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SLOPE STABILITY & LANDSLIDES

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SLOPE STABILITY AND LANDSLIDES

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GEOLOGIC PERSPECTIVES--THE CINCINNATI EXAMPLE

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Introduction

Landslides are a common and longstanding problem in the Greater Cincinnati area. Reports of a resulting road closure, construction delay, or destruction of property occur altogether too frequently, and correction of slope-movement problems involves large expenditures of time and money. Estimates for correction of a major landslide that occurred in 1974 in connection with the construction of Interstate Highway 471 ranged upward from about $8 million. During the period 1927-30, a landslide in Riverside, west of downtown Cincinnati, destroyed some 40 houses (Von Schlichten, 1935).

Although most slides appear to occur in the late winter or early spring, some happen during each month of the year. Not all slides along a valley wall on Hillside Avenue in Cincinnati have occurred on the steepest slopes. Stability problems are not restricted to the valley walls along the Ohio River and major tributaries but are common throughout much of the metropolitan area. For example, Yahne (1974) identified 49 active and incipient landslides in a 9-square-mile (23-km^2) area immediately north of downtown Cincinnati. These seemingly unrelated facts make it difficult to grasp significant relationships among slope problems--and, taken as a whole, the slope-movement problems in the Greater Cincinnati area seem inordinately complex. Nevertheless, recognizing the apparent relation of slope movements to distinct and mappable geological units enables one to make some definitive statements about potentially hazardous areas. Many unfortunate experiences can be avoided if these areas are identified and described in a form that can be understood by public officials, contractors, and developers, as well as by the public in general. The purpose of this paper is to illustrate the relationships between various types of slope movements and the geological conditions that prevail in those areas.

Geology and landslides

Hamilton County, Ohio, occupies 414 square miles (1,072 km^2) of extreme southwestern Ohio. The maximum relief in the county is 507 feet (155 m). The altitude varies from about 962 feet (293 m) at the Mt. Airy water tower, at the intersection of Colerain and North Bend Road in Cincinnati, to about 455 feet (139 m) at the mouth of the Miami River near the southwestern corner of Ohio (fig. 1). The topography is characterized by a rolling upland surface, hill slopes along the Ohio River and major tributaries, and flood plains and terraces of the rivers. The uplands are generally covered by glacial deposits, mostly till of Illinoian age. Bedrock, consisting of thin-bedded limestone and shale, underlies the glacial deposits at various depths in the upland areas and is exposed on some hill slopes where erosion has removed the glacial cover. The major river of the area, the Ohio, and its tributaries, Mill Creek, Little Miami River, Whitewater River, Miami River, and Licking River, contain flood plains and terraces of varying widths and heights. Materials in the terraces and flood plains vary from sand and gravel to laminated silts and clays.

The geology of the Greater Cincinnati area can best be described in two parts: (1) the bedrock of the area and the soils developed on bedrock, and (2) the glacial deposits of the area and the soils developed on them. Units related to both the bedrock and glacial deposits are unstable and, moreover, the modes of failure appear related to particular material types. Therefore, this report considers first the nature and distribution of the bedrock units, together with an analysis of the slope movements that are indigenous to them, and then presents a similar discussion of the glacial units.

Bedrock

The Greater Cincinnati area is underlain by shales and limestones of Middle and Late Ordovician age. Geologic maps of the bedrock geology of four quadrangles (Madeira, Cincinnati East, Cincinnati West, and Addyston-Burlington) in Ohio have been published by the Division of the Geological Survey, Ohio Department of Natural Resources (Osborne, 1970, 1974; Ford, 1972, 1974). The U.S. Geological Survey is nearing completion of a program

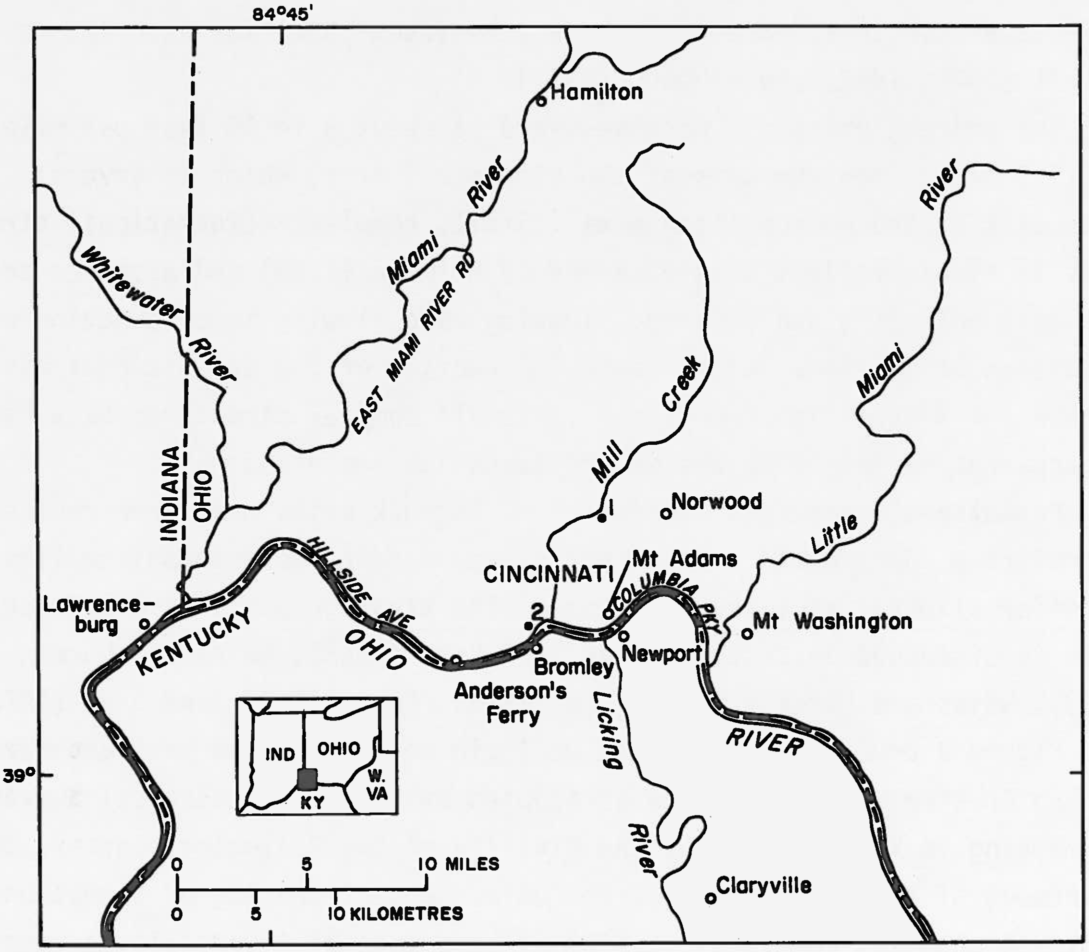

Figure 1.--Sketch map of Cincinnati area.

