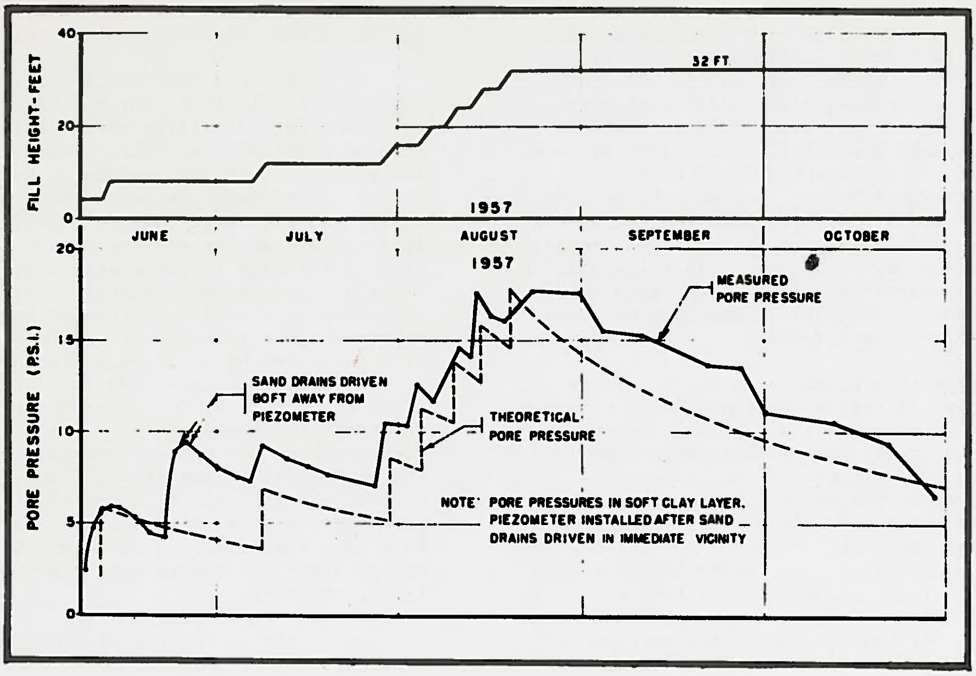


Figure 4. Typical Time - Pore Pressure Data
Oswego Boulevard, Syracuse, New York

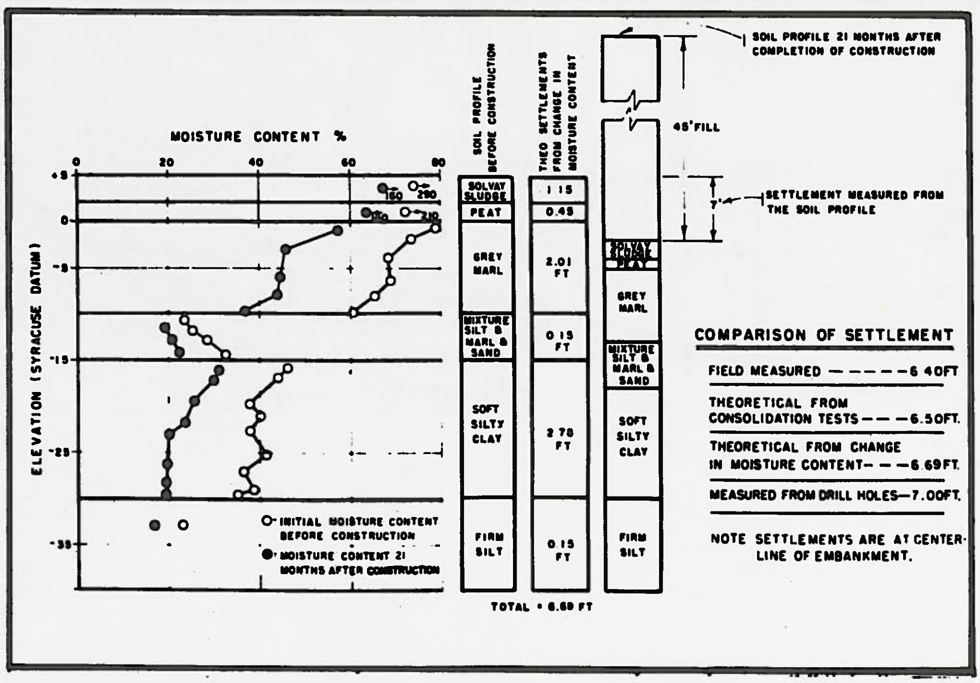


Figure 5. Analysis of embankment foundation settlements
Oswego Boulevard, Syracuse, New York

INTERSTATE ROUTE 81, SOUTH OF SYRACUSE, NEW YORK. This case history demonstrates the use of new available technology as a means of economically solving foundation problems. The problem site occurred under an 80 foot high embankment crossing a rather narrow, steep sided tributary valley. The project was constructed in 1966. The embankment was stable until the early 1970's, when settlements of the roadway required patching occurred across the 400 foot long embankment. In 1973, a severe crack occurred, dropping the pavement several inches and this movement continued requiring cumulative patches up to 18 inches in thickness. A 36 inch concrete drainage pipe, in the axis of the gulley underwent considerable distortion and soon lost its drainage effectiveness.

The soil profile in the axis of the narrow valley, as shown in Fig. 6, consisted of a variable surface layer of sand, silt and gravel, probably sloughed glacial till, underlain by firm silt and clay over sand and silt over rock. A detailed exploration program indicated a high water table existed in the fill with considerable seepage emerging on the lower fill slopes. Also, artesian pressures were encountered at the boundary between the silt and clay and the underlying sand and silt. This artesian condition was not found in the design explorations. The instability of the embankment was caused by the increase in effective weight of fill and also the artesian pressures influencing the strength of the clay. Bedrock was shallow on both sides of the valley walls and provided an ample supply of water for the soil deposits in the axis of the valley. It is probable that the embankment filling the valley caused the change in groundwater conditions. Stability analysis indicated that a buttress berm would be required at the toe of embankment slope and in addition, a shear key excavated 30 feet to bedrock and backfilled with stone would be required under the berm. The estimated cost of this treatment was $800,000.

At about this time there were several drilling companies in the East that had recently acquired equipment for installing horizontal drains with slotted plastic pipe. This procedure has been used for many years in Western states for stabilization of wet cut slopes. It was decided to install horizontal drains to lower the groundwater in the fill and a granular surface deposit and also to provide drainage in the underlying clay and sand deposits. Drains were installed at two levels as indicated on Fig. 6 and extended under the entire embankment for a length of 260 feet. The drains were installed by drilling in flush-coupled casing to the desired length. Two inch diameter plastic pipe, with slots 0.010 inches in width, was then inserted inside the casing. The drill bit was detached from the casing, and left in place. Then the casing was jacked from the hole leaving the slotted plastic pipe in contact with the soil. In plan seven drains were installed in the lower level in a fan shaped pattern to encompass the entire valley width and twelve drains were installed at the upper level.

The drains were effective and have worked continuously for the last four years with flow quantities reflecting the seasonal rainfall. The flow from the upper drains increases after rainfall while the lower drains in the subsoil flow more constantly. The total quantity of flow varies between 5,000 gallons per day in the fall to 30,000 gallons per day in the spring. The pavement has been stable since this treatment and a new culvert drainage pipe has been installed at a higher elevation in the embankment.

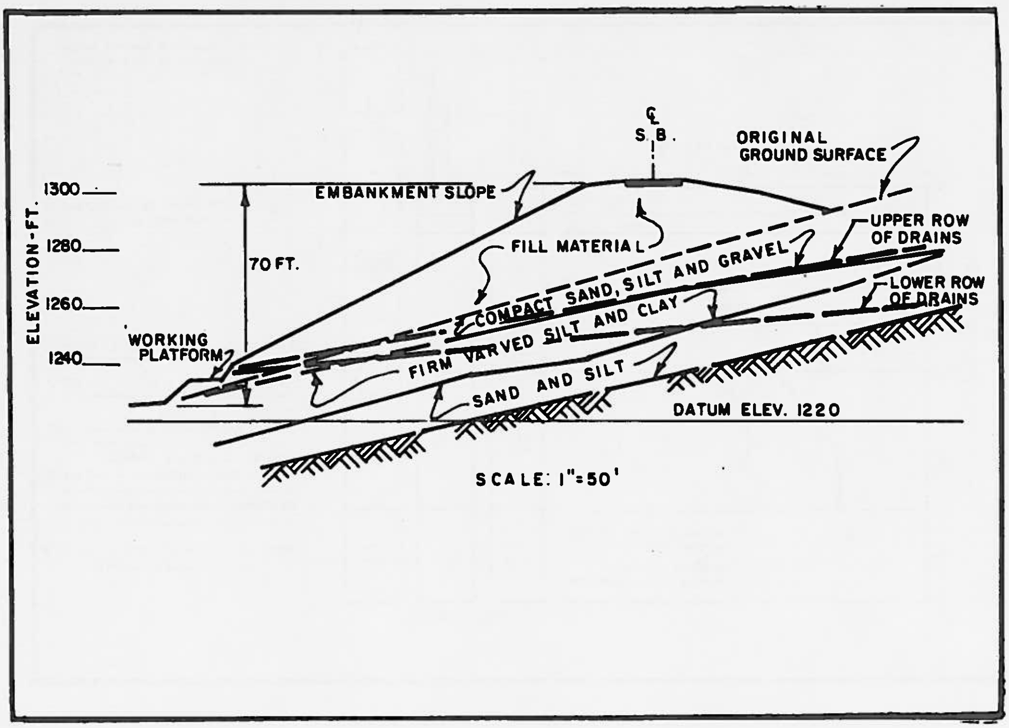


Figure 6. Horizontal Drain Installation
Interstate Route 81, South of Syracuse, New York

The important consideration in this treatment was that the cost of the horizontal drain installation was $40,000 as compared to the $800,000 estimate for the buttress berm and shear key design. This is an example of how implementation of a new available procedure can provide significant savings.

INTERSTATE ROUTE 88, NEAR ONEONTA, NEW YORK. The third case history illustrates the importance of selecting the critical soil layers for testing and design and the importance of selecting appropriate embankment unit weights for stability analysis. The problem site involved an embankment 3,000 feet long, crossing a valley with fill heights of 35 to 45 feet. The subsurface deposit was alternating thick layers of silt and plastic clay over 100 feet in depth. Undisturbed samples were obtained for design and the test results indicated that silt layers predominated in the profile and the soil would have adequate strength to support the embankments during construction. Extensive shear failures occurred when the fill was constructed to within two to five feet of final grade. Additional post-mortem explorations indicated that the clay layers were more predominant in the soil profile than found in the original testing. Stabilizing side berms were constructed in this area and the embankment was brought to final grade without further failures.

A review of the problem indicated that there were several factors working in the same adverse direction to cause the shear failure. The first factor was that the Contractor elected to use a well-graded glacial till for the embankment and total unit weights for this embankment in place exceeded 150 pounds per cubic foot. The assumed unit weights in design were 135 pounds per cubic foot. This discrepancy alone decreased the safety factor by 0.11.

Second, the Contractor had a very efficient filling operation and these embankments were constructed in a period of two to three weeks allowing little time for consolidation and strength increase in the underlying soil. It was assumed that the consolidation in the silty material would proceed almost as fast as the embankment construction. This did not occur. If a design is based upon a strength increase during construction, then controls on the rate of embankment construction should be included in the contract documents.

Third, the upper 12 inches of material in all of the undisturbed sample tubes used for design testing was predominantly silt with moisture contents in the low 30's. Removal of the remaining material from the tubes, after construction, indicated that the clay content increased significantly with corresponding moisture content increase to the low 40's in the lower portions of the tube. The untested lower strength in the clay layers should have been the design strength to use at this site.

This case history indicates that Murphy's Law should be considered when conducting embankment foundation analyses. It would appear that all of the above factors were acting together in an adverse direction to contribute to the instability. However, failure to select the most critical clay layers for testing was probably the most important factor. As a result of this experience, the Bureau adopted a policy of x-raying all undisturbed tube samples prior to testing. X-ray equipment used for inspection of metal welds was available in the Department's Materials Testing Laboratory. The x-rays show silt and clay layers in various shades of gray and sand has a salt and pepper appearance. The x-rays also show disturbed material caused by improper sampling procedures and the presence of pebbles and stones which can affect the quality of the testing. The x-rays also allow the design engineer to determine the soil profile in the tube and select the strata for testing prior to opening the tube. The x-rays have also been very useful to demonstrate to drilling personnel the affect of improper sampling procedures on the quality of the sample. Fig. 7 shows an x-ray of a tube containing varved silt and clay in the lower portion and clay in the upper portion. Note the arched layers in the lower portion of the tube caused by improper sampling techniques.(4)

The selection of proper embankment densities are very important for stability analysis. We have participated in several post-mortem stability failure analyses conducted by the academic community to determine the most applicable method of stability analyses to use for the failure condition. In every case, we have been surprised by their lack of concern for the assumed unit weights for the embankment as compared to their interest in the proper analytical procedure. Since unit weights for embankments can vary from 100 pounds for rock fills to 155 pounds for well-graded glacial till, the computed factor of safety can vary by more than 0.25 depending upon the assumption used.

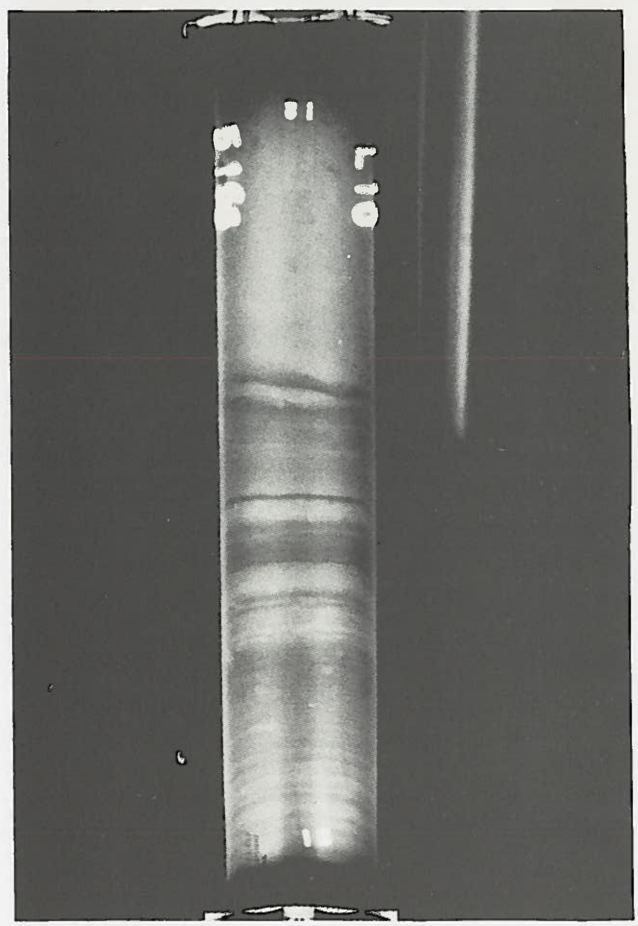


Figure 7. X-ray of undisturbed tube sample

SOUTHERN TIER EXPRESSWAY, EAST OF HORNELL, NEW YORK. This final case history demonstrates that failures can occur while using the reasonable practices of investigation and design. Some soil deposits are so complex that their behavior under loading cannot be anticipated from the information obtained by accepted investigation procedures.

A 70 foot high embankment was constructed on the side of a gently sloped valley (12^{o} slope). The foundation soils consisted of a rather complex deposit of interlayered clay and unassorted granular materials described on Fig. 8. When the embankment was within several feet of final grade, a 300 foot long section dropped 3 feet and the mudwave emerged near the bottom of the valley slope some 200 feet from the toe of the embankment berm. Since the mudwave was encroaching on an existing State highway, material was immediately removed from the embankment and an emergency berm, approximately 10 feet high, was constructed with this material. See Fig. 8. A subsurface investigation program indicated that extremely high artesian pressures were present in a 5 foot sand and silt layer, 25 feet below the ground surface. The design explorations had indicated a water table 0 to 8 feet below the ground surface. The original design stability analysis indicated that the clay layers had reasonable strengths to support the 70 foot embankment and the design included a small berm 20 feet high and 50 feet wide to keep the factor of safety against a circular arc failure above 1.30.