of geologic mapping of all the quadrangles in Kentucky, and, at the time of this writing, maps of most of the areas in Kentucky that are part of the Greater Cincinnati area have been published. See, for example, maps by Luft (1971, 1972) and Gibbons (1972).

The bedrock units dip northwestward at about 8 to 10 feet per mile (1.5-1.9 m/km) from the axis of the Cincinnati Arch, which is several miles east of the metropolitan area. Small, complex, deformational structures in stream valleys were observed by Hofmann (1966) and ascribed to erosional unloading and sliding, slumping, and flowing in conjunction with deposition of the beds in the general direction of the depositional dip. Neither the slight dip of beds nor the small complex structures have had any apparent influence on the modern landslide processes.

Formational names and boundaries of bedrock units have been revised several times during the past 20 years, and recent publications continue to differ slightly as to nomenclature. The stratigraphy of the bedrock units is discussed in Caster, Dalvé, and Pope (1955), Weiss and Norman (1960), Weiss and Sweet (1964), Peck (1966), Ford (1967), and Gray (1972).

Figure 2 presents generalized geologic columns, based on lithology, for the Greater Cincinnati area as adopted by the U.S. Geological Survey for mapping in Kentucky and by the Division of the Geological Survey, Ohio Department of Natural Resources, for Ohio. Interfingering of formations, thickness changes, and local warping of beds make it impossible to specify elevations of the formational contacts in the entire area. These are best obtained from the published maps and projected, if necessary, into unmapped areas.

The bedrock units appear relatively stable in most slopes. There are a few minor instances of rockfalls associated with steeper slopes and abandoned limestone quarries in the Fairview Formation. In one more serious incident, a large mass of Bellevue Limestone was observed to have moved laterally, apparently on the Miamitown Shale, a distance of several inches, posing a potential threat to property downslope (R. H. Durrell, oral commun., 1973). However, if this were the extent of the slope stability problem in the Greater Cincinnati area there would be no cause for the present concern.

System	Series	KENTUCKY (U.S. Geol. Survey)	OHIO (Ohio Dept. Natural Resources)
ORDOVICIAN	Upper Ordovician	Bull Fork Formation 85+ (26+) 50% shale	Undifferentiated 0-135+ (0-41+) % shale varies
		Grant Lake Limestone (Bellevue tongue) 0% shale 2-33 (2-10)	Bellevue Limestone 0-25 (0-8) 0% shale
		Miamitown Shale of Ford (1967)	Miamitown Shale 0-35 (0-11) 90% shale
		Fairview Formation ∿100 (∿30) 45-60% shale	Fairview Formation ∿70-100 (∿21-30) 60-75% shale
		Kope Formation 200+ (61+) 80-85% shale	Kope Formation 200+ (61+) 70-80% shale
	Middle and Upper Ordovician	Point Pleasant Formation ∿100 (∿30) 30-50% shale	Not exposed

Figure 2.--Generalized geologic columns for the Greater Cincinnati area in Kentucky and Ohio, showing thickness in feet (metres) and percentage of shale for each unit.

The stability problems, in general, involve not the intact, unweathered bedrock but rather the colluvium derived from the bedrock by weathering. Most of the shales in the bedrock sequence, particularly those in the Kope Formation, are not tightly cemented and slake readily to their constituent grains. Certain hill slopes are covered by a layer of bedrock-derived colluvium that may vary in thickness from a few inches to at least 30 feet (9 m). Figure 3a is a view of a near-vertical cut in the Kope Formation. The interbeds of shale and limestone in this cut are apparently stable. Other similarly cut slopes in the same material, such as at the Newport Shopping Center, Newport, Ky. (fig. 1), have stood for several years without landslide problems. The shale interbeds, however, slake and form a pile of debris at the foot of the cut slope. Figure 3b is a closeup view of a block of previously unweathered shale that has been exposed for only a short time and has already begun to disintegrate.

There have been no published studies of the engineering characteristics of the shales in the area. Observations of the slaking behavior suggest that the shales in the Kope Formation are more susceptible to physical disintegration than shales in other formations in the area. This may be more apparent than real, however, simply because the Kope contains a greater proportion of shale and because it occupies the lower part of most slopes and can collect debris transported by water or creep from upslope.

The overlying Fairview Formation supports a steeper slope than does the Kope Formation, and the soil cover is noticeably thinner. Natural slopes on the Kope Formation may be as low as 9° or 10°, while those on the Fairview Formation may exceed 20°. There is commonly a noticeable topographic break at or near the interformational contact. Whether the steeper slope and thinner soil are a result of (a) cementation in the shale, as suggested by Von Schlichten (1935), or (b) transport of slaked debris off the Fairview Formation by water or creep, or (c) a difference in clay mineralogy (Bassarab and Huff, 1968) is not known. The presence of more limestone in the Fairview Formation could, in itself, account for the steeper slope. Whatever the reason, landslides related to bedrock in the Greater Cincinnati area are almost exclusively contained in the colluvial cover overlying the Kope Formation. In the relatively unpopulated part of northwestern Hamilton County, where the Miamitown Shale reaches its maximum thickness, smaller but similar problems occur on that unit.

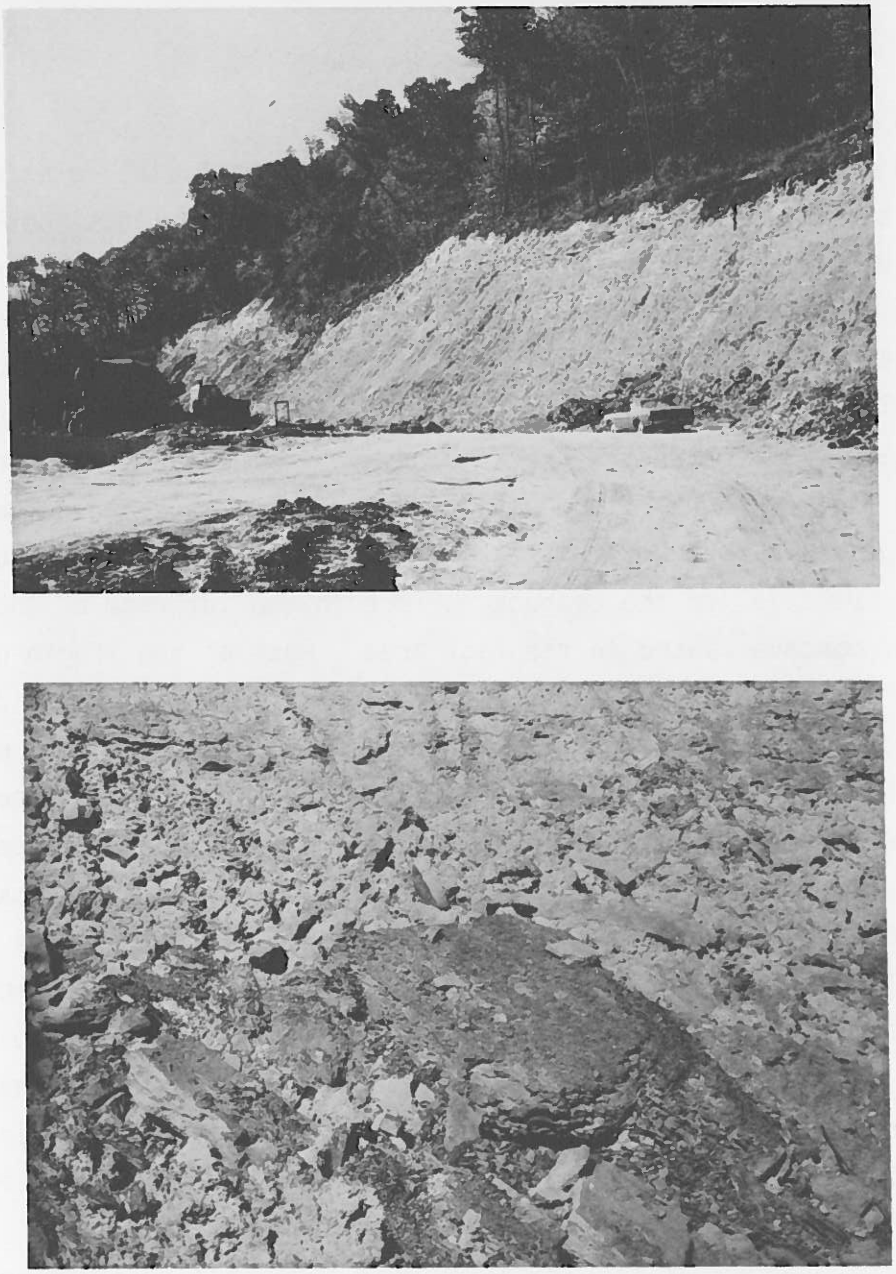

Figure 3.--Photographs of the Kope Formation along Hillside Avenue near Anderson's Ferry.

a. Excavation into Kope Formation showing its rocklike character. Note accumulated debris at foot of excavation and slide in colluvial material uphill from ready-mixed concrete truck.

b. Closeup view of a block of shale that is slaking after short exposure. Largest dimension of shale block is about 2 feet (0.6 m) across.

Landslides associated with the Kope Formation

Two different modes of failure occur in the colluvial materials overlying the Kope Formation. In the first mode large landslides involving areally several thousand square feet of failed materials, such as those that have occurred along the Ohio River in Bromley, Kentucky, and Hillside Avenue and East Miami River Road in Hamilton County, Ohio, have the morphology of a typical slump (fig. 4). The crown area is characterized by open, crescent-shaped cracks, and the foot area by a series of transverse ridges which are suggestive of earthflow. The principal failure surface appears to have a composite geometry that is nearly vertical in the crown area, parallel to the bedrock surface in the interior of the slide, and curved, concave-upward in the foot area. Much of the length of the failure is along the gradational contact between the colluvium and weathered bedrock. A slide of this type is first indicated by one or more crescent-shaped cracks that develop in the upper part of the slope. A slope can exist in this state for several years. A fully developed failure surface is indicated by downward slumping in the area of the cracks, accompanied by bulging in the lower part of the slope. Movement tends to be slow, usually only a few feet per year, and there appears to be scant change in the morphology of the slope failure after it has become fully developed. The characteristic transverse ridges and bulges in the foot area that are suggestive of earthflow do not typically mobilize into flows. Small slumps commonly form in the main scarp of the landslide after it has become fully developed.

Weathered bedrock is significantly more permeable than the underlying unweathered bedrock or the overlying colluvium. Water is commonly encountered in borings in this zone, and water levels after completion of drilling are, in many instances, higher than where first encountered. This suggests that excessive uplift pressures may exist under the colluvium, and failure may thus be enhanced. Excess water pressures seem to be a reasonable explanation for slope failures where strength of the material as measured by several types of tests in the laboratory appears sufficient to support the slope. The areal extent and magnitude of the excess water pressures have not been thoroughly studied, and their influence remains speculative.