The unusual condition determined by the post-failure borings indicated that the 5 foot sand and silt layer was unable to drain the free water generated by the compression of the underlying firm to soft clay. The measured artesian head in this 5 foot sand and silt layer was 50 feet under the high embankment, 25 feet at the toe of embankment and 20 feet at the toe of failure at a distance of 200 feet from the embankment toe. A sliding wedge analysis applied to the geometrics of the failure indicated that a factor of safety of one would occur in the nondraining sand and silt layer under the measured artesian pressures. The post-failure drill holes provided a means of drainage for the sand and silt layer and the water pressures decreased slowly. It was determined that the emergency berm was adequate to provide a permanent factor of safety and the embankment was constructed to final grade with a rate of construction controlled by the pore pressure dissipation.

The design for this project assumed that the sand and silt layer would provide a means for drainage and allow consolidation and strength increase of the clay to proceed at a relative fast rate. However, it became evident after the failure that the sand and silt layer was surrounded by clay and offered no means of positive drainage. This is an example as to how failures can occur after applying reasonable and careful techniques for exploration and design.(5)

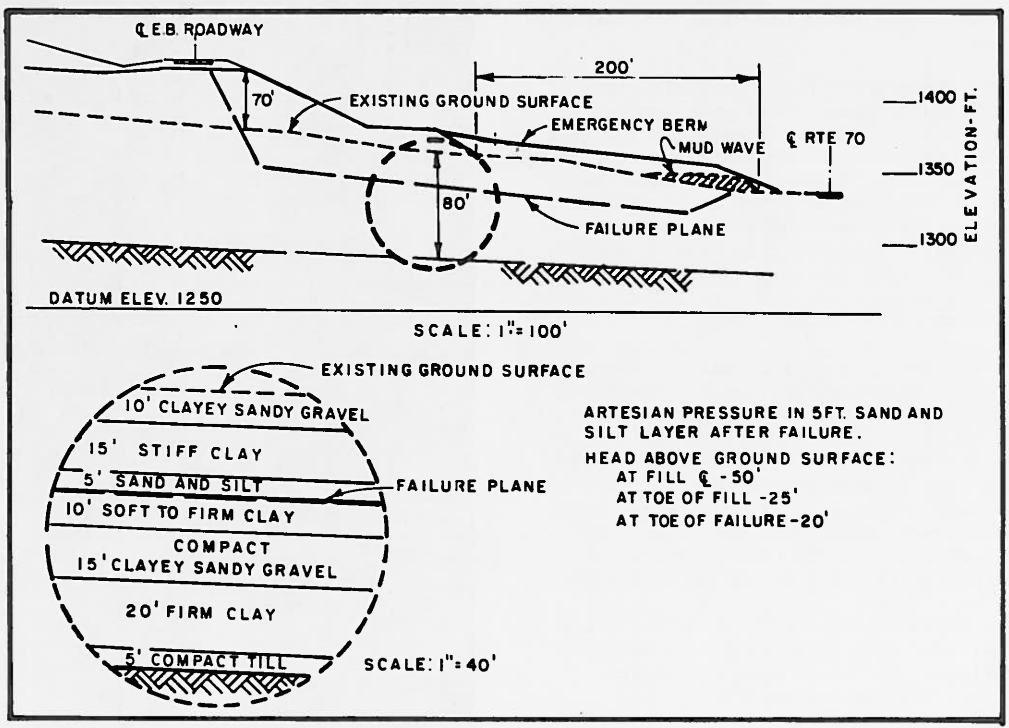


Figure 8. Embankment stability problem
Southern Tier Expressway, East of Hornell, New York

SUMMARY

This presentation has attempted to provide an overview of the approach to investigation, design, and construction of highway embankment foundation treatments in New York State. Hopefully the abbreviated case history accounts will provide the reader with some additional insight on soil behavior and methods of treatment which may be of use.

The majority of foundation problems in New York State are associated with lacustrine varved silt and clay deposits in valleys. The engineering properties of these clays are influenced by recent geologic events - lowered water tables causing dessication, erosion that removed former overburden loads, and ancient landslide movements that have reduced the in situ soil strength usually under existing slopes. Subsurface investigation programs in these deposits should determine the stress history of the varved silt and clay profile and attempt to locate old shear zones. Fig. 9 is an attachment to the paper that summarizes the variety of stress history conditions that can be expected. Some exploration testing and design guidelines are provided for each case. The stress history concept as related to strength and consolidation parameters has been developed in more detail by Ladd and Foott.(6)

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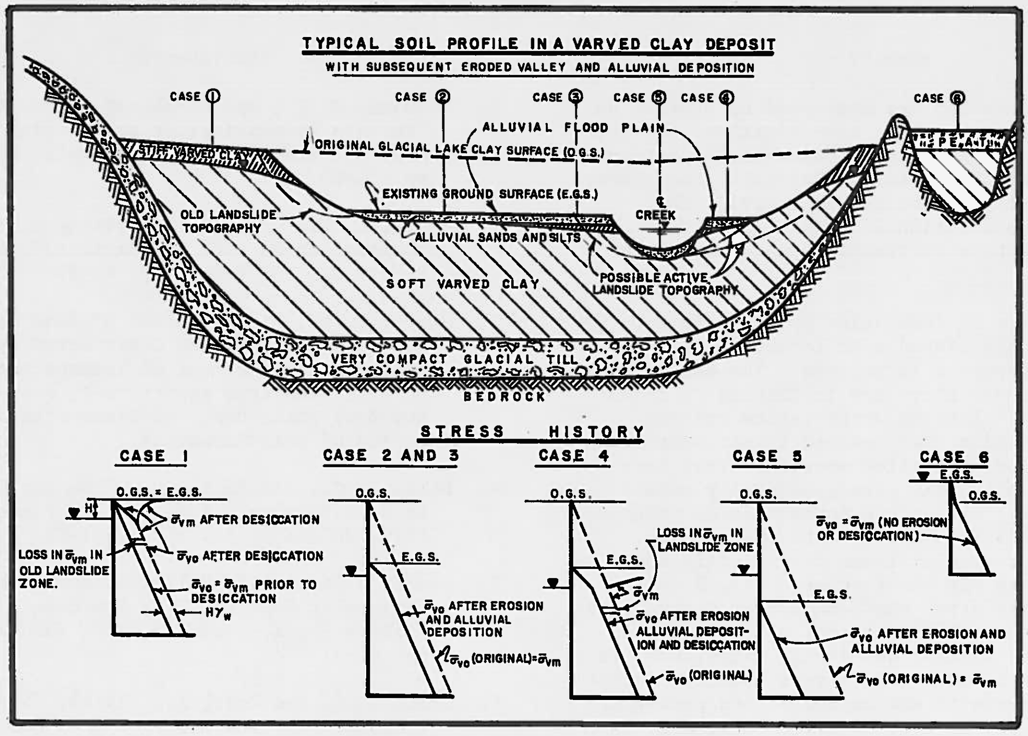

Figure 9. Typical stress history conditions found in varved silt and clay deposits - New York State

EXPLORATION AND DESIGN GUIDELINES

CASE 1

1 Progress continuous samples in upper 20 to 30 feet to accurately determine elevation of desiccated zone. Critical strength for stability is usually just below desiccated zone. Five foot error in depth assumption can reduce factor of safety significantly.

2 If the embankment is placed near the top of the clay slope or on the slope, analysis may be complicated because this slope was formed by previous undercutting action of creek. If this undercutting was rapid there will be deep landslide scars beneath the slope that have since healed. The beneficial effects of precompression on strength and consolidation along this zone will have been eliminated (i.e. the soil will act normally consolidated along this zone). In these zones, designing cuts and fills on bulk laboratory strength data can be disastrous. Look for usual evidence of old landslides (i.e. aerial photographs excellent preliminary source of information). For these reasons continuous tube sampling to a depth of 20 to 30 feet is recommended to find old failure scars along these slopes.

3 The $\bar{\sigma}_{vm}$ curve may occur in steps due to periodic drying (desiccation curves) of the glacial lake during the depositional stage. This will complicate settlement and stability analyses.

CASE 2

Determine if final pressure $\bar{P}_F$ will be greater than $\bar{\sigma}_{vm}$. If $\bar{P}_F < \bar{\sigma}_{vm}$ then stability and settlement problems should be minor.

CASE 3

1 Same as for Case 2

2 Additional fills or structures should not be placed on stream banks in zones of active movement. Design erosion protection to prevent future undermining of embankment.

CASE 4

In the zones of active landslide movements the shearing strength should be determined from the normally consolidated value at the existing overburden pressure $\bar{\sigma}_{vo}$.

CASE 5

A boring should be taken in the creek with continuous tube samples to a depth of 20 to 30 feet.

CASE 6

1 Clay is normally consolidated in areas where no erosion or desiccation occurred. Usually organic deposits cover the surface of the clay deposit. Clay has low strength due to small overburden pressure.

2 Organic deposits usually have to be removed and backfilled due to settlement considerations. Clay may also require removal because of low strength.

3 Obtaining good undisturbed samples is very difficult. Special strength tests are required to allow for sample disturbance and anisotropy. (i.e. CAU triaxial compression tests consolidated to 4$\pm$ times insitu stresses plus field vane shear testing.)

MOVEMENTS OF A NATURAL SLOPE AND AN EMBANKMENT
TWO CASE HISTORIES

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ABSTRACT. Two case histories of ground movement are presented; one case concerns naturally placed materials and the other concerns man-placed materials. The methods used to investigate the causes and nature of the movements are discussed along with conclusions and recommended corrective measures. One case history deals with land movement on a steep colluvial slope underlain by alluvial silts and silty sands resting on bedrock of slickensided red clay shale. The area is adjacent a large river and is the site of a major industrial facility. In this case, theretofore unnoticed movements occurring along failure planes within the alluvial silt and the upper few feet of bedrock were increased due to natural and man-made changes in the slope's configuration and drainage. The second case deals with a series of roadway embankments, constructed predominantly of shale, in which slow degradation of the shales over a period of ten years gradually reduced the stability of the embankments and caused excessive settlements. Several embankment failures occurred and several others were endangered due to infiltration of surface waters and percolation of groundwater along the critical failure surfaces.

INTRODUCTION

This paper presents two case histories of ground movement, one in a natural slope and the other in man-made embankments. Each case was studied in detail, using field investigations to define site conditions, such as topography, configuration and nature of the bedrock and overburden, water levels and pore water pressures, and nature and characteristics of the configuration of failure surfaces, along with rate and direction of movement. Extensive investigation was made in each case to determine the changes in conditions, whether by nature or man, that caused the movements.

Development of programs to gather and evaluate information to resolve the cause of movements and prescribe remedial measures in large natural slopes is usually more difficult than for man-made embankments, because the overall geometry and strength characteristic of the material of embankments is usually well understood in comparison to large natural slopes. Movements within embankments generally result from degradation of fill material or unpredicted high pore water pressures or water levels. Movements in natural slopes most commonly occur within colluvium, alluvium or glacial material, or a layered sequence of these soils. Due to their mode of deposition, the thickness and lateral extent of these deposits is difficult to determine. In addition, these deposits commonly consist of layers of silt, clay, sand and boulders with varying amounts of fine and coarse materials, each having different strength characteristics and permeabilities. These factors make determination of existing slippage planes, potential critical failure surfaces and evaluation of meaningful soil strengths difficult, at best. For these reasons, designs to prevent ongoing or anticipated movement within natural slopes commonly require more costly and detailed investigation, analysis and design than those made by man.

CASE I - MOVEMENT OF A NATURAL SLOPE

BACKGROUND

Figure 1 shows a major industrial facility constructed on a hillside between a steep valley wall and a large river. Principal man-made features of the site include plant structures, tailings ponds and main line and service line railroad tracks. With the exception of the main line tracks, these facilities were constructed during an approximate one-year period. Upon completion of the facility, solid waste was placed on the hillside at a rate of [redacted] tons/month, beginning at the western end of the disposal area.

31,000

Six months after beginning operations, displacement of the main line tracks adjacent the river was observed. The principal zone of movement was 350 ft east of Pond No. 3. Prior to the construction of

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the plant, this section of the track required only moderate maintenance of alignment. Three and one-half years after construction of the facility, minor disturbance of the ground surface was observed along the service line track and roadway uphill of the displaced main line track and minor movements were observed at many plant structures. At that time, investigation to determine the cause of movement and appropriate remedial measures were undertaken. The investigation included:

- o Determination of the regional and local geological conditions using published literature, aerial photography taken over the past 33 years and field mapping.
- o Establishment of subsurface conditions by drilling, sampling and testing soil and bedrock in the vicinity of hillside movement.
- o Installation of inclinometers to establish the rate, depth and direction of hillside movement.
- o Installation of piezometers to monitor groundwater levels.
- o Determination of the history of man's alternations to the ground surface from the time of construction of the main line track to the present.
- o Development of remedial steps to control groundwater levels and arrest further movements using data from field instrumentation.

In brief, the investigation revealed that the geological development of the hillside soils and bedrock resulted in a slope having a marginal factor of safety. Modifications of the slope by nature and man, even though relatively minor in themselves, resulted in hillside instability.

GEOLOGIC SETTING

Stratigraphy of the region includes late Mississippian Period and early Pennsylvanian Period sediments. These strata are nearly horizontally disposed with differences in attitudes of bedding planes from one outcrop to the next being minor. At the site, bedrock dips approximately eight degrees to the northwest. Examination of aerial photographs and topographic maps, field studies on foot and observation from the air indicate there is no significant faulting in the area.

The Mississippian Period Mauch Chunk Group, consisting predominately of gray, black and red clay shales, occurs from the surface of the river at El. 1055 ft to approximately El. 1400 ft. The slope of the erosional surface of this bedrock is approximately seven degrees toward the river. A closely spaced pattern of slickensides occurs in the red shale. Although there is some controversy concerning their genesis, many geologists ascribe the formation of these slickensides to the consolidation of shale occurring during diagenesis or periods of gentle uplift that formed the Appalachian Plateau.

At the site, as common in the region, the Pennsylvanian Period, Pottsville Group, rests disconformably upon the Mauch Chunk Group and consists predominantly of sandstone with some shale and coal. This disconformity is demonstrated in part by the juxtaposition of the Mauch Chunk Group shales and

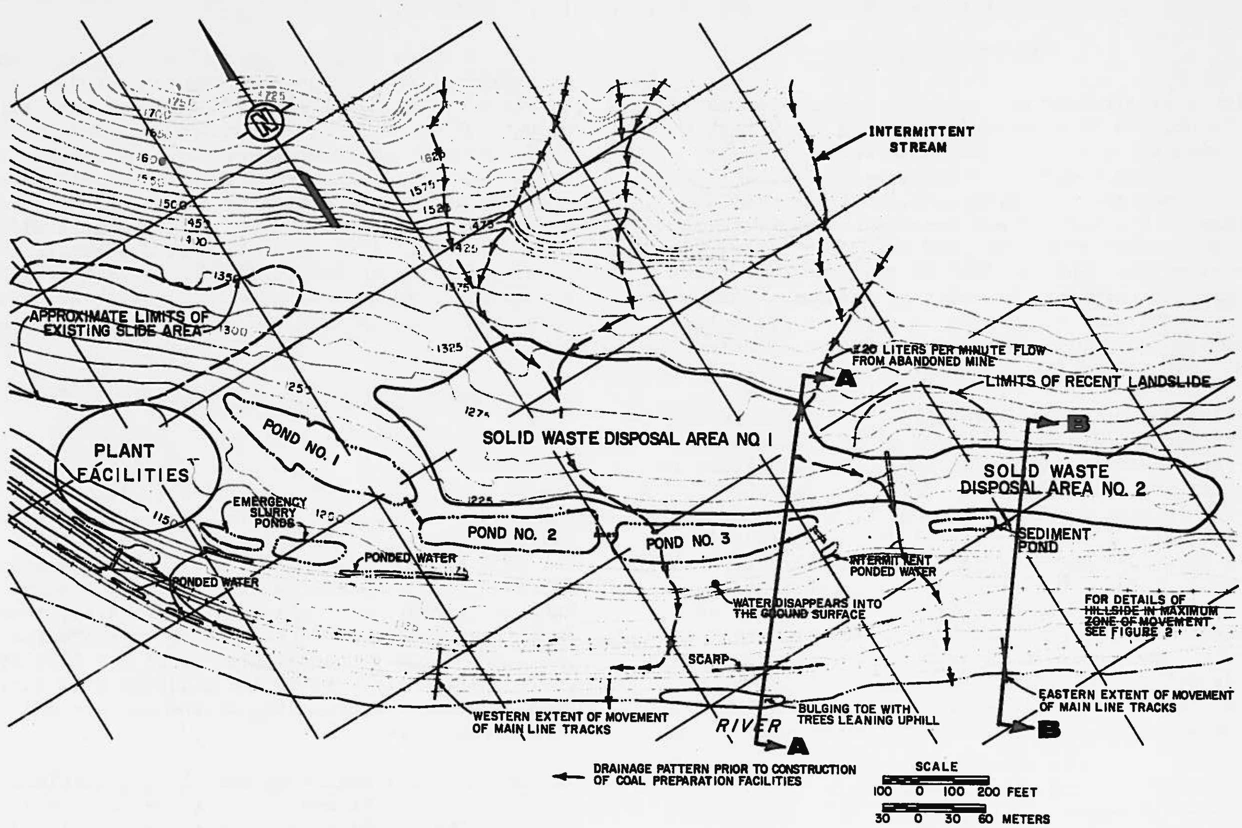


FIGURE 1 PLAN SHOWING SITE OF INDUSTRIAL FACILITIES

the Pottsville Group sandstone, as these two rock types formed in different depositional environments. Spring flow from the Pottsville Group sandstones is common in the valley wall above the industrial facilities. Valley slopes formed on Pottsville strata are typically 45 degrees and greater.