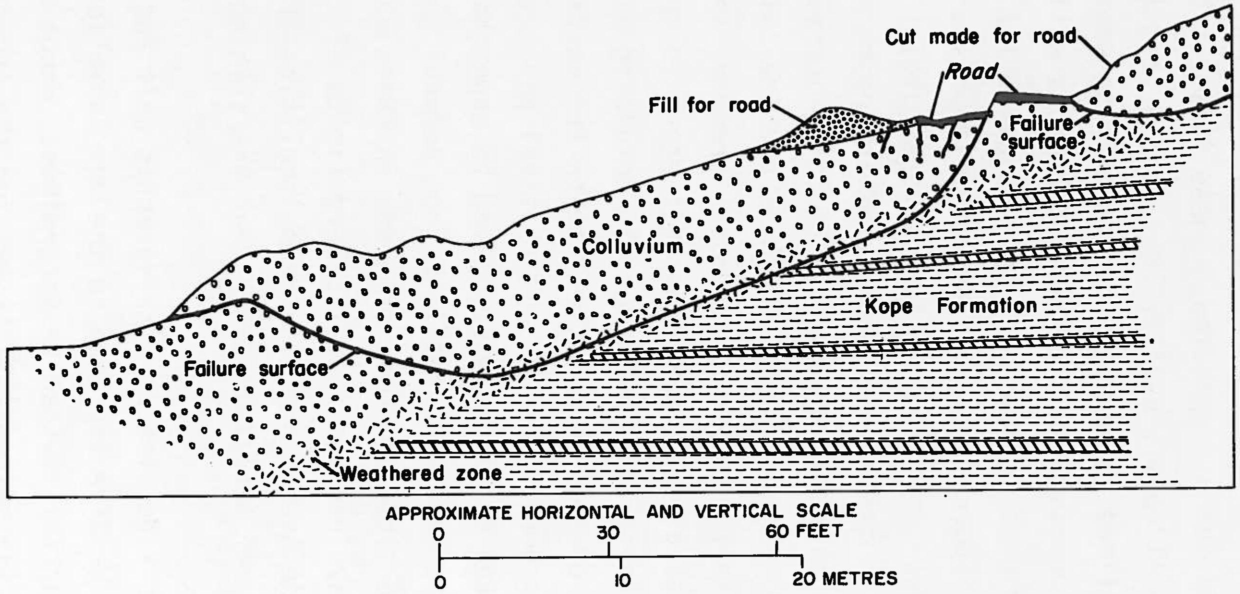

Figure 4.--Sketch of typical deep-seated landslide in colluvial material on the Kope Formation.

A second mode of slope movement associated with the colluvium overlying the Kope Formation is rapid earthflow. Failures of this type are common in steeper slopes of the Kope Formation in areas such as Columbia Parkway and Elberon Avenue (location 2 on fig. 1) in Cincinnati. Such failures most commonly occur in the early spring, after the soil has thawed but before vegetation has sprouted. These failures occur in areas where the entire colluvial cover, usually less than 6 feet (2 m) in total thickness, becomes mobilized and exposes the underlying weathered bedrock. Water apparently is a dominant agent in this process as well. A layer of weathered bedrock which extends under the shallow colluvial cover has similar potential for development of excess hydrostatic pressure to that of the larger, deeper seated landslides. After several cycles of freezing and thawing in the winter months, the density of the upper part of the soil is at a minimum. Heavy rains saturate the soil, increase its weight, and reduce its strength. These factors, perhaps combined with excess pressures in the water in the weathered bedrock, produce rapid failure.

Figure 5 illustrates a rapid earthflow that occurred in 1975 along Columbia Parkway. The earthflow moved over the retaining wall into the street, exposing layers of weathered bedrock below the surface of rupture.

Table 1 contains a compilation of some physical property data on slide materials in colluvium and on slightly weathered to unweathered shale of the Kope Formation. Data have been obtained from several geotechnical firms with experience in slope-movement problems in these materials. The large differences in clay fraction and Atterberg limits of the colluvial materials at the Hillside Avenue and Mt. Adams localities (fig. 1) appear to reflect contributions of materials other than the slaking of the Kope Formation.

Because the Kope is a persistent stratigraphic unit and because the overlying colluvium can be directly tied to the Kope Formation, areas of relative landslide susceptibility can be determined. Hough and Fleming (1974) prepared a landslide susceptibility map for the incorporated areas of Cincinnati by combining the location of outcrop of the Kope Formation with maps of 10-percent slope (5.7°) and 20-percent (11.3°) slope. On the basis of empirical observations, areas of 20-percent slope or steeper

Figure 5.--Earthflow on Columbia Parkway, spring 1975. The semicircular channel on the left is part of the exposed failure surface.

Table 1.--Summary of physical property data related to Kope Formation

[Sources of data are listed in "Acknowledgments"]

	Liquid limit (% H_2O)	Plastic limit (% H_2O)	Moisture content (percent)	Dry density (lb/ft^3)	Unconfined compressive strength (t/ft^2)	Clay (% <2µ)
Colluvium on Kope Formation						
Hillside Ave------	27 (4)*	18 (4)	20 (3)	108.5 (4)	2.6 (9)	19 (3)
East Miami River Road------	-------	-------	22 (30)	104.2 (47)	3.2 (39)	-------
Mt. Adams---------	46 (26)	23 (26)	24 (5)	100.4 (4)	2.6 (5)	50 (44)
Clifton-McMicken--	36 (1)	19 (1)	18 (4)	----------	--------	-------
Shale in Kope Formation						
Mt. Adams---------	43 (8)	19 (8)	5 (2)	----------	61.3 (2)	46 (7)
Clifton-McMicken--	-------	-------	5 (4)	142.0 (4)	20.8 (4)	-------

*Number in parentheses is number of determinations averaged to obtain indicated property.

appear to be in "critical natural stability" (Hough and Fleming, 1974), and areas of between 10- and 20-percent slope are susceptible to sliding if improperly graded.

Merritt (1975) has prepared a similar map for the Hamilton County Regional Planning Commission for the unincorporated areas of Hamilton County. In that study, he elected to consider slopes in the Kope Formation inclined at 15 percent (8.5°) or more to be in a potentially unstable state. His model for landslide susceptibility also considers presence or absence of vegetation and Fairmount soil as stability factors. The Fairmount soil as classified and mapped by the U.S. Department of Agriculture, Soil Conservation Service, appears to include the colluvial soils developed by weathering of the shales.