Colluvial soil, exceeding 105 ft of thickness, consisting of clays and silts with blocks of sandstone cover the valley walls below El. 1400 ft in most areas. At higher elevations, the predominant ground cover is talus of sandstone blocks. Well-sorted alluvial sands and silts resting on bedrock were encountered in borings sampled near the river. These represent remnants of terraces deposited when the depth of the river increased due to interim changes in base level or natural damming of the stream by landslide. On the gentle bedrock slopes, underlain by the Mauch Chunk Group shales, the combined thickness of colluvium and alluvium range from 21 to 120 ft. Natural ground surface slopes in the colluvial soil commonly range from 11 to 14 degrees (Figure 2).

FORMATION OF SUBSURFACE CONDITIONS AND MAN-MADE ALTERATION OF GROUND SURFACE CONDITIONS

Investigation to ascertain the nature of any hillside movement requires an understanding of the natural events that formed the surface and subsurface conditions as well as the history of ground surface alterations by man. Man's activities at the site were investigated through use of the literature, old aerial photographs and maps and personal interviews. Findings of this endeavor are given in Table I.

EVOLUTION OF THE SURFACE AND SUBSURFACE CONDITIONS. As the Appalachian Plateau was uplifted, the mature pattern of the present-day river incised itself through the sandstones of the Pennsylvanian Period, Pottsville Group, and subsequently into the stratigraphically lower and relatively softer shales of the Mississippian Period, Mauch Chunk Group. When the river encountered the very soft shales of the Mauch Chunk Group at El. 1400 ft, it was less contrained laterally and tended to broaden the floor of the river bottom, undercutting the Pottsville Group rocks. During this time, a stream flowed intermittently from the plateau above the site into the river adjacent the site. Today this stream flows during heavy rains and also carries the water discharging from an abandoned deep mine, operated from 1886 to 1925, 600 ft of elevation above the refuse disposal area.

As the river channel moved southwestward (laterally) to its present bed, a broad "cut" terrace was formed in the red clay shales of the Mauch Chunk Group on the north side of the river. The intermittent stream cited above cut a channel through the soft bedrock of this terrace. The inclination of the

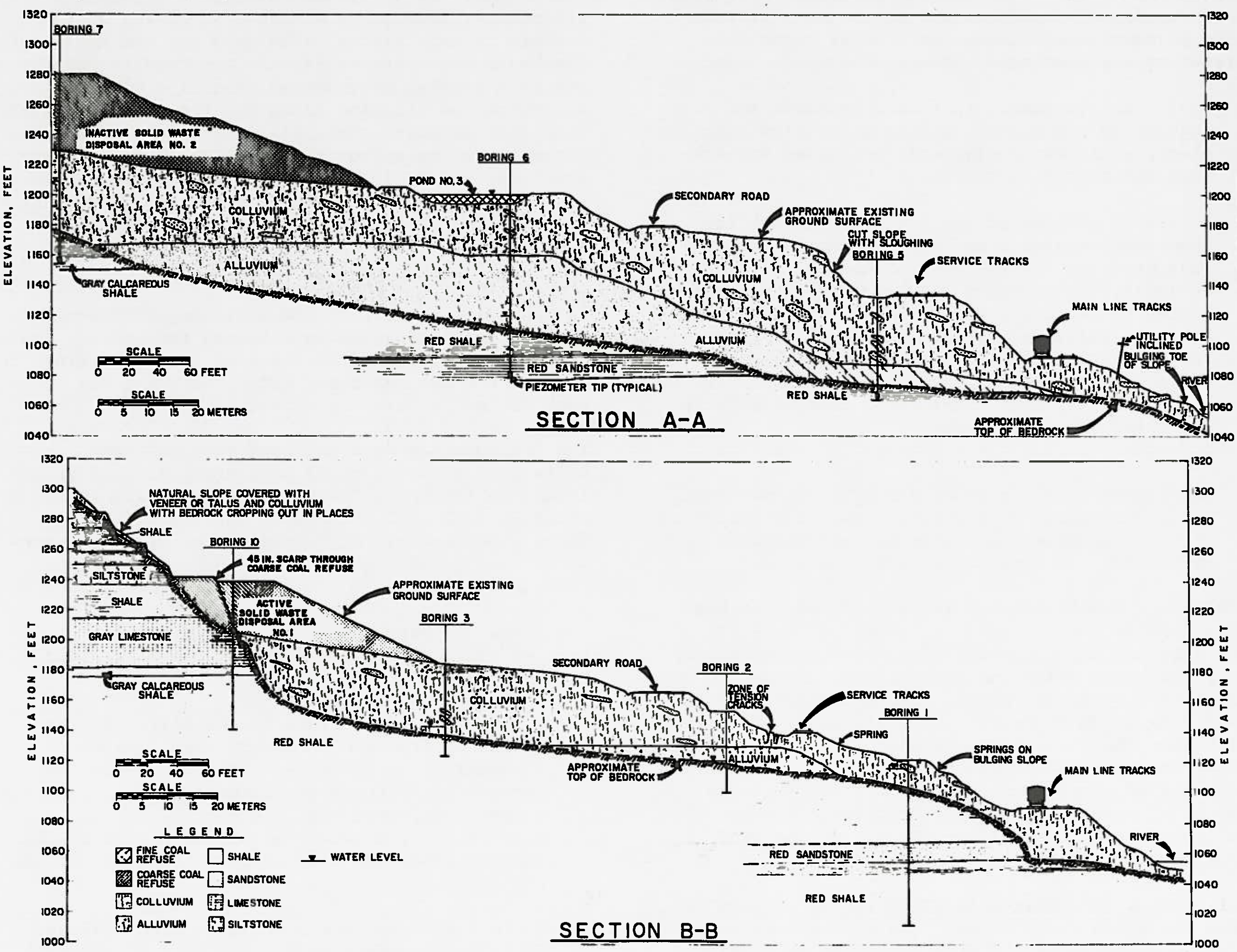


FIGURE 2 PROFILES SHOWING SUBSURFACE CONDITIONS IN THE ZONE OF MAXIMUM HILLSIDE MOVEMENT

erosional bedrock surface exceeds 23 degrees in places in the ancient channel and locally along the edge of the present-day river.

After the stream left the terrace, its elevation was nearly the same as that of the terrace, thus the terrace was frequently flooded. During flood stages, fine sand and silt were deposited on the gentle slopes of the terrace. Blocks of sandstone from the Pottsville Group and soil slid from the river valley walls onto the terrace forming colluvial deposits. With the passage of time, the river cut deeper into its present channel, the terrace was less frequently flooded and deposition on the terrace shifted from being predominantly alluvial to mostly colluvial. At the area of concern, the terrace was relatively wide and the colluvium accumulated to form an 11- to 14-degree slope. Downstream from the plant facility, colluvium has accumulated at the same rate. At that location, the bedrock terrace is relatively narrow; therefore, the same volume of colluvial material formed a relatively steeper slope. For this reason, the entire hillside northwest of the plant facilities exceeded its "angle of repose" and began failing prehistorically, whereas the colluvium on the hillside east of the plant facilities had accumulated on a relatively wider terrace, forming a slope that is presently near its "angle of repose."

SURFICIAL AND SUBSURFACE INVESTIGATIONS. Surficial and subsurface investigations in the area of hillside movement began three years after plant construction was completed. These studies included:

- Noting all pertinent surficial features and changes in the ground surface associated with the plant, solid waste disposal facilities and service and main line tracks.

- Obtaining samples of the soil and bedrock for laboratory analysis and development of subsurface profiles for the area of maximum hillside movement. Development of the profiles was augmented by mapping both surficial features indicative of hillside movement and man-made ground surface alterations in the zone of movement. Piezometers were installed in 17 borings. Inclinometer casings installed in 21 borings were also used as piezometers.

- Quantifying the nature of the hillside movement by determination of the rate, depth and direction of hillside movement. This was accomplished by installing and monitoring inclinometers.

SUBSURFACE CONDITIONS. A log typical of test borings drilled for the study is given in Table II. The alluvial soils consist of soft to medium-stiff, gray and brown silts and very well-sorted medium-dense to very dense, brown, fine to medium-grained sand. As shown in Profile-A of Figure 2, the alluvium near the river rests on bedrock of shale and sandstone from the Mauch Chunk Group. The colluvium consists of poorly sorted clays, silts, sand and blocks of sandstone from the Pottsville Group. The colluvium rests directly on bedrock in the area of Profile A of Figure 2.

All bedrock encountered in the drilling program is from the Mauch Chunk Group. The predominant rock type encountered at the site is broken to massive, soft to medium-hard, red shale. Scores of slickensides were observed in core recovered from each boring in this rock type. The red shales characteristically weathered rapidly and softened when in contact with air or water.

In the area of maximum hillside movement, indicated on Figure 3, massive, hard, fractured gray limestone overlies the red shales. The limestone occurs above El. 1150 ft. A 15 ft thick fractured massive, hard, fine to medium-grained red sandstone layer occurs within the red shales at El. 1110 ft. These limestones and sandstones, shown in Figure 2, contribute groundwater to the zone of contact of soil and bedrock.

A contour map of the top of bedrock shows that, in general, the contours of the top of bedrock parallel the natural ground surface contours. However, in the area of maximum hillside movement shown in Figure 3, an anomaly occurs in the top of bedrock. This previously cited feature is an ancient channel of a stream that flowed intermittently across the bedrock terrace.

Surface observation in zones where colluvium and bedrock are in contact indicate that groundwater springs are flowing from the sandstones and fractured limestones into the colluvium. Observation of piezometric levels within the colluviul and alluvial soils indicates that the groundwater level is near the top or rock in the upper portion of the colluvial slope. On the lower portion of the slope, groundwater levels are within 8 ft of the ground surface in many places. The gray and red shales of the Mauch Chunk are relatively impermeable, resulting in a perched groundwater condition within the colluvium and alluvium along the river. Piezometric data also indicate that this water flows toward the river along the approximate contact of bedrock and soil; therefore, it is possible that seepage channels above the interface have developed, resulting in high pore water pressures within the soil mass.

INCLINOMETER INSTALLATION

To monitor movements of the hillside, inclinometer casings were installed in selected boreholes. The casings were anchored at least 20 ft into bedrock to provide a fixed reference point, assuming the bedrock was not undergoing movement.

The inclinometer data gathered from selected boreholes over a period of 97 days are presented graphically in Figure 4. Rates of movement are summarized in Table III. Three of the seven casings shown were severely distorted by the hillside movements soon after installation, thereby precluding further monitoring.

The rates and depths of movement suggest that the area of maximum hillside movement may be divided into two zones:

1. A localized toe failure in the hillside area downslope from the secondary road. Rates of movement of inclinometers near and below the service line tracks (Inclinometers Nos. 1, 2 and 4) are consistent with the average 2.5 in. per month rate of movement of the main line tracks. In this area, the failure surface occurs within the colluvium.

2. A block failure in the hillside area upslope from the secondary road. The rate of movement in this zone is .5 to .75 in. per month, as determined from Inclinometers Nos. 10 and 3,

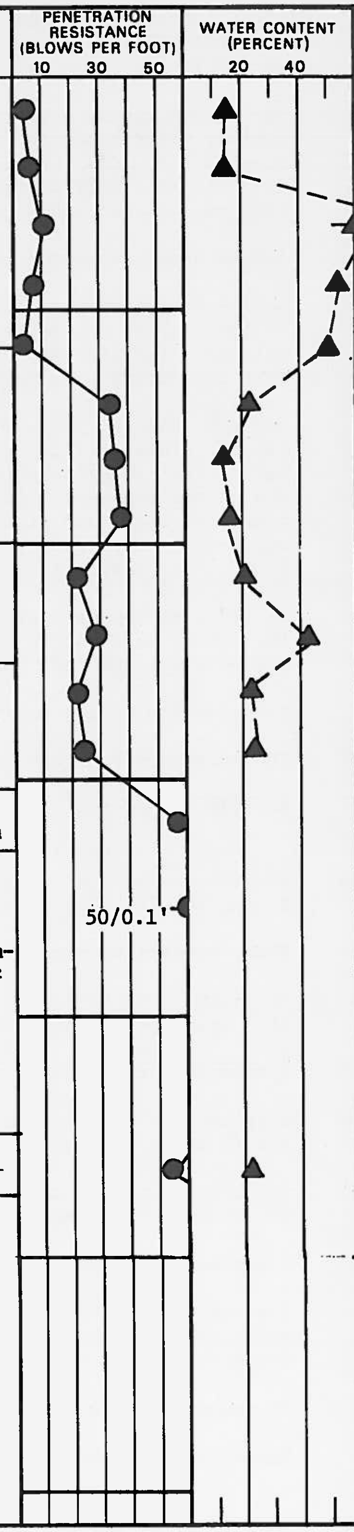
TABLE I. HISTORY OF
GROUND SURFACE ALTERATION BY MAN

Date	Alteration
1869-1873	The main line tracks were placed on the bank of the river. This construction included balanced cutting and filling of the natural slope. Near the site, construction of the railroad required filling an area approximately 400 ft long. Maximum depth of this fill is approximately 25 ft.
1886-1925	About 1886 mining began on the hillside 600 ft of elevation above the site. During this period, water began to flow from the mine. According to local persons, operations at the mine ended before 1925. United States Geological Survey topographic maps issued in 1913 and 1929 indicate that service tracks for the mine existed where the present-day service tracks are located. A small gage mine track ran from the end of the service tracks to the mine entrance.
1929	The secondary road was constructed.
1945	Aerial photographs indicate that activities at the mine had been abandoned for several years by 1945. Most of the residential structures indicated on the 1929 USGS quadrangle no longer existed. The vicinity of the mine entrance was sparsely vegetated and mine water was flowing over the valley wall in a channel diverting it away from the narrow gage track.
1957	Between 1945 and 1957, two coal seams had been stripped from the north side of the river valley. This activity had disturbed tailings at the old mine entrance and removed the vegetation from the mine entrance to a point 370 ft down the valley wall. Approximately 400 ft of the narrow gage track from the mine entrance downhill were no longer present. This change was caused by the strip mining which increased the flow of water into the valley near the mine entrance. Water from the mine and strip bench was flowing in a channel near the former location of the narrow gage track.
1969	By 1969, no houses remained at the site and mine water flowing over the steepest portion of the valley wall was in the same channel existing in 1957. This water flowed across the colluvial slope and into the river near the area of maximum hillside movement.
Jan. 1972	Construction of the existing plant facilities began.
Spring 1972	The secondary road was rerouted in the area of the plant facilities.
Jan.-Dec. 1972	Settling Ponds Nos. 1, 2 and 3 were constructed consecutively.
Apr. 1972 to Dec. 1972	Service tracks for the plant were constructed from the main line trackage to the vicinity of Boring No. 19. The rail bed was formed by balanced cutting and filling.
Jan. 1973	Plant production began with solid waste being placed in Disposal Area No. 1.
Winter 1972-Spring 1973	The last 1,400 ft of the single track service line was constructed. Approximately 650 ft of this trackage is built on 25 ft of fill.
Spring 1973	A diversion ditch above the plant facilities was excavated.
Summer 1973	Displacement of the main line tracks below Disposal Area No. 2 became evident and the service tracks began to require reballasting.
Spring 1975	The secondary road was rerouted from the area now occupied by the toe of Disposal Area No. 2. Solid waste began to be placed in Disposal Area No. 2.
Fall 1975	A sediment pond was constructed below Disposal Area No. 2.
July 1976	A scarp with 3.5 ft of vertical displacement developed in Disposal Area No. 2. Efforts to maintain the service tracks below Disposal Area No. 2 were given up due to rapid hillside movement.
Aug. 1976	Placement of solid waste in Disposal Area No. 2 was terminated.
Aug.-Dec. 1976	Subsurface investigation conducted.
Spring 1977	The entire slope west of Settling Pond No. 3 was regraded and seeded. Drainage ditches for the area were lined with rubber.
Summer 1977	Installation of horizontal drains began.