Generalized nature and distribution of glacial deposits

Landslides that occur in glacial deposits in the Greater Cincinnati area are not so readily predictable in terms of location and potential hazard as those associated with the Kope Formation because the distribution and nature of the glacial deposits are less predictable.

Southwestern Ohio was invaded by at least three continental glaciers that caused profound drainage changes and deposited a variety of materials in the valleys, along the hillsides, and on the upland surface. The distribution of the various materials is directly related to the sequence of glacial events and their effects. The following discussion, which is taken largely from Durrell (1961a, b) and Teller (1970, 1973), summarizes this history for the Cincinnati region.

Prior to the first ice advance in pre-Illinoian time, drainage was developed on a surface with 100 to 200 feet (30-60 m) of relief between upland divides at elevation 950 feet (290 m) MSL and drainageways. This drainage system, called the Teays system, flowed northward. The first ice advance into the Cincinnati area (called Kansan(?) or pre-Illinoian by different authors) caused ponding of the north-flowing streams, resulting in deposition of laminated clays in the lakes and initiation of major drainage changes. Erosional remnants of the clays are exposed near Claryville in Campbell County, Kentucky (fig. 1), and in some upland areas of Hamilton County, Ohio (Ettensohn, 1970; Teller, 1970). After retreat of the ice, the new river and many of its tributaries were cut several hundred feet below the Old Teays valleys. The new river, called the Deep Stage drainage system, established the modern course of the Ohio River except in the vicinity of Cincinnati (fig. 6). Here the river flowed northward up the present valley of the Little Miami River, westward through Norwood in what is now called the Norwood Trough, northward again in the present Mill Creek valley into Butler County near Hamilton, Ohio, and then southwestward in a large valley now occupied by Miami River and Whitewater River drainage (fig. 1). The major river in Deep Stage time joined the modern course of the Ohio River near Lawrenceburg, Indiana (fig. 1). Once established, this river was entrenched 450 feet (137 m) below the Teays-age valley levels, or about 100 feet (30 m) below the present Ohio River flood plain.

The next ice advance, the Illinoian, blocked the Deep Stage drainage and created a large lake upstream, that is south and east, from Hamilton, Ohio. Lake clays and silts were deposited in the lake up to about elevation 600 feet (180 m) MSL. The ponding was of sufficient duration that the long, winding lake rose in height until it overtopped a divide about 3 miles (5 km) west of downtown Cincinnati near Anderson's Ferry (fig. 1). This final diversion produced the modern channel of the Ohio River. The ice advanced over the lake to about the present position of the Ohio River, capping the lake clays and silts with a layer of till. Following retreat of the Illinoian glacier, much of the till and lake clay from the valleys that were upstream from the glacial dam was eroded, leaving terraces of these materials as remnants. These terrace levels are at about elevation 650 feet (200 m) along the Mill Creek valley.

The last ice advance, the Wisconsin, did not extend as far south as previous advances, so the drainage modifications were minimal. Melt water transported large amounts of sand and gravel into the drainageways, which buried the Illinoian deposits at lower elevations in the valleys. Since Wisconsin time these deposits of sand and gravel have been dissected into matched and unmatched terraces extending up to 540 feet (160 m) MSL. Figure 7 is a generalized cross section across the Mill Creek valley showing the dissected upland capped by Illinoian Till, the dissected Illinoian terraces capped by Illinoian Till, the Wisconsin terrace level, and the flood plain of Mill Creek.

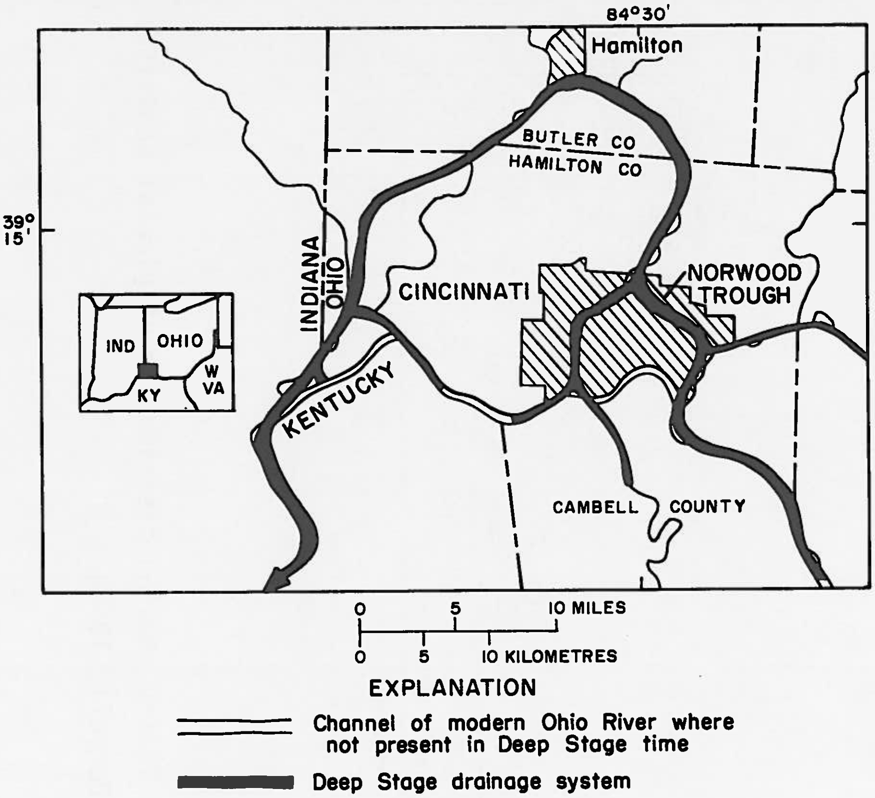

Figure 6.--Course of the Deep Stage drainage through Cincinnati.
(From Teller, 1973.)