TABLE II

LOG OF A TYPICAL TEST BORING IN THE ZONE OF MAXIMUM HILLSIDE MOVEMENT

ELEV (FEET)	DEPTH (FEET)	SAMPLE	DESCRIPTION	U.S.C.S.
1180	5, 10	S-1, S-2, S-3, S-4, S-5	VERY LOOSE TO MEDIUM DENSE BROWN SHALE FILL PLACED 47 YEARS PRIOR TO PLANT CONSTRUCTION 11.5'	
1170	15, 20	S-6, S-7, S-8	DENSE FINE TO MEDIUM BLACK-BROWN TO GRAY SILTY SAND--SOME SANDSTONE FRAGMENTS 20.0'	sm
1160	25	S-9, S-10	MEDIUM STIFF TO STIFF GREENISH-GRAY SANDY SILT SOME SHALE AND SANDSTONE FRAGMENTS 25.0'	ml
	30	S-11, S-12	STIFF TO VERY STIFF DARK GREEN AND BLACK SILTY CLAY 30.5'	cl
		S-13	VERY DENSE GREENISH-GRAY SILTY FINE TO MEDIUM SAND 33.0'	sm
1150, 11-2-76, 1140	35, 40, 45	S-14	GRAY FINE-GRAINED SANDSTONE SOME SILTY-CLAYEY FINE SAND. THIS ROCK IS COLLUVIAL. 45.0'	sm-sc
		S-15	SOFT TO STIFF GRAY SILTY CLAY - SOME SHALE FRAGMENTS 47.5'	cl
	50	RUN NO 1	BEDROCK ENCOUNTERED AT 47.5' NOTE: RECOVERY OF RUN NO. 1 IS 51% RQD IS 17%	
1130	55, 60	RUN NO 2	SOFT TO HARD BROKEN TO MASSIVE UNWEATHERED TO SLIGHTLY WEATHERED WELL-BEDDED RED SHALE NOTE: RECOVERY OF RUN NO. 2 IS 93% RQD IS 51%	
1123.04	61.5		BOTTOM OF BORING 61.5'	



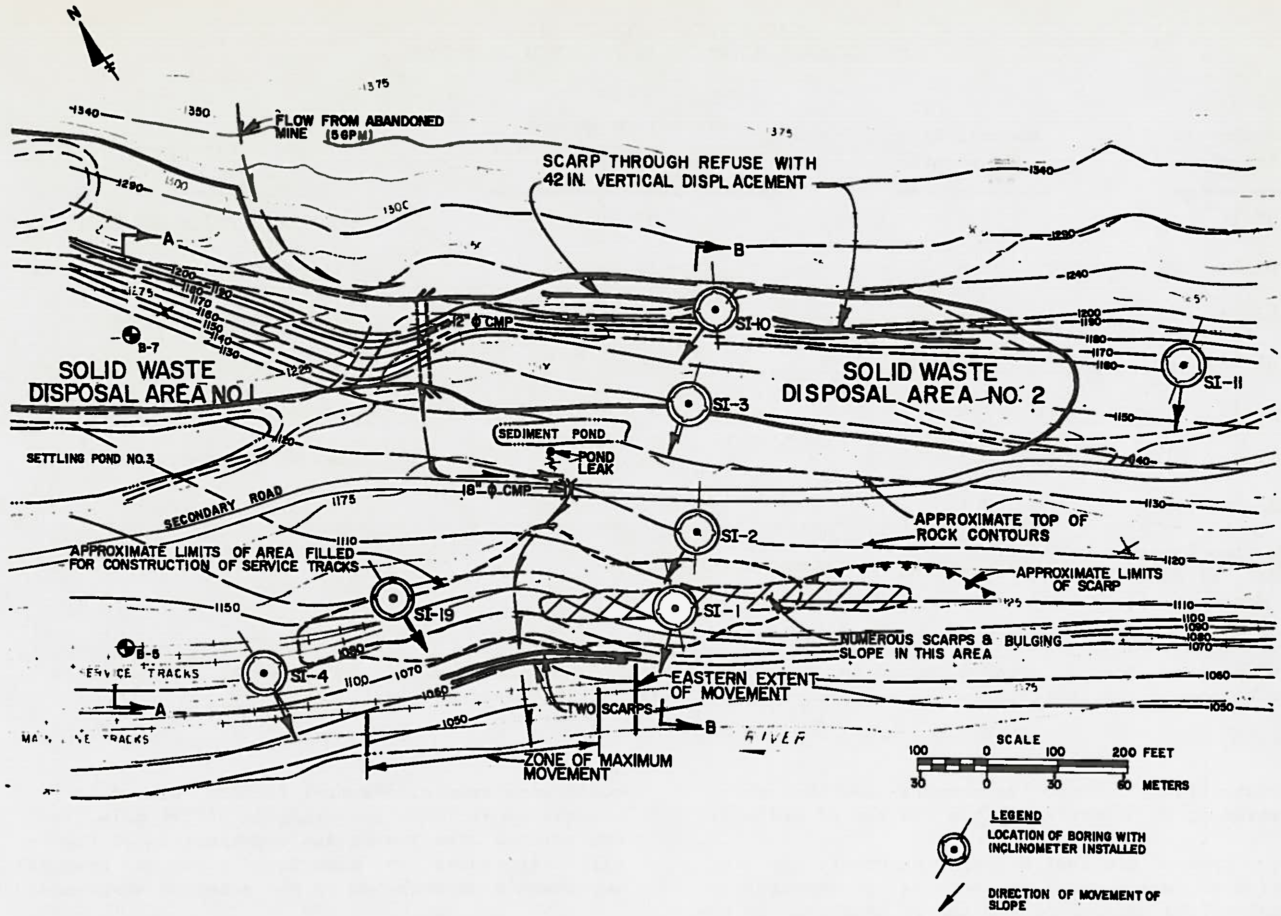


FIGURE 3 PLAN OF THE ZONE OF MAXIMUM HILLSIDE MOVEMENT SHOWING INCLINOMETER LOCATIONS

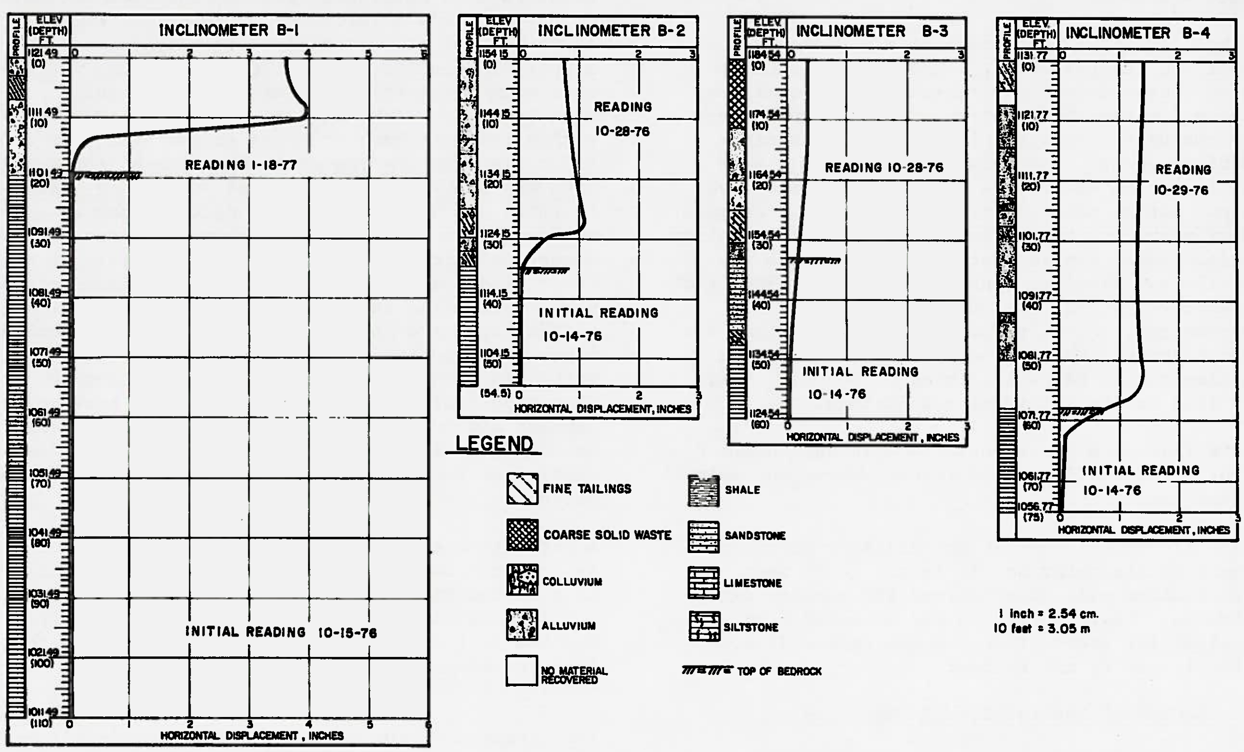


FIGURE 4 GRAPHIC REPRESENTATION OF MOVEMENT OF SELECTED INCLINOMETER CASINGS

TABLE III. AVERAGE RATES
OF HILLSIDE MOVEMENT AT SELECTED LOCATIONS

Inclinometer Designation	Average Rate ([illegible]/month) cm	Observation Period Through January 17, 1977 (days)*	Locations
SI-1	3.2	97	Between Service and Main Lines
SI-2*	5.6	14	Above Service Line
SI-3*	2.0	14	Above Secondary Route
SI-4*	8.1	14	Above Service and Main Lines
SI-10	1.4	97	Disposal Area No. 2
SI-11	0.5	97	East of Disposal Area No. 2
SI-14	0.5	150	A Plant Structure
Main Line Tracks at Zone of Maximum Movement	6.5	1,100	

*Inclinometers Nos. 2, 3 and 4 became too deformed to permit the passage of the inclinometer instrument to the bottom of the installation 14 to 28 days after their initial readings were taken.

respectively. These instruments show that the movement is occurring within the top of bedrock.

In the area of greatest hillside movement, the direction of inclinometer movement is approximately normal to the contour of the top of bedrock. In the direction of movement converges causing both compression and local increase in the elevation of the service tracks.

Inclinometer No. 11 is moving nearly directly downhill at 0.2 in. per month. This installation is located approximately 600 ft east of Disposal Area No. 2, a considerable distance from the conditions contributing to the slope failure in the maximum zone of movement. The relatively slow rate and direction of movement also indicate that its movement may not be related to the previously discussed area of movement, but rather to hillside instability associated with the excavation made near the toe of the colluvial slope to construct the final 900 ft of the service line tracks. Unlike the other six inclinometers, No. 11 shows movement well above the top of bedrock. There is no displacement of the main line tracks below Inclinometer No. 11. These facts lead to the following conclusions:

1. The failure plane associated with Inclinometer No. 11 is shallow and surfacing above the main line tracks.

2. The principal cause of the hillside movement near Inclinometer No. 11 is the 26 ft deep excavation made to construct the service line tracks. Contribution to the movement from activities associated with the refuse disposal facilities is not evident.

CAUSES OF INSTABILITY OF THE HILLSIDE

There was no one single cause of the hillside movements and the distress to the main line tracks, service tracks, secondary road and the refuse disposal embankments. Several factors related to natural conditions, construction of the main line and service line tracks and construction of the plant structures, and associated ponds and disposal embankments contributed to the observed movements.

The natural geologic development of the hillside deposits of colluvium and alluvium at the site resulted in a marginally stable condition exclusive of any of man's alterations. The thick, red clay shale bedrock characteristically weathers easily to a soft clay and is somewhat expansive when in contact with water or air. The shale is heavily slickensided, which suggests that minor movements within the rock have occurred in geologic time considerably reducing the shear strength of the weathered top of bedrock. As stated previously, these features are intrinsic to the rock and not associated with the recent land movement. Stress release associated with erosion of overlying strata in the river valley and weathering along the slickensides have reduced the shear strength of this rock. Where slickensides are abundant, weathering of bedrock is severe and the failure plane of the slope occurs within the upper portion of the top of bedrock. This hypothesis results from evaluation rock core samples and inclinometer data that show movements to be contained in both the alluvium-colluvium mass sliding on top of bedrock and within the top of the bedrock.

A periodic rise and fall of the river levels causes an increase in pore water pressures with a decrease in shearing resistance along the toe of the hillside soils, possibly producing localized instability of the toe of the slope and triggering movements that progress deeper and upslope, embodying a much larger portion of the hillside materials.

Infiltration of surface runoff during rainfall periods causes an increase in the groundwater levels. Due to past erosional conditions, the contours of the top of bedrock slope steeply toward the river and in the area of greatest movement. Consequently, groundwater tends to flow toward the zone of maximum movement. Inclinometer and piezometer data have shown excellent correlation of rising ground water levels and slope movement with heavy rain. The abandoned mine workings upslope of the plant facilities have been a continuing source of surface water into the site. Although this factor cannot be strictly categorized as a natural condition which could contribute to instability of the hillside, it did exist prior to construction of the present-day facilities.

Construction of the main line tracks required excavating, filling and general alteration of the surface water flow patterns. All of these modifications most likely induced localized low factors of safety in the toe area of the hillside and contributed to instability of a much larger mass upslope.

Construction of the service line tracks required placement of fill in the gulley of the intermittent stream and suspected area of past sliding. This gulley area is directly over the ancient stream channel incised in the bedrock and coincides with the zone of maximum recorded movements. The fill embankment added weight to the slope at the upslope extent of the bedrock anomaly, and since proper underdrains were not installed or the foundation soils properly prepared during embankment construction, a toe failure developed. Movements in this area essentially caused unloading of the toe of the main slope, resulting in movements propagating uphill. A similar condition apparently occured in the area just above the service tracks approximately 600 ft east of Disposal Area No. 2. Here, unloading of the toe resulted from excavating the toe of the slope to construct the easterly 260 ft of the service tracks. Movements of 0.2 in./month were observed at this location.

Many ground surface alterations were made during construction of the plant facilities. Vegetation and topsoil were removed from the hillside for construction of the plant and disposal area, thereby exposing soils to increased infiltration from rainfall. Placement of solid waste on the hillside added weight to the top of the hillside slope. Surface water drainage patterns were modified for construction of the plant, main line and service line tracks, and solid waste embankments. Several large, settling ponds, emergency ponds and a sediment pond constructed in the area may have contributed to increased groundwater levels.

REMEDIAL MEASURES

Remedial measures have been designed to improve the hillside stability at minimal cost and disruption to plant and railroad operation and are being implemented in a staged sequence, with close monitoring of the movements to determine the degree of success prior to initiating other more costly measures. Measures to control the movements were to be implemented in the following order:

1. Regrading and seeding the slope and providing proper surface drainage in rubber-lined ditches;

2. Lowering the groundwater level within the slope using horizontal drains;

3. Removing the settlement ponds;

4. Unloading the upper portions of the slope;

5. Flattening the overall slope; and

6. Buttressing the toe of the slope.

At this writing the first step of the remedial work has been implemented on portions of the slope and horizontal drains have been installed beneath the plant structures. The drains were drilled through the colluvium and alluvium into bedrock resulting in a groundwater level decrease ranging from 19 to 34 ft. Prior to installation of the drains, movement at plant structures were appropriately 0.2 inches/month. No movement has been observed at these structures since drain installation 3 months ago. Flow from all 12 horizontal drains at the plant area averages approximately 28 gpm. During periods of rain, flows increase to approximately 40 gpm. Horizontal drains are presently being installed in the area of maximum hillside movement. In this area, the average flow from individual drains is 0.5 gpm in dry weather and 3.5 gpm in rainy periods.

CASE II - MOVEMENT OF AN EMBANKMENT

BACKGROUND

An investigation was made of a five-mile section of interstate highway in the Midwest which began developing embankment stability and settlement problems in the early 1970's, approximately ten years after its construction. The highway traverses an area of rolling hills and streams, having a maximum relief of 150 feet. Thirteen embankments crossing minor streams and hollows were within the area investigated. The depth of fill in the affected embankments varied from 7 to 110 feet. The fills are a mixture of shale, residual soils and limestone from the intervening highway cuts. Major deep-seated slides had occurred in three of the embankment slopes and extensive surficial sloughing had occurred in three others prior to the investigation. By 1975, all of the embankments had settled noticeably; from 2.5 inches in a fill seven feet deep to 34 inches in a fill 90 feet deep. Average settlements were one to two percent of the fill depth. Previous stability corrections included removing the slide material, providing subsurface drainage and keying well-compacted replacement fills into bedrock or competent residual soil.

DESIGN FEATURES AND CONSTRUCTION PROCEDURES

When the highway was built in the early 1960's, a roadway soil survey was not a normal part of the design phase of a highway project. Therefore, embankment designs were not tailored to the subsurface conditions. Preparation of embankment areas consisted generally of stripping the topsoil and placing fill directly on the in situ soil. Deep benching of the fill into the soil or bedrock was not a routine practice. Although culverts were provided to carry surface waters beneath the embankment areas, drainage of subsurface water was usually not provided.

The standard highway specifications employed at the time of construction did not include compaction specifications for shale/limestone mixtures. It was reported that most of the shale fill was treated as "rock," because it was occasionally mixed with hard limestone, and the rock fill specifications allowed placement by end-dumping and bulldozing the materials in lifts up to four feet thick. Under such conditions, limestone slabs would tend to create bridging beneath the compaction equipment, leaving uncompacted zones or voids in the fill. Even without the limestone slabs, the compactive energy would not have been sufficient to compact such thick lifts. As a result, it was suspected that loose areas or voids permitted surface and subsurface water to percolate through the fill. The longitudinal slope of the highway was only three percent or less, and a significant portion of the rainwater falling within the 60-foot-wide grass medial strip and running off the pavement did not reach the widely spaced drop inlets along the center. Thus, rainwater and snowmelt could enter the fill. Settlement of the embankments would further inhibit longitudinal drainage. It was also suspected that embankment settlements had caused cracking and leakage of the storm drainage culverts which collect water from the drop inlets.

GEOLOGY

The embankment stability and settlement problems that developed along this section of the highway were related in part to the geology of the area. The stratographic sequence of the underlying bedrock and the weathering characteristics of the rock strata determined the topography of the area, the pattern of surface and subsurface water flow, and the engineering properties of the in situ soils. Reportedly, the region of this highway is one of the most slide-prone areas of the state. Natural slopes on the hillsides vary from approximately 25 to 50 percent and show little evidence of instability. However, many cut slopes in the soil overburden along the highway in the general vicinity contain slump-type failures in slopes of 50 percent (2:1) or greater. Highway cuts in rock up to 100 percent (1:1) show little evidence of instability, but differential weathering of the exposed rock causes occasional maintenance problems.

Soil along this section of the highway is derived from decomposition of the calcareous shales and fossiliferous limestones of Ordovician age. Shales comprise 65 to 70 percent of the bedrock and are relatively impervious, having few open joints. X-ray analyses of the shales indicate various amounts of illite, kaolinite, chlorite, and mixed-layer clays including montmorillonite. Although illite is a stable, nonexpansive clay mineral, chlorite and the more complex mixed-layer clays are often expansive in the presence of water; one of the reasons for the shales' high susceptibility to weathering. Laboratory test results demonstrated that the effects of weathering cause loss of cohesion in the shale with little effect on the angle of internal friction. Only after shear strains have produced slickensides in the material is there an appreciable loss of friction angle.

The limestone occurs in thin strata and is well jointed, typically breaking out into slabs a foot or less across and a few inches thick. Water movement through the bedrock, which is nearly level, is confined primarily to the open joints in the limestone. Data from piezometers in the limestones showed that, in many cases, water flows through the limestone under artesian head. Vertical water movement is slight due to the very low permeability of the overlying residual soils and the in situ shales. Open highway cuts through these rocks show significant differential weathering between the shales and limestones and appreciable erosion of the fine-grained detritus formed by decomposition of the shale.

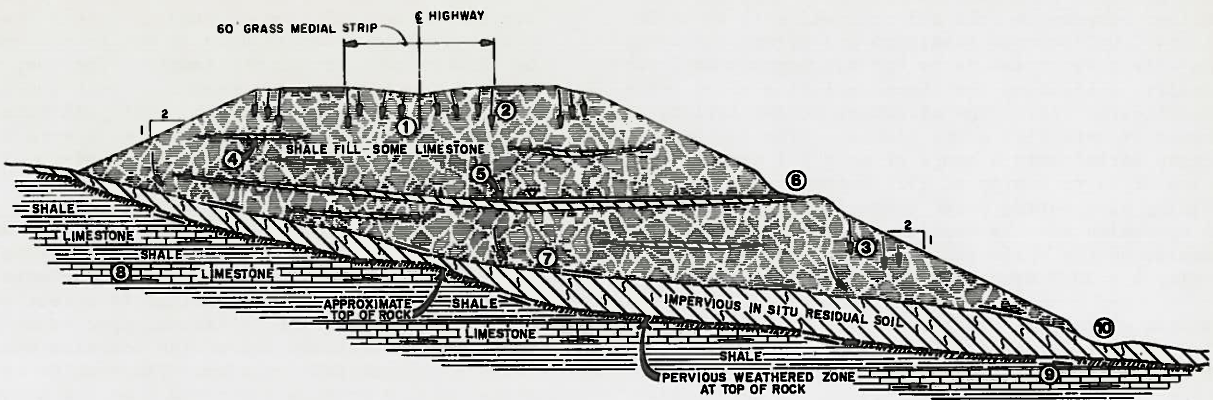
Water moving through the residual soils and colluvium on the natural slopes tends to follow irregular paths. One zone of soil may be dry while other nearby zones are saturated. Planes of weakness may develop anywhere, but the zones of least strength are usually at the soil-bedrock interface and at the fill-soil interface, where water moves readily downslope.

FIELD AND LABORATORY INVESTIGATIONS

A field and laboratory investigation was commissioned by the state highway department to determine corrective measures to prevent future embankment failures and, if possible, to reduce embankment settlements. Of the embankments under study, only four exhibited characteristics common to previous slides. Three were side-hill fills and one was a deep, cross-valley fill that had settled almost three feet and had extensive sloughing on the slopes. The other designated embankment areas were essentially cross-valley fills exhibiting varying degrees of settlement, surficial sloughing and erosion at the toe of slope, but no outward signs of major distress. Studies of the relative sizes of the various fill embankments and the characteristics of adjacent excavation areas from which the fill materials were obtained suggested that the aggregate properties of the fill in the various embankment areas along the highway might be similar, with the only variations being those caused by differences in the relative amounts of shale, limestone and residual soil in the fill. Accordingly, only one embankment was studied extensively, using borings, piezometers and inclinometers. This area was chosen because the subsurface topography and surficial evidence bore the greatest resemblance to other slide areas, and because its proximity to both deep and shallow cuts promised to provide an aggregation of fill materials most representative of this entire section of highway. Borings and instrumentation were also installed in eight other embankment areas, and optical surveys to monitor vertical and lateral movements were performed in all of the embankment areas.

Investigation of surface conditions revealed significant cracking and erosion at the edges of the concrete pavements, even in "cut" areas, indicating variable subgrade conditions. Concentrations of surface runoff from the traffic lanes entered the fill at these points. Also, many failures were observed in the concrete toe gutters (paved side ditches) along the bottom edges of the embankments. Much of the surface water intended to be collected by these gutters flowed beside and beneath them, causing significant erosion. In many places, the gutters had cracked or fallen into the erosion channels.

Subsurface investigation verified that the fill was a heterogenious mixture of gray and brown clay shale and limestone fragments with occasional zones of residual soil fill. The soil fill did not appear to exist in continuous layers, and there did not appear to be an impervious soil "encasement" near the surface or on the slopes to prevent infiltration of water. Although the relative amounts of shale and limestone in each sample varied, on the average the shale constituted approximately 70 percent of the fill, and the limestone fragments were essentially floating in a matrix of weathered shale and residual soil fill.



① — WATER INFILTRATES THROUGH GRASS MEDIAL STRIP.
② — CONCENTRATIONS OF WATER ENTER FILL AT EDGES OF PAVEMENT.
③ — LESS WATER ENTERS FILL BENEATH EMBANKMENT SLOPES.
④ — DISCONTINUOUS POCKETS OF RESIDUAL SOIL FILL.
⑤ — OCCASIONAL CONTINUOUS LIFT OF IMPERVIOUS RESIDUAL SOIL FILL CAUSES PERCHED WATER.
⑥ — PERCHED WATER EMERGING ON SLOPE CAUSES SURFICIAL SLOUGHING.
⑦ — WATER CONCENTRATES AND FLOWS ALONG ORIGINAL GROUND SURFACE.
⑧ — WATER FLOWS THROUGH LIMESTONE JOINTS UNDER PIEZOMETRIC HEAD.
⑨ — WATER CONCENTRATES AND FLOWS THROUGH PERVIOUS ZONE AT TOP OF ROCK UNDER PIEZOMETRIC HEAD.
⑩ — LOSS OF TOE SUPPORT DUE TO STREAM EROSION OR GUTTER FAILURE.

FIGURE 5 SCHEMATIC PROFILE OF A SIDE-HILL FILL EMBANKMENT

The in situ residual soils underlying the embankments varied in thickness from five to fifteen feet, with thicker soil occurring in the valleys, and consisted of medium stiff to stiff, low plasticity clayey silts and silty clays with floating fragments of limestone. Clay and silt size particles form the matrix; therefore, the soils are relatively impervious. At the interface of bedrock and residual soil a permeable zone of weathered bedrock, approximately two feet thick, occurred in most of the borings and test pits. The zone of weathering at the top of rock collects groundwater flowing from various limestones seams and conducts it to the valley bottoms. Piezometer data also indicated that the groundwater flowing in the limestones and the zone of weathering had a slight artesian head. Perched water was encountered in two of the embankments and appeared to be localized. The perched water is shallow relative to the depth of the fill, therefore, it is probably recharged by surface seepage. A schematic cross-section through a typical side-hill fill embankment is illustrated in Figure 5.

Laboratory testing indicated that the residual soils beneath the embankments possessed a higher degree of plasticity, finer grain sizes and higher water contents than the average sample of fill. However, the liquidity of the residual soil was not significantly higher than that of the fill. In both cases, the in situ water contents were below the plastic limits. A comparison was also made of the activity of the residual soil and the fill (i.e., the ratio of the plasticity index to the clay fraction). The data showed low, but virtually identical, activity indices for the fill and the residual soil (0.6 ± 0.25), apparently due to the common mineralogy of the parent clay shales. Comparisons of these indices for the fills and residual soils at the various embankment areas also indicated no significant difference along the highway.

Since it appeared from the statistical analyses of index properties that the matrix material in the various embankments and underlying soils throughout the project area were quite similar, it was expected that the average shear strength properties would also be similar. This hypothesis was verified by shear strength testing on various samples of fill and residual soil from different embankment areas. It was also noted that the matrix material would control the aggregate strength of both the fill and the residual soil, since the larger rock fragments were "floating" in the matrix. It was therefore concluded that the shear strength of the fill material and the residual soils would be similar in all of the embankments in the project area and could be established by testing of the matrix materials. A schematic view of the aggregate strength behavior is depicted in Figure 6.

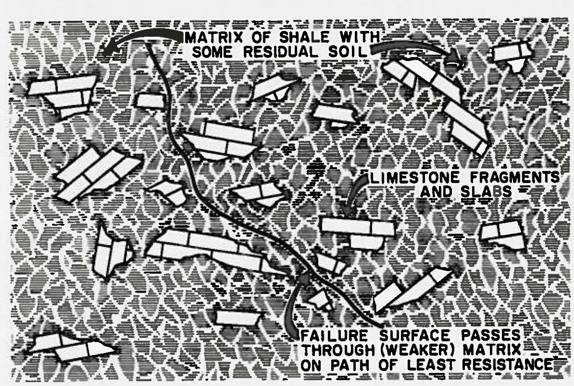


FIGURE 6 SCHEMATIC OF MATRIX MATERIAL CONTROLLING AGGREGATE STRENGTH OF FILL

Consolidated undrained triaxial shear tests with pore pressure measurements ($\overline{CU}$) were conducted on various samples of fill and residual soil in their natural, undisturbed condition and after remolding. The effective stress paths for all samples were very similar, exhibiting the characteristics of an over-consolidated clay. The effective stress failure envelopes were also quite similar. The measured $\bar{\phi}$ angles varied over a range of only 1.7 degrees, from a low of 24 to a high of 25.7 degrees. After remolding at constant water content and recompacting to approximately the density in the embankments, samples of the shale exhibited a higher friction angle, $\bar{\phi}$ = 30.8 degrees, but virtually no cohesion.

Samples of in situ weathered shale from block samples were also tested. The effective friction angle on inclined failure planes was only 28 to 30 degrees. However, the unit cohesion was much higher than the more decomposed shales in the fill.

Upon completion of each of the triaxial shear tests on natural samples cut from the blocks of weathered shale, each sample was completely remolded, reconsolidated and retested in $\overline{CU}$ tests to establish the structural sensitivity of the clay of which the parent shale is composed. The angles of internal friction measured in these tests were virtually identical to those measured on samples of shale fill. However, the cohesion of the weathered shale had been completely destroyed by remolding.

The results of the shear testing indicated that the unit cohesion of the fill, the residual soil and the in situ weathered shale could vary considerably, depending on the degree of weathering or mechanical breakdown that occurred prior to testing. However, $\bar{\phi}$ was surprisingly stable. It was also learned from triaxial shear testing of a sample of actual shear plane material from an area of existing downhill creep, that the effective friction angle begins to decrease only after measurable shear strains occur.

The effective failure envelopes for the shale fill and residual soils established the peak strength of these materials. To investigate the residual shear strength and anisotropy, a series of very slow, cycled, direct shear tests were conducted on samples of undisturbed weathered shale from test pit block samples. The direct shear tests were run at a rate of displacement of approximately 1/2 inch per day to permit full dissipation of any pore water pressure created by the shearing. Thus, the measured total stresses on the sample were also the effective stresses. The results of a typical cycled direct shear test on the weathered shale are shown in Figure 7 as a plot of the stress ratio, τ (shear stress/normal stress) versus cumulative shear displacement. Each sample typically achieved a peak shear stress and, with continued strain, the stresses on the failure plane decreased appreciably. Each test was conducted until no further reduction of strength was apparent.

Results of the cycled direct shear tests are presented in Figure 8. The failure envelopes are based on regression analyses of the data. High correlation coefficients indicated good correlation of the data from various samples. The peak effective strength parameters are $\bar{\phi}$ = 23.8° and $\bar{c}$ = 1,520 psf. For comparison, the Mohr effective failure envelopes from three series of $\overline{CU}$ triaxial tests on inclined planes are also shown in Figure 8. These results indicated only slight anisotropy in highly weathered samples of the shale.

Comparison of the mean residual failure envelope to the mean peak failure envelope in Figure 8 illustrates substantial loss of strength after shear failure. This strength loss is due to the creation of slickensides within the sample. The test simulates a downhill creep movement through the weathered shale at the top of rock. The peak strength parameters from the direct shear tests were used in the stability analysis for the two-foot-thick zone of primary weathering at the top of rock.

It is noteworthy that the loss in strength due to excessive creep movement is much greater than that caused by complete remolding of the weathered shale. After remolding, which is similar in effect to slaking and weathering by natural processes, the shale loses virtually all of its cohesion but retains high internal friction. The results of the shear testing program indicated that the natural weathering processes within an embankment may destroy the structural integrity of the shale and produce a loss of cohesion but this degradation does not materially affect the angle of internal friction. Large shear movements are required to lower the friction angle.

EMBANKMENT STABILITY ANALYSES

Sidehill fills are more susceptible to stability failure than cross-valley fills, primarily because the embankment slope and the underlying original slope are closer to being parallel than in any other type of embankment. Also, the cross-section of a side-hill embankment is more uniform in a longitudinal direction. The most critical cross-section in a sidehill fill is usually where the slope of the underlying ground surface is steepest. The most critical cross-section in a cross-valley fill is usually at the deepest portion of the fill, but the most critical cross-section is usually flanked by cross-sections of greater stability. Therefore, analysis of the most critical cross-section in a cross-valley fill will usually underestimate the stability of the embankment.