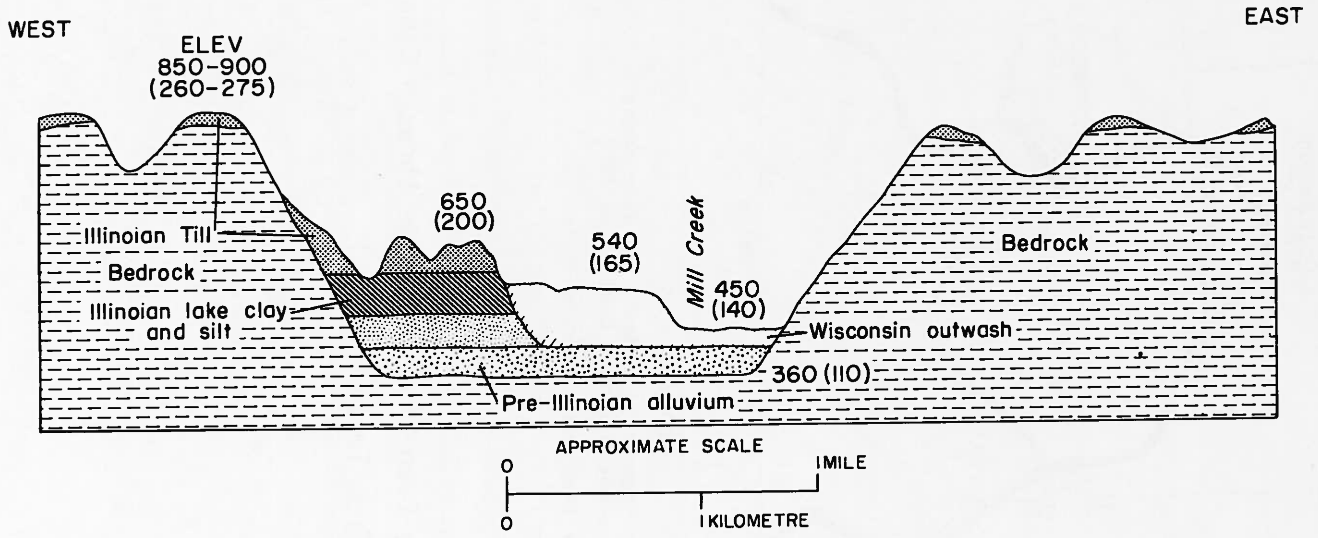

Figure 7.--Sketch of cross section of Mill Creek Valley showing distribution of glacial deposits. (Modified from Durrell, 1961b.)

Stability problems associated with the glacial deposits

Landslide problems in glacial materials are associated with the lake clays and silts and with the Illinoian Till.

Older lake clays deposited during the ponding of the Teays-age streams generally are in sparsely populated areas, and experience with excavations and fills in these materials is limited. Where they have been disturbed, principally around Claryville, Kentucky, the clays are somewhat unstable, but landslides are neither large nor widespread. Klusman (1973) collected several samples of the clays from that area and performed index tests on them. His results are shown in table 2, combined with similar data for glacial till.

The laminated silts and clays which were deposited as a result of the blockage of Deep Stage drainage by Illinoian ice are unstable in slopes. Furthermore, the outcrop elevations of these clays overlap with the Kope Formation and the clays commonly overlie the colluvium on the Kope Formation. A major landslide occurred in 1958 in Mill Creek valley at the Courter Technical High School (location 1, fig. 1), where excavation at the foot of the slope for a freeway triggered a failure several hundred feet long. The failure surface was both in colluvial materials on the Kope Formation and in the silts and clays (R. H. Durrell, oral commun., 1972). Nearby, the clays have failed along the east side of Interstate Highway 75, between Mitchell and Ludlow Avenues in Cincinnati. Here the failure surfaces are thought to be entirely in lake clays or in lake clays capped by Illinoian Till.

Because of the complex history of the lake clays, their distribution is not readily predictable. They occur in dissected terraces in Mt. Washington on the east edge of Cincinnati, in Mt. Adams near downtown Cincinnati, and along Mill Creek valley. Ponding extended to tributaries of the principal river in Deep Stage time, and these smaller valleys were subsequently covered with till, masking their location. At present not enough is known about the properties, distribution, or modes of failure to generalize as to their behavior patterns. Physical property data from one site of a buried tributary in northwestern Cincinnati are shown in table 2.

The glacial till of Illinoian age is widespread on the dissected upland and caps the upper-level terraces. No landslides have been observed in natural slopes in this material, but landslides in man-modified cuts and

Table 2.--Summary of physical property data for some glacial deposits

[Sources of data are listed in "Acknowledgments"]

	Liquid limit (% H_2O)	Plastic limit (% H_2O)	Moisture content (percent)	Dry density (lb/ft^3)	Unconfined compressive strength (t/ft^2)	Clay (% <2μ)
Illinoian Till						
Northwestern Cincinnati---	38 (7)*	20 (7)	19 (7)	105.8 (8)	2.2 (4)	------
North-central Cincinnati---	28 (4)	16 (4)	17 (4)	118.0 (4)	4.5 (4)	------
Teays-age lake clays						
Claryville, Kentucky-----	54 (8)	28 (8)	-------	---------	-------	82 (7)
Deep Stage-age lake clays						
Northwestern Cincinnati---	67 (5)	31 (5)	35 (10)	94.8 (2)	0.7 (2)	------

*Number in parentheses is number of determinations averaged to obtain indicated property.

fills are common. Of the 49 slides described by Yahne (1974) in the 9-square-mile (23-km^2) area on hill slopes and uplands about 1 mile (1.6 km) north of downtown Cincinnati, 16 occurred in glacial till as compared to 12 in colluvium on the Kope Formation. Another 20 landslides could not be ascribed to any particular formation, because they were failure of transported fills. However, the preponderance of this fill material was glacial till, and most of the fills were placed on glacial till.

These landslides have the morphology of a typical slump and are usually each a single, well-defined unit. The failure surface appears to be curved throughout, although not necessarily circular. The failure surface in some slides that are in fill appears to follow the contact between the fill and the underlying, undisturbed material.

Yahne (1974) could find no strong correlation between presence or absence of vegetation and distribution of slope failures. Only 4 of the 49 failed slopes faced southward, and more than half of them faced northward or northeastward. This probably reflects a tendency for north-facing slopes to be more moist and therefore not as strong as their south-facing counterparts.