Based on field surveys and considerations of underlying topography, nine embankment cross-sections were analyzed, five-side hill fills and four cross-valley embankments. In all cases, effective stress analysis was used; that is, it was assumed that all failures would occur under drained conditions. Each slope was studied for three sets of conditions:

- Case 1 - As-measured slope conditions, assuming the peak effective strength parameters measured in the laboratory.
- Case 2 - Loss of cohesion in the fill and the weathered shale at the top of rock due to decomposition of the shale.
- Case 3 - Progressive failure, assuming as-measured peak strength for the fill and residual soil and as-measured residual strength in the weathered shale at the top of rock.

The results of a typical stability analysis are shown in Figure 9 for the most critical cross section in a deep sidehill fill. For each of the cases analyzed at this cross-section, the most critical failure surface is shown, along with the corresponding safety factor. The Case 1 safety factor of 1.14 is the best estimate of the existing stability of this slope. Theoretically, this slope has only a small reserve of strength to prevent a

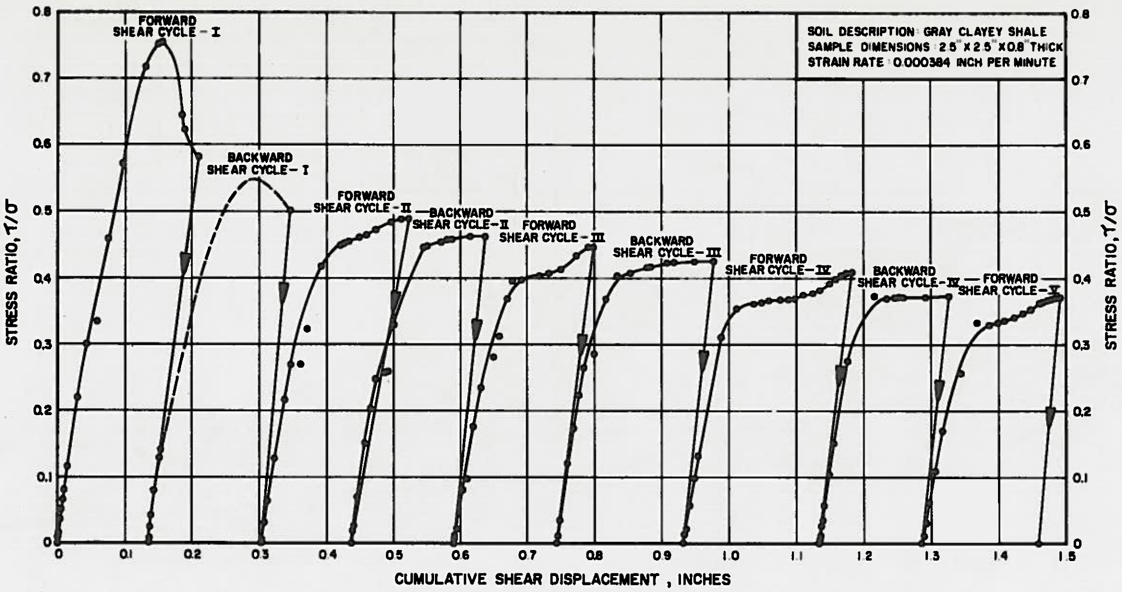


FIGURE 7 CYCLED DIRECT SHEAR TEST ON WEATHERED IN SITU SHALE

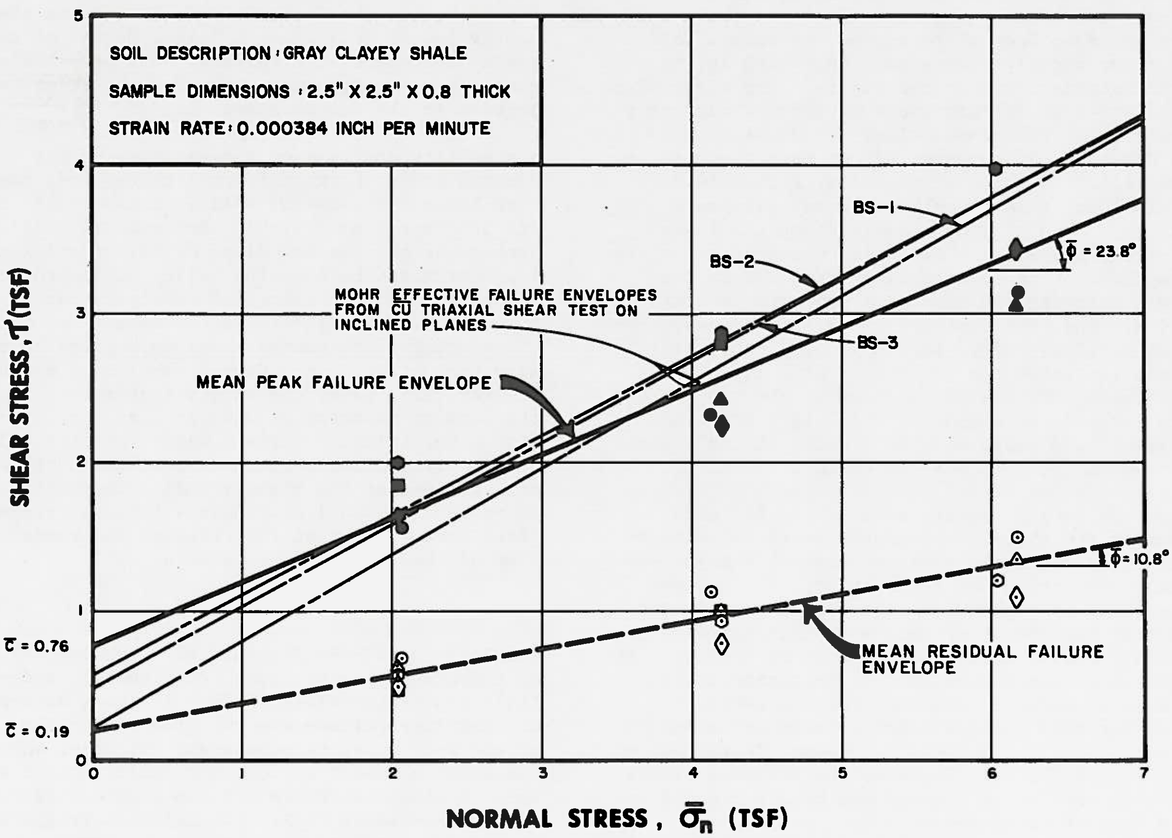


FIGURE 8 MOHR FAILURE ENVELOPES FROM CYCLED DIRECT SHEAR AND TRIAXIAL SHEAR TESTS

SOIL STRENGTH PARAMETERS USED IN STABILITY ANALYSES (DRAINED FAILURE)

STRATA	SOIL PARAMETER	CASE 1 (AS MEASURED PARAMETERS)	CASE 2 (LOSS OF COHESIONS)	CASE 3 (PROGRESSIVE FAILURE)
FILL	$\bar{\phi}$ (DEG)	24.4	24.4	24.4
	$\bar{c}$ (PSF)	360.0	0.0	360.0
RESIDUAL SOIL	$\bar{\phi}$ (DEG)	25.5	25.5	25.5
	$\bar{c}$ (PSF)	114.0	114.0	114.0
WEATHERED SHALE @ TOP OF ROCK	$\bar{\phi}$ (DEG)	23.8	23.8	10.8
	$\bar{c}$ (PSF)	1520.0	0.0	380.0

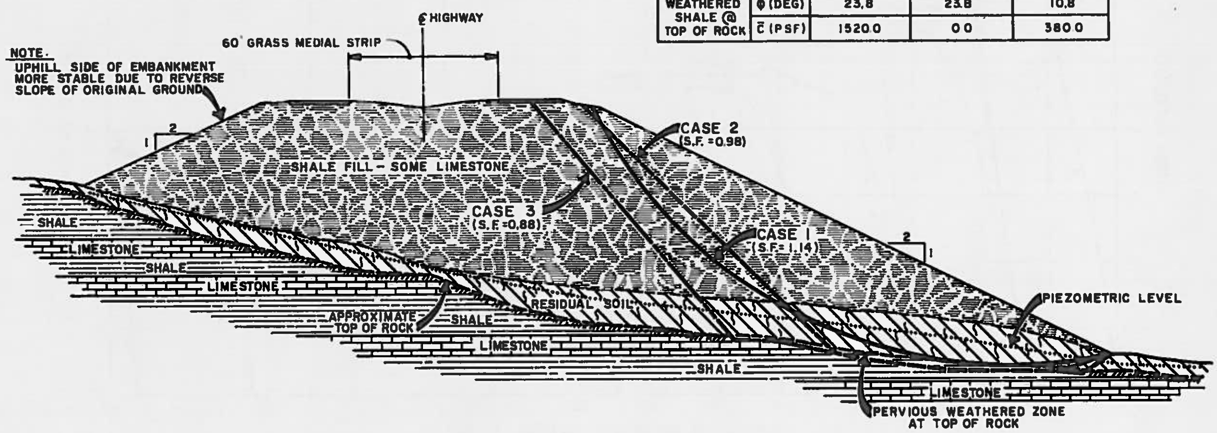


FIGURE 9 STABILITY ANALYSIS OF TYPICAL SIDE-HILL FILL EMBANKMENT

major slope failure. If continued percolation of groundwater through the fill and along the top of rock causes continued degradation of the shale and a corresponding loss of cohesion, the safety factor would theoretically reduce to 0.98, which is the Case 2 analysis shown in the figure. The first sign of a Case 2-type failure would be local sloughing on the surface of the slope, since the slope of 2:1 (26.6 degrees), is somewhat higher than the $\bar{\phi}$ angle of the fill. In fact, at the time of the field investigation, this slope had already developed a significant area of local sloughing on the lower half of the slope. The sloughing occurred at points on the slope where perched water from within the fill was emerging and causing a complete loss of cohesion. The Case 3 analysis for this slope yielded a theoretical safety factor of only 0.88. This analysis simulates the potential for a major slope failure along the weathered shale at the top of rock due to loss of toe support. The Case 3 critical failure surface could also be located at the interface between the fill and the residual soil.

Because the safety factors were so low for this embankment and since considerable surficial sloughing of the slope was already occurring, a major embankment correction was recommended. It involved construction of a granular buttress keyed into the bedrock at the toe of slope. With this correction, the safety factor of the slope would be increased to 1.34 for the Case 2 condition. Recommendations were also included to redesign the toe drainage gutters for this and all other embankments along this section of the highway to prevent initiation of Case 3-type failures. Three of the previous slides along this section of highway had been triggered by such a loss of toe support.

One of the as-built slopes that actually failed was also analyzed for all three cases. The analysis of this embankment indicated safety factors of 1.15, 1.01 and 0.78 for Cases 1, 2 and 3, respectively. The Case 1 and 2 safety factors were very close to those obtained for the deep sidehill fill, shown in Figure 9, but the Case 3 safety factor was significantly lower, indicating a higher degree of sensitivity to progressive failure. The deep-seated slide at this embankment was, indeed, triggered by erosion at the toe of slope.

A stability analysis on a deep cross-valley fill indicated safety factors for all three cases higher than those for sidehill fills. Cross-valley fills are inherently more stable than sidehill fills because of the shallow slope of the underlying valley bottom and because the valley walls tend to "lock" the fill in place. A safety factor of 1.25 for Case 1 was considered acceptable. For this type of embankment, recommendations were given to minimize the potential for Case 2 conditions which would theoretically lower the safety factor to 1.1. Since the loss of cohesion assumed in Case 2 is associated with percolation of surface water into the fill, recommendations were given to seal the surface along the shoulder of the highway and in the medium area. To prevent a Case 3 progressive failure, recommendations were also given for redesign and reconstruction of the toe drainage gutters.

CONCLUSIONS

RELATIVE EMBANKMENT STABILITIES. Two general types of embankments exist along the highway: side-hill fills and cross-valley fills. The most susceptible to stability failure are the side-hill fills. Three of the four previous embankment failures, one existing creep movement and one marginally stable area were in side-hill fills and one previous failure was in a cross-valley fill. (It had been triggered by massive erosion and loss of toe support.)

MODES OF FAILURE. Two different modes of failure had occurred in the past and could occur in the future: (a) deep-seated failure surfaces passing through the soil-bedrock interface or the base of the fill and steeply up through the fill to the edge of the pavement or to the top of slope, and (b) progressive, surficial sloughing on the slopes of the embankments at points where concentrated amounts of perched water were emerging from the fill. Three of the four previous failures, one existing creep movement and one marginally stable area were due to deep-seated failure surfaces.

CAUSES OF INSTABILITY. The primary cause of past failures and present instabilities along this section of highway is the unstable nature of the clay shales in the fill embankments and at the soil-rock interface beneath the embankments. The shales decompose in the presence of water, initially causing a loss of cohesion and, with shear strain, a loss of friction angle. Excessive settlement also results. A secondary cause of instability is embankment toe erosion due to failure of the concrete drainage gutters originally constructed at the toe of slope. Contributing factors to degradation of the shale are:

(a) excessive lift thickness and inadequate compaction during embankment construction because the shale-limestone mixture was treated as "rock" fill,

(b) infiltration of surface water into the poorly compacted, pervious shale fill due to lack of "encasement" by an impervious soil fill, gentle longitudinal highway grades, a wide pervious medial strip, widely spaced storm water inlets, ponding of water in the medium area due to excessive embankment settlements, possible subsurface failure of storm water drainage pipes (due to excessive settlements), and pavement cracking and erosion along the highway berms (due to poor subgrade conditions and differential embankment settlements).

(c) flow of groundwater at top of rock, originating in fractured limestone seams.

SUMMARY

These case histories demonstrate the importance of recognizing the genesis of natural soils and rock formations as well as the properties of the materials. The complex nature of the soil and rock materials described in each of these cases made prediction and prevention of the encountered problems impossible using subsurface investigation and laboratory testing programs considered by many engineers to be entirely adequate. In Case I, more extensive subsurface investigation and study of the surface changes and geology of the region prior to construction may have revealed the potentially hazardous conditions. Design of the service rail to suit these conditions could have minimized the cutting, filling and alteration of drainage at the toe of the slope. Also the settling ponds and refuse piles could have been placed elsewhere. In Case II, more detailed subsurface investigation would have revealed the artesion condition in the limestone and subsurface drainage could have been included in the embankment designs. An understanding of the properties of the shale would have revealed the need for thinner lifts, better compaction and protection of the shale from percolating water. These measures along with keying the embankment into competent soils or bedrock would have prevented the slope failures and excessive settlements.

COMMENTS ON SEISMIC STABILITY EVALUATION OF EMBANKMENT DAMS

by

Gonzalo Castro*

Introduction

Embankment dams had been designed and built, based on empirical criteria accumulated in the course of years, well before the birth of soil mechanics. As soil mechanics ideas and principles were introduced, they were evaluated against that experience. Thus methods of analysis and testing gradually became acceptable and are used today on a routine basis.

Unfortunately, we do not have such an opportunity when dealing with earthquake analysis of embankment dams. Therefore, one should be much more cautious in drawing conclusions from earthquake analysis than one would for static analysis.

We can however profit from the accumulated experience on dam design. In effect, most of the problems one would associate with earthquakes have been dealt with for a long time for static problems. As an example, differential settlements along the crest can be induced statically by differential compression of the foundation soil. As a result of permanent shear deformations transverse to the dam due to an earthquake, similar longitudinal differential settlements can occur. The effect of such

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differential settlements can be transverse cracking, the effects of which can be minimized by internal drainage of large flow capacity, with thick transition zones if needed, (see for example H. Cedergren's paper in the proceeding of this conference), higher freeboard, less-erosion-susceptible soils, etc. In general, if a dam is made safer for static loads, it also becomes safer for earthquake shaking. As will be discussed later, the deformations to be expected from an earthquake cannot be reasonably predicted with our present (1976) state of knowledge. Regardless of what the results of a sophisticated seismic stress-strain analysis might indicate, one should provide a good drainage system with high water removing capabilities that would minimize damages resulting from seismic deformations of the embankment.

There are differences in the stress-strain and strength considerations that are applicable to the static and to the seismic behavior of soils. The purpose of this presentation is an attempt to clarify the difference and to emphasize the similarities between static and seismic behavior of cohesionless soils.

In a cross-section of an embankment dam and its foundation, we define in Fig. 1 the following stresses for typical elements. Normally the static system of effective stresses $(\bar{\sigma}, \tau)_s$ is applied slowly enough so that the soils can be assumed to be fully consolidated to these stresses just prior to an earthquake. The earthquake induced system of stresses $(\sigma, \tau)_e$ are a function of time and are superimposed on the static stresses. If the soils are saturated one can assume that the earthquake stresses are applied under undrained conditions except in the

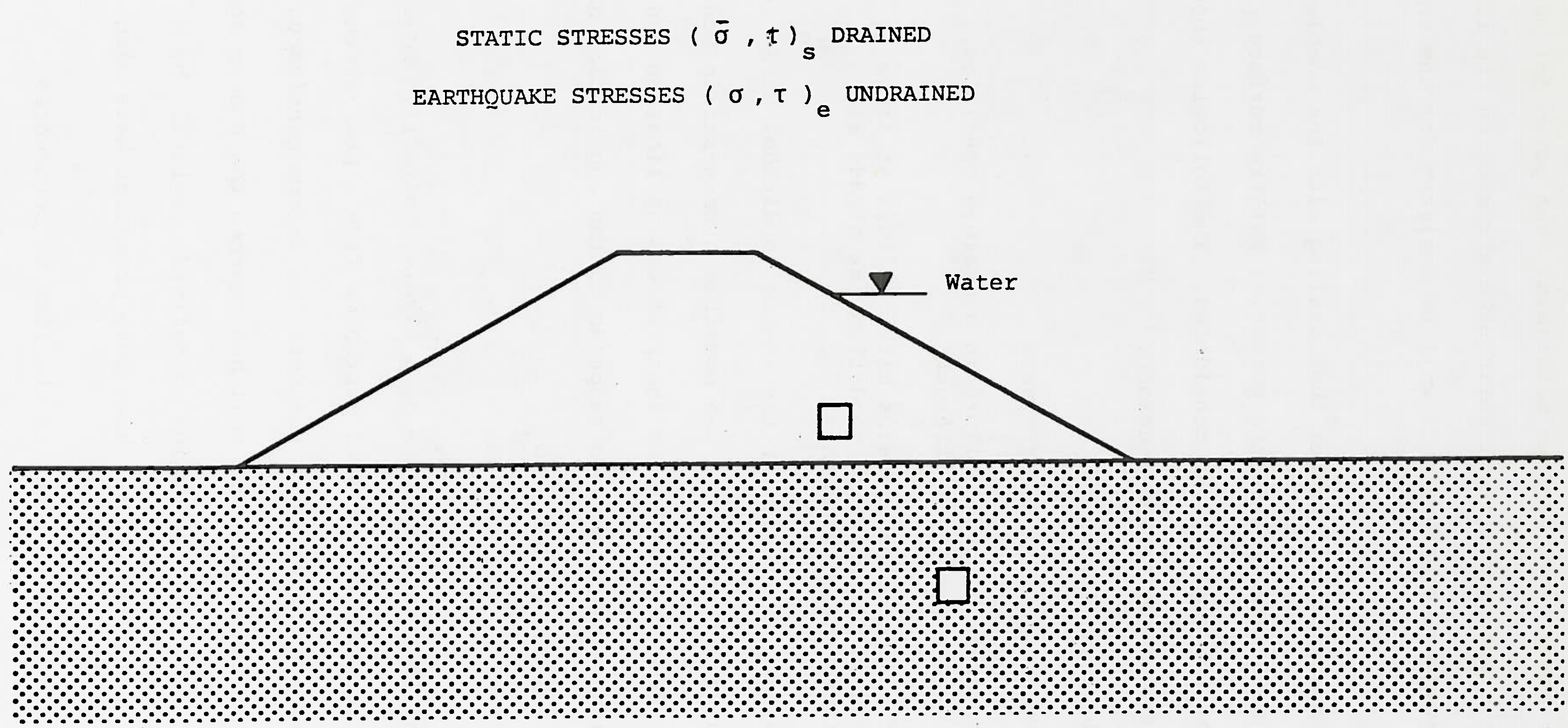

Fig. 1 Stresses in Embankment Dam and its Foundation

case of rock fill or other very pervious materials. The potential different effects of such superposition of earthquake stresses on the static stresses in a saturated cohesionless soil will be analyzed for the undrained case.

For clarity of presentation, rather than dealing with the complete stress systems, the shear stresses on the potential failure surface through an element of interest will be considered. The following shear stresses can be defined on the failure surface:

τ_s = static shear stress

τ_e = earthquake induced shear stress

S_d = shear stress causing failure in a drained condition, i.e. drained shear strength

S_u = shear strength that remains after failure at large strains in an undrained condition, or steady state undrained shear strength

Since the embankment is stable for the static condition, S_d is larger than τ_s. Figs. 2 and 3 illustrate the two possible stress-strain behaviors that can result from the earthquake shear stress application and also the stress-strain behavior that corresponds to the application of monotonic undrained loading to define S_u.

Liquefaction

Figs. 2b and 3b illustrate the case of a highly contractive behavior where the shear strength of the soil decreases from a peak strength to a low shear strength S_u under which the soil can deform continuously without change in resistance, i.e. the soil has reached the steady state of shear or critical void ratio condition as defined originally by A. Casagrande (1).* At the steady state, high pore pressures have been

*Numbers in parenthesis refer to items in the list of references.

CYCLIC LOADS

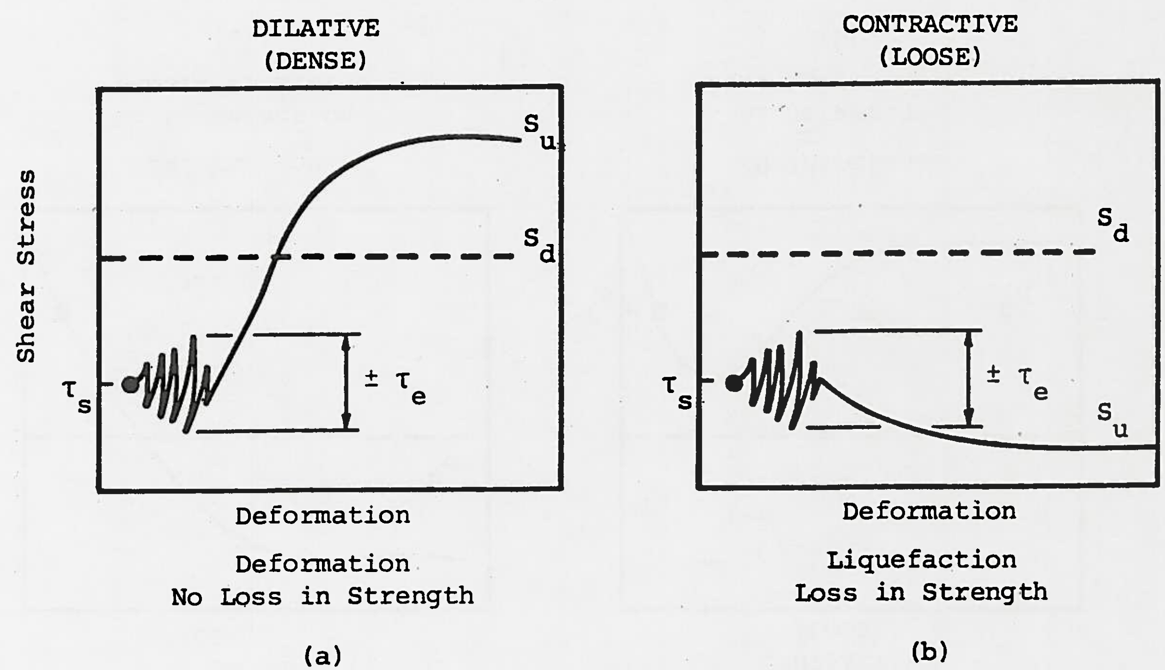

Fig. 2 Stress-strain Cruves. Undrained Cyclic Loading of Saturated Sands.

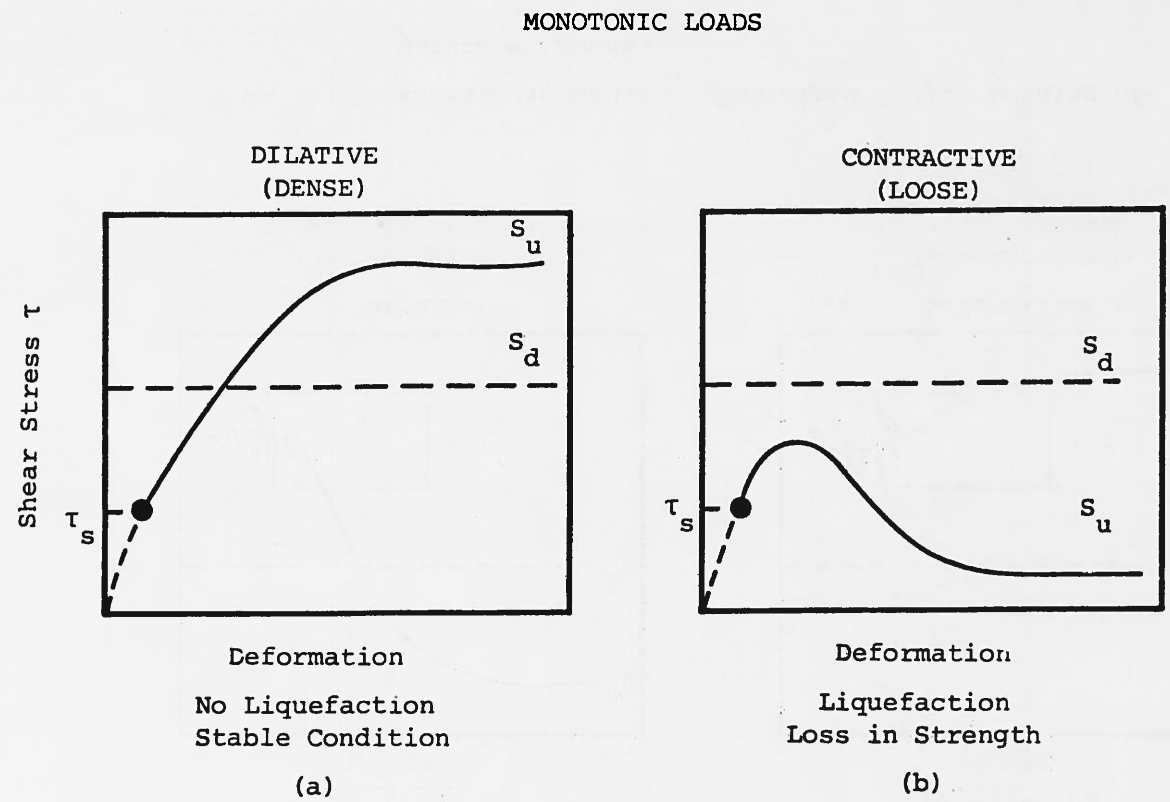

Fig. 3 Stress-strain Curves. Undrained Monotonic Loading of Saturated Sands.

developed in this highly contractive soil. For the stress conditions illustrated in Figs. 2b and 3b, a flow slide would ensue as a result of either a monotonic, Fig. 3b, or cyclic, Fig. 2b, change in shear stresses occurring under undrained conditions. Once the failure has been triggered, extremely large movements will take place, since the shear strength of the soil S_u would be smaller than the static shear stress τ_s. The change of stress that triggered the failure can cease and still the flow slide would continue until the mass of soil comes to rest with a low enough slope so that for the final condition the static stresses are compatible with S_u. The soil is said to have developed liquefaction. The failure during construction of the Fort Peck Dam (2) and the failure during an earthquake of the Lower Van Norman Dam (9) are examples of liquefaction triggered by static and by earthquake loading, respectively. It can be said that whenever $S_u < \tau_s$, the saturated soil is in an unstable condition which is unsafe, and the failure can be triggered by a number of causes. Liquefaction as defined above cannot occur if $S_u < \tau_s$. The value of S_u has been shown to be, for all practical purposes, only a function of the void ratio of the sand, and it is independent of the manner in which the failure was initiated (3, 4, 5).

The slope of a plot of S_u vs. e is rather flat, i.e. a small change in e causes a large change in S_u, as illustrated in Fig. 4. When investigating the stability of a dam and its foundation, S_u can be determined by performing consolidated-undrained triaxial tests on undisturbed specimens where the consolidation stresses represent the static stresses in-situ. However, sampling of cohesionless soils does produce changes

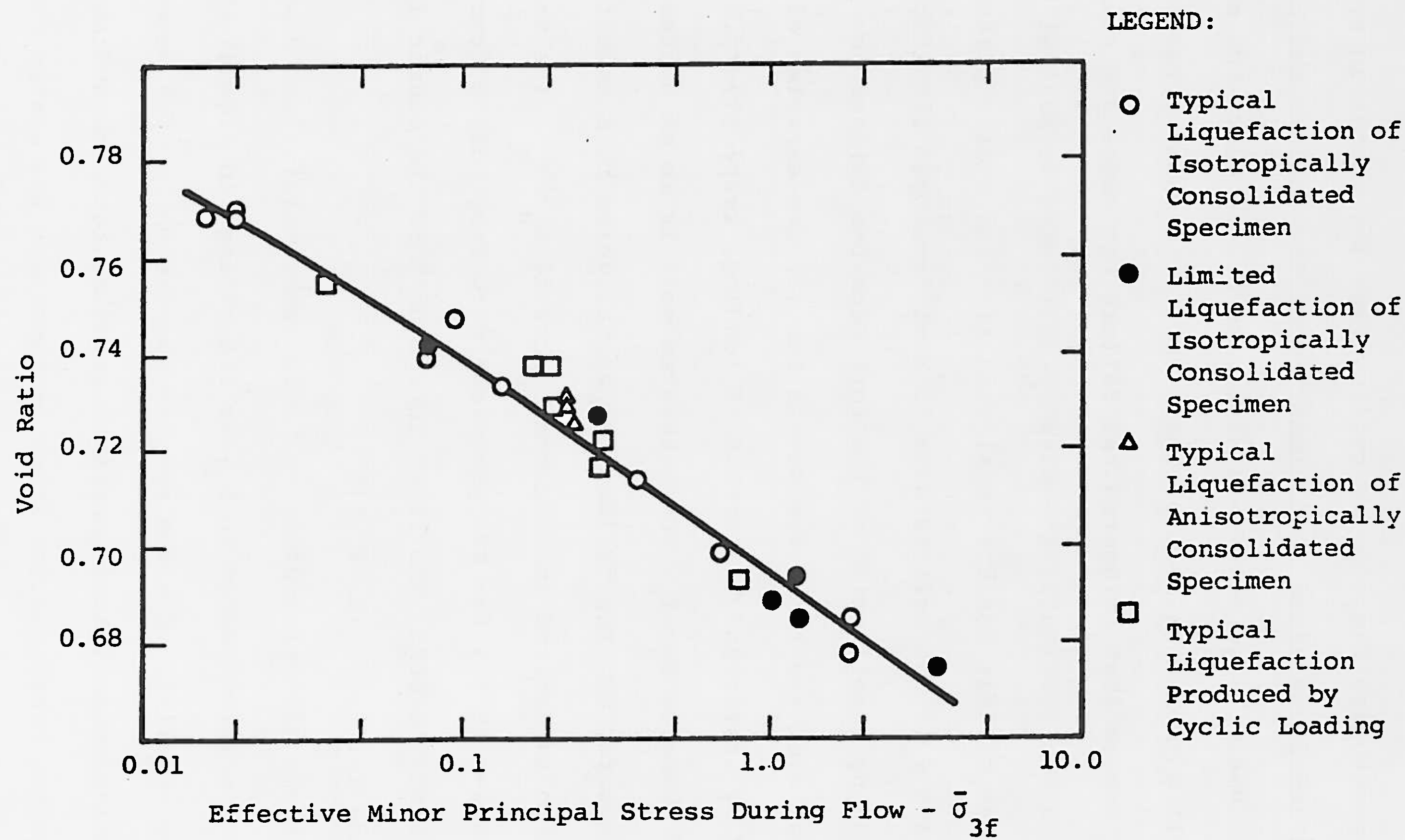

Fig. 4 Steady State Line - Uniform Fine Sand (Ref. 4)

in the void ratio of the soil, and therefore satisfactory safety against liquefaction should be demonstrated by a wide enough margin of safety to embrace the effects of disturbance during sampling of loose sands.

Dilative Behavior

If the result of the monotonic undrained load application is a negative induced pore pressure, the stress strain curve shown in Fig. 3a is obtained. Since the undrained shear strength at the steady state S_u is higher than the drained shear strength, this soil is safer in terms of strength for undrained (fast) load application than for drained (slow) load application. This is also the case for cyclic loading as shown in Fig. 2a in which the effect of cyclic loading is the accumulation of some deformation, with possibly some increase in pore pressure, but the cyclic loading has no effect on the potential shear strength S_u, as has been shown elsewhere (6). That this should be the case follows from the concept of steady state or critical void ratio, and the corollary that S_u is only a function of the void ratio.