The slope angles of till before sliding ranged from 23° to 38° (42-78 percent) for 13 of the 14 slopes; the average slope angle before sliding was 29° (55 percent). The other slope failed to such an extent that the original slope angle could not be determined. Table 2 summarizes some of the physical properties of the till.

Slope failures in Illinoian Till are usually associated with a small grading operation such as placement of material from a basement excavation into a backyard for fill or excavation of a small cut for access. In many of these instances the grading seemingly was performed without benefit of a stability analysis. Large earth-moving projects in glacial till rarely result in landsliding, especially when they are preceded by adequate analysis and control of construction.

Summary

Landslide problems are common and long standing in the Greater Cincinnati area. The preponderance of landslides occurs in colluvium derived from slaking of shales in the Kope Formation, in laminated clays and silts that were deposited in lakes in ancient valleys formed by blockage of the rivers by glacial ice, and in glacial till. The distribution of the Kope Formation is fairly well known and the location and nature of problems can be anticipated, but much more work is necessary to determine the distribution of glacial materials and the processes leading to their instability.

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IMPORTANCE OF GEOLOGIC STRUCTURE IN STABILITY OF ROCK SLOPES

By

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INTRODUCTION

Understanding the stability of a rock slope must be based on intelligent utilization of the science of geology and of engineering. Geology is important because the stability of a rock slope is dependent upon geologic structure, rock type, ground water, and other aspects of the geologic environment. Engineering plays a major role because the application of mechanics and of pertinent data on the physical properties of the rock comprising the slope is necessary if one is to adequately understand the stability of a slope.

This paper deals with the role of geology, and particularly of geologic structure, in the stability of rock slopes. As has been noted by several authors (for example, Terzaghi, 1950, 1962; Piteau, 1970, 1972; Robertson, 1970; Hoek, 1971; Patton and Deere, 1970, 1971; Call, 1972; Jennings, 1972; and Cruden, 1975), almost all failures in rock slopes are controlled by geologic structure and, more specifically, by structural discontinuities such as joints or faults in the rock. Such discontinuities control slope behavior under both static and dynamic loading.

In general, the stability of a rock slope is a direct function of the geometry and distribution of these discontinuities. As noted by Piteau (1970), the basic principles upon which rock-slope stability studies rest are:

> (1) the systems of jointing and other discontinuities, (2) their relationship to possible failure surfaces and (3) the strength parameters of the joints which includes an investigation of properties of both the joint planes and any infilling materials that occur in these. In addition to these three factors which are properties of the rock mass, one must add a fourth very important factor, namely water pressures in the joints.

A simple example illustrating that discontinuities commonly control the stability of rock slopes was presented by Terzaghi (1962). Terzaghi calculated the critical height, H_c, of a vertical face of relatively weak, intact (free of discontinuities) rock as being approximately:

$$H_c = \frac{q_u}{\gamma} = 4{,}200 \text{ feet } (1{,}290 \text{ m})$$

where the unconfined compressive strength, q_u, = 5,000 psi (34.5 MN/m^2), and the unit weight, γ, = 170 lb/ft^3 (2,720 kg/m^3). For intact hard rocks such as granite, q_u is several times greater than 5,000 psi; however, no vertical faces as high as 4,200 feet exist, and innumerable nonvertical slopes with a much lesser height than H_c have failed. This indicates that the critical height of rock slopes is reduced by the structural discontinuities of the rock, and not by the strength of the rock itself.

The relationship of stability of slopes to discontinuities in the materials underlying the slope is considerably different for rock than for soil. The slope behavior of soils, and especially of relatively soft soils, depends upon the properties of the intact soil, the shear strength being almost independent of discontinuities in the soil. Therefore, in soils failure tends to occur within the intact soil mass. In rock, on the other hand, the path of failure commonly follows preexisting surfaces or zones of weakness. Jennings (1972) concluded that there is some level of shear strength for intact geologic materials below which behavior is independent of the existence of any discontinuities and above which it is controlled almost entirely by the existence of such preexisting surfaces. Jennings suggested that this value of shear strength of the intact material is approximately 100 psi (700 kN/m^2), a value with which the authors agree. If a geologic material has a lower strength, it will tend to behave as a soil, with slope failures commonly occurring as classical cylindrical surfaces through intact material; thus, joints and other discontinuities will have reduced influence on the failure process. If, however, the intact material has a strength that is higher than about 100 psi, it will behave as a rock and failure will tend to occur along preexisting discontinuities.

STRUCTURAL DISCONTINUITIES IN ROCK

Types of structural discontinuities

In rock-slope engineering the term *discontinuity* is commonly applied to those geologic structural features, such as joints, faults, and bedding surfaces, which divide the rock mass into individual blocks. Although accepted in this context by most engineers, the use of discontinuity in this sense conflicts with common usage in the geologic literature. For example, the American Geological Institute's "Glossary of geology" (Gary and others, 1972) defines a structural geologic discontinuity as "a surface separating two unrelated groups of rocks; e.g. a fault or an unconformity." Obviously, the AGI definition is considerably narrower than the engineering definition in that it excludes joints, the most common category of discontinuity within the engineering definition. The authors prefer to use the broader definition by Hoek and Bray (1974, p. 38): "the term *discontinuity* will generally be used to define the structural weakness plane upon which movement can take place." A list of types of discontinuities encountered in rock-slope studies and their definitions is presented in table 1.

Continuous structural features such as faults which may be so weak that, compared with other discontinuities in the rock mass, they dominate the behavior of a particular slope, have been designated *major discontinuities* by Hoek and Bray. Multiple discontinuities with approximately the same inclination and orientation have been termed *discontinuity sets*.

Physical characteristics of discontinuities

The important physical characteristics or properties associated with structural discontinuities and of importance in stability of rock slopes are as follows:

(1) *Geometry*.--This parameter concerns the orientation and position in the rock mass of discontinuities. It is the most important of the physical characteristics. If joints are oriented critically with regard to potential slope failure, the effects of other physical characteristics probably are secondary. Discontinuities dipping in the same direction as the slope can result in instability; the problem becomes more critical as the strike of the discontinuities approaches that of the slope.