The effect of cyclic loading on dilative soils is not one of loss of shear strength but of accumulation of deformations. For the special case in which the earthquake shear stresses are higher than the static shear stresses, the pore pressure can become so high that the effective stresses become zero momentarily when the shear stress passes through zero. In such a case the undrained strength of the soil remains unchanged too, and the problem is still one of deformations. Confusion has resulted from referring to the momentary zero effective stress condition as "initial liquefaction" (8), even though it does not correspond to the loss in shear strength that occurs in flow slides, i.e. it is not a problem of loss in strength but one of accumulation of deformation. In the case of liquefaction in the sense of a loss in strength, the movement does not stop when the cyclic loading stops, but the latter is only the triggering mechanism. Evidence that most of the movement occurred after the earthquake stopped was presented for the Lower Norman Dam failure on the basis of an analysis of the seismoscope records (9). Had the mechanism of failure been one of accumulation of deformation on a dilative soil, the movement would have stopped as soon as the strong shaking stopped.

Evidence has been presented elsewhere (7) that the two cyclic loading problems discussed above, strength and deformation, are affected differently by different parameters, e.g. confining pressure and static shear stress. An analogy to illustrate this point consists of the bearing capacity and the settlement considerations for two footings on sand as shown in Fig. 5. Both are subject to the same bearing pressure, but footing A is smaller in area than footing B. From the bearing capacity (soil strength) point of view, footing B is safer than A. From the settlement (deformation) point of view, footing B will settle more than A. Thus one can see that the influence of one parameter, size is the opposite for the strength and for deformation problem. Thus traditionally bearing capacity and settlement are dealt with separately in foundation analysis. Similarly, the strength (liquefaction) and deformation (cyclic mobility) problems should be considered separately when analyzing effects of earthquakes on embankment dams.

STRENGTH PROBLEM (BEARING CAPACITY): B BETTER THAN A

DEFORMATION PROBLEM (SETTLEMENT): A BETTER THAN B

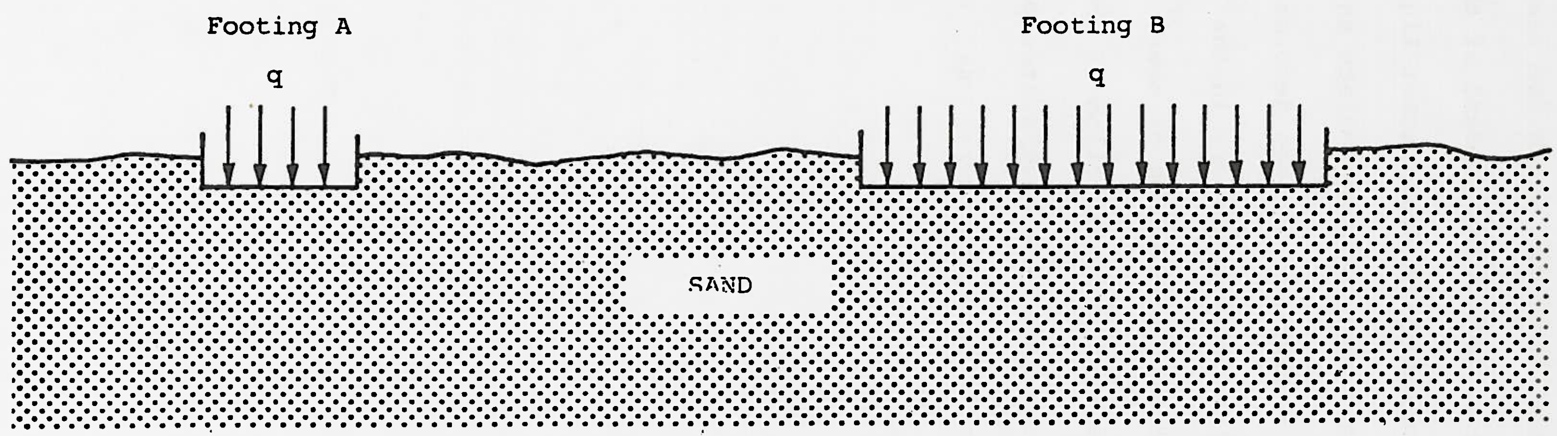

Fig. 5 Comparison of Strength and Deformation Considerations for a Footing on Sand.

The strength (liquefaction) problem should be analyzed on the basis of steady state undrained strength, which for practical purposes is only a fraction of void ratio and is independent of the details of the stress history. If it is found that the soil cannot liquefy, one still should evaluate the deformations that will accumulate as a result of the earthquake. The analytical evaluation of such deformations requires an accurate representation of the stress history in the laboratory. As in other areas of soil mechanics, the evaluation of shear deformations is subject to larger uncertainties than the evaluation of shear strength, and therefore a careful evaluation should be made of the possible errors in the test results and the analytical procedures used.

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APPENDIX

PAST PROCEEDINGS OF THE OHIO RIVER VALLEY SOILS SEMINARS

ORVSS I
BUILDING FOUNDATION DESIGN AND CONSTRUCTION

October 16, 1970
Lexington, Kentucky

AND WHY DO WE DO IT THIS WAY? by R. C. Deen, Assistant Director of Research, Kentucky Bureau of Highways.

A CASE STUDY - PIPE PILING FOR A DEEP FOUNDATION PROBLEM by Don Fuller, P.E., L.S., Fuller, Mossbarger and Scott Civil Engineers, Inc.

DESIGN, CONSTRUCTION AND PERFORMANCE OF SPREAD FOOTINGS ON A NON-UNIFORM FOUNDATION by Robert Lennertz, M. ASCE, The H. C. Nutting Company.

DESIGN, CONSTRUCTION AND TESTING OF DRILLED PIERS ON SHALE AND THINLY BEDDED LIMESTONE by George J. Thelen, P.E.

BUILDING FOUNDATION FAILURE RELATED TO SOIL SHRINKAGE by Aubrey D. May, Chief Engineer, Stokley and Associates.

INNOVATIONS IN FOUNDATION DESIGN AND CONSTRUCTION by D. J. Hagerty, Assistant Professor, Civil and Environmental Engineering Department, University of Louisville.

ORVSS II
EARTHWORK ENGINEERING, START TO FINISH

October 15, 1971
Louisville, Kentucky

PRELIMINARY SITE SELECTION STUDIES WITH REMOTE SENSING TECHNIQUES by David J. Barr, Department of Civil Engineering, University of Cincinnati.

SITE EVALUATION WITH REMOTE SENSING TECHNIQUES APPLIED TO SUSPECT KARST TOPOGRAPHY by Melville D. Hensey, Engineering Division, The Proctor and Gamble Company.

EXPLORATION METHODS by Dr. Vincent P. Drnevich, P.E., Assistant Professor of Civil Engineering, University of Kentucky.

THE SELECTION OF STRENGTH PARAMETERS AND THEIR USE IN THE STABILITY ANALYSIS OF EARTH ROCK EMBANKMENTS by Woody McGraw and Don Tupman, Louisville District, U.S. Army Corps of Engineers.

CONTRACTOR'S PROBLEMS by Richard Geottle.

ENVIRONMENTAL CONSIDERATIONS IN SOILS ENGINEERING by D. Joseph Hagerty, Asst. Professor of Civil Engineering, University of Louisville and Joseph L. Pavoni, Asst. Professor of Civil and Environmental Engineering, University of Louisville.

ORVSS III
LATERAL EARTH PRESSURES

October 27, 1972
Fort Mitchell, Kentucky

INSTRUMENTATION FOR STRESS AND STRAIN MEASUREMENTS IN SOILS by Stanley D. Wilson, Executive Vice-President, Shannon and Wilson, Inc.

DEEP EXCAVATION AND BUILDINGS SUPPORTED BY TIE-BACK SYSTEM by Richard Geottle, P.E., Richard Geottle, Inc., James J. Flaig, P.E., H. C. Nutting Company, Arthur J. Miller, P.E., Miller, Tallarico, McNinch, and Hoeffel Consulting Engineers, and Steven E. Schaefer, EIT, Miller, Tallarico, McNinch, and Hoeffel Consulting Engineers.

LATERAL PRESSURES AND MOVEMENTS CAUSED BY PILE DRIVING by D. Joseph Hagerty, A.M. ASCE, Assistant Professor of Civil Engineering, University of Louisville.

FACTORS TO BE CONSIDERED IN THE DESIGN OF BACKFILL FOR LARGE DIAMETER FLEXIBLE METAL CONDUITS by David C. Cowherd, M.S., P.E.

GENERALIZED SLIDING WEDGE METHOD FOR SLOPE STABILITY AND EARTH PRESSURES ANALYSIS by Dr. Vincent P. Drnevich, Assistant Professor of Civil Engineering, University of Kentucky.

LATERAL PRESSURES AND PRESTRESSED TIE-BACK WALLS by P. C. Rizzo and R. D. Ellison, E. D'Appolonia Consulting Engineers, Inc., and R. J. Shafer, Herbert F. Darling, Inc.

DESIGN AND CONSTRUCTION OF SUPPORTED TEMPORARY EXCAVATIONS IN URBAN ENVIRONMENT by Yves Lacroix and Walter T. Jackson, Woodward-Moorhouse and Associates, Inc.

ORVSS IV
GEOTECHNICS IN TRANSPORTATION ENGINEERING

October 5, 1973
Lexington, Kentucky

RETRIEVAL AND USE OF GEOTECHNICAL INFORMATION by D. Joseph Hagerty, Assistant Professor of Civil Engineering, University of Louisville, Nicholas G. Schmitt, Soils Engineer, Law Engineering Testing, William J. Pfalzer, Research Engineer Assistant, Kentucky Bureau of Highways.

IN SITU SHEAR STRENGTH PARAMETERS BY DUTCH CONE PENETRATION TESTS by C. T. Gorman, Research Engineer Assistant, Kentucky Bureau of Highways, T. C. Hopkins, Research Engineer Principal, Kentucky Bureau of Highways, and V. P. Drnevich, Associate Professor of Civil Engineering, University of Kentucky.

REINFORCED EARTH - DESIGN AND CONSTRUCTION by David S. Gedney, Regional Engineer, Region 15, Federal Highway Administration and John L. Walkinshaw, Highway Engineer, Federal Highway Administration.

APPLICATIONS OF THE FINITE ELEMENT METHOD TO GEOTECHNICS AND TRANSPORTATION ENGINEERING by Yang H. Huang, Associate Professor of Civil Engineering, University of Kentucky.

RELIABILITY ANALYSIS AND COST OPTIMIZATION OF EMBANKMENTS by Warren J. Baker, Chrysler Professor and Dean, College of Engineering, University of Detroit.

STABILITY ANALYSIS - ITS POTENTIAL AND ITS LIMITATIONS by T. H. Wu, Professor of Civil Engineering, The Ohio State University.

ORVSS V
ROCK ENGINEERING

October 18, 1974
Clarksville, Indiana

EXPLORATION AND TESTING IN ROCK by Joseph Hagerty, Civil Engineering Department, University of Louisville, and James Coulson, Tennessee Valley Authority.

A ROCK CLASSIFICATION SCHEMA by R. C. Deen, Assistant Director, Kentucky Bureau of Highways, Division of Research, C. D. Tockstein, Research Engineer Assistant, Division of Research, and M. W. Palmer, Research Engineer Assistant, Division of Research.

FOUNDATIONS ON OR IN ROCK by E. D'Appolonia, E. D'Appolonia Consulting Engineers, Inc.

ROCK FOUNDATIONS (A Panel Discussion) Comments by: C. R. Lennertz, H. C. Nutting Company, Harry Thomas, US Army Corps of Engineers, and Milton M. Greenbaum, Greenbaum and Associates.

DESIGN OF OPEN EXCAVATIONS IN ROCK by D. Ross-Brown Dames & Moore.

ROCK EXCAVATION (A Panel Discussion) The Producer's Viewpoint by J. A. Waddell, Martin Marietta Aggregates, Consideration of Effects on Surroundings by D. J. Hagerty, University of Louisville, and Geologic and Support Considerations for Rock Excavations by J. Mahar, University of Illinois.

ROCK ENGINEERING ON SOME RECENT PROJECTS by Don U. Deere, Consultant in Engineering Geology and Rock Mechanics, University of Florida.

ORVSS VI
SLOPE STABILITY & LANDSLIDES

October 17, 1975
Fort Mitchell, Kentucky

GEOLOGIC PERSPECTIVES--THE CINCINNATI EXAMPLE by Robert W. Fleming, U.S. Geological Survey.

IMPORTANCE OF GEOLOGIC STRUCTURE IN STABILITY OF ROCK SLOPES by Robert L. Schuster, William K. Smith, and Fitzhugh T. Lee, U.S. Geological Survey.

CUT AND FILL ORDINANCE AS ADOPTED BY THE CITY OF CINCINNATI by James. R. Krusling, P.E.

DRILLED PIER RETAINING WALLS by Merle F. Nethero, Geotechnical Engineer - Vice President, H. C. Nutting Company.

REGRADING FAILED SLOPES by H. A. Mathis, Kentucky Department of Transportation.

EFFECTS OF WATER ON SLOPE STABILITY by Tom C. Hopkins, Research Engineer Chief, David L. Allen, Research Engineer Principal, and Robert C. Deen, Assistant Director, Kentucky Department of Transportation, Division of Research.

ORVSS VII
SHALES AND MINE WASTES:
GEOTECHNICAL
PROPERTIES, DESIGN, AND CONSTRUCTION

October 8, 1977
Lexington, Kentucky

GEOTECHNICAL PROPERTIES OF SOME EASTERN KENTUCKY SURFACE MINE SPOILS by V. P. Drnevich and R. J. Ebelhar, University of Kentucky, and G. P. Williams, U.S. Department of Agriculture, Forest Service

GUIDELINES FOR COMPACTED SHALE EMBANKMENTS by L. E. Wood and C. W. Lovell, Purdue University, and W. W. Sisiliano, Indiana State Highway Commission

ENGINEERING EVALUATION OF A CLEVELAND SHALE by V. K. Khosla, Heron Testing Laboratories, Inc. and R. L. Murdoch, Barber and Hoffman, Inc.

PHYSICAL AND ENGINEERING CHARACTERISTICS OF COAL PREPARATION PLANT REFUSE by C. S. Bishop and J. G. Rose, University of Kentucky

REBOUND PROPERTIES OF REMOLDED CLAY SHALES by C. R. Ullrich, University of Louisville

GENERAL REPORT ON DESIGN AND CONSTRUCTION by G. A. Leonards, Purdue University

USE OF MINE WASTE IN TAILINGS DAMS by W. F. Brumund, Golder Associates

COAL REFUSE: ITS BEHAVIOR RELATED TO THE DESIGN AND OPERATION OF COAL REFUSE DISPOSAL FACILITIES by R. G. Almes and A. Butail, E. D'Appolonia Consulting Engineers, Inc.

USE OF COMPACTED SHALE AS DAM EMBANKMENTS by C. A. Fetzer, U.S. Army Corps of Engineers

AIRPORT EMBANKMENT UTILIZES COAL STRIP MINE WASTE by J. M. Sheahan and T. F. Hunkele, Richardson, Gordon and Associates

OBSERVATION EVALUATIONS OF COAL REFUSE EMBANKMENT STABILITY by W. W. Parker and R. E. Gray, GAI Consultants, Inc.

CONVENTIONAL AND UNCONVENTIONAL APPROACHES USED TO LOCATE AND ELIMINATE HAZARDOUS MINE WASTE DAMS IN OHIO by V. V. Rajadhyaksha, Ohio Department of Natural Resources

GEOTECHNICAL OVERSIGHT NULLIFIES PROPER PROCEDURES by B. Rosen, Marks-Rosen, Inc.

WEST VIRGINIA EXPERIENCES WITH REVIEW OF COAL REFUSE DISPOSAL FACILITIES by G. Hall and M. K. Robinson, West Virginia Coal Refuse and Dam Control Section, and R. E. Smith, Woodward-Clyde Consultants

CLOSED CIRCUIT COAL REFUSE AS A STRUCTURAL FILL by D. C. Cowherd, Bowser-Morner Testing Laboratories, Inc.

DYNAMIC DESIGN CONSIDERATIONS OF LOOSE FINE COAL REFUSE by R. D. Ellison and Y. Y. Cho, E. D'Appolonia Consulting Engineers, Inc.

PROPERTIES OF COAL MINE FLOOR SHALE by R. W. Stephenson and J. D. Rockaway, University of Missouri-Rolla

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