Table 1.--Types of discontinuities in rock commonly related to failure of slopes[1]

joint	A surface of actual or potential fracture or parting in a rock, without displacement; the surface is usually plane and often occurs with parallel joints to form part of a joint set.
tension joint	A joint that is a tension fracture.
shear joint	A joint that is a shear fracture; a potential plane of shear.
sheeting structure	The type of fracture or jointing formed by pressure-release jointing or exfoliation.
fault	A surface or zone of rock fracture along which there has been displacement, from a few centimeters to a few kilometers in scale.
shear zone	A tabular zone of rock that has been crushed and brecciated by many parallel fractures due to shear strain.
bedding surface	A surface, usually conspicuous, within a mass of stratified rock, representing an original surface of deposition; the surface of separation or interface between two adjacent beds of sedimentary rock. If the surface is more or less regular or nearly planar, it is called a bedding plane.
foliation	A general term for a planar arrangement of textural or structural features in any type of rock, e.g. cleavage in slate or schistocity in a metamorphic rock.

[1] Definitions are from the American Geological Institute, "Glossary of geology" (Gary and others, 1972).

(2) *Extent.*--Although it is a difficult property to measure, the extent (or continuity) of discontinuities is of considerable importance to the stability of rock slopes. Whereas major discontinuities such as faults extend for considerable distances, smaller features such as joints may be individually very limited in extent. If failure is to occur in a rock mass in which discontinuities terminate within the mass, it is necessary that some intact rock fail; thus, considerably larger forces must be involved than would be required for failure wholly along preexisting discontinuities.

(3) *Spacing.*--Discontinuities, and particularly joints, commonly develop in approximately parallel sets or combinations of sets. The spacing within these sets is a prime factor in determining the relative effects of the discontinuities and the intact rock on potential failure within the system. Close joint spacings commonly indicate a weak rock mass.

(4) *Surface irregularities.*--The nature of the irregularities along failure surfaces in soils seldom has to be considered in stability analysis. In rock, however, the irregularities along a discontinuity can mean the difference between stability and failure of a slope (Patton, 1966; Patton and Deere, 1970, 1971). Because failure of rock slopes commonly involves movement along discontinuities, the roughness of the surface of a discontinuity strongly influences this process. Surface irregularities, or *asperities*, as they are often called, affect the shear strength along a discontinuity by resisting shear movement, as shown schematically in figure 1. If a specimen of rock containing an irregular discontinuity is subjected to shear and normal loads, as illustrated, movement along the discontinuity can occur only if the irregularities ride over each other or if they are sheared through.

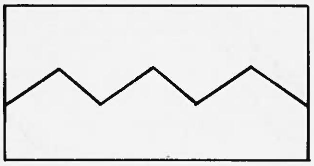
As shown in figure 2, two orders of surface irregularities are commonly recognized; these have been termed *first-order* and *second-order irregularities* by Deere and others (1967). First-order irregularities (waviness) are unlikely to shear off under the forces involved; instead, as shown in figure 1*b*, the direction of movement is modified as the irregularities ride over one another. Hoek and Bray (1974, p. 82) analyzed the forces involved as follows:

> In the case of the projections riding over one another, initial movement is no longer parallel to the shear stress τ but it takes place along a line inclined at an angle i to the direction of τ where i is the angle of incidence of the projections. Consequently, the shear and normal stresses acting along and perpendicular to the direction of movement must be considered.
>
> Resolving along the line of initial movement, the shear stress τ_m is given by
>
> $$\tau_m = \tau \cos i - \sigma \sin i \qquad [1]$$
>
> Similarly, the normal stress σ_m is obtained by resolving at right angles to the line of initial movement
>
> $$\sigma_m = \sigma \cos i + \tau \sin i \qquad [2]$$
>
> If the relationship between the shear stress τ_m required to cause movement is related to the normal stress σ_m by
>
> $$\tau_m = \sigma_m \tan\phi \qquad [3]$$
>
> where ϕ is the basic friction angle of the material and it is assumed that the surfaces have no cohesive strength, then equations [1] and [2] can be substituted into equation [3] giving
>
> $$\tau = \sigma \tan(\phi + i) \qquad [4]$$
>
> This relationship was confirmed on models with regular surface projections by Patton (1966) who must be credited with having emphasised the importance in rock slope stability analysis of the simple relationship presented in equation [4].

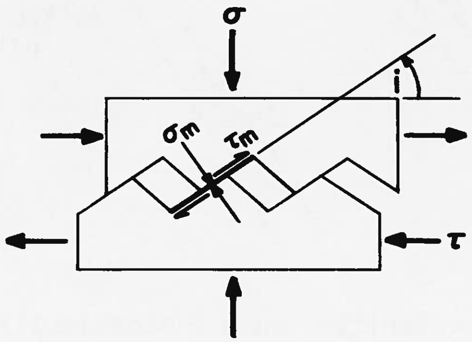
Second-order irregularities (roughness), being much smaller than first-order, are likely to undergo shear failure under high stresses. This shearing involves fracturing of the intact rock, as shown in figure 1*c*; hence, the surface will exhibit apparent cohesion, c, and its failure behavior can be defined by the simple Mohr-Coulomb equation

$$\tau = c + \sigma \tan\phi$$

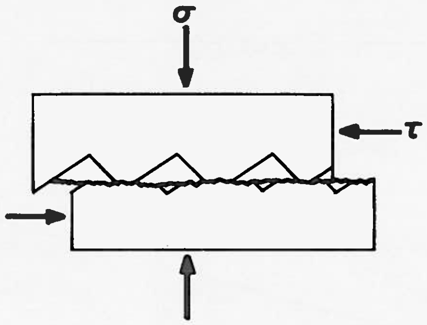
Idealized Mohr failure envelopes for the overriding and shearing cases, as well as for the case of failure along a smooth discontinuity, are shown in figure 3. Where previous shear displacement along a discontinuity has broken off or smoothed the surface irregularities, the shear strength will approximate the residual shear strength of figure 3 (Krsmanovic, 1967).



a. Diagrammatic sketch of surface irregularities



b. Movement by irregularities riding over one another



c. Movement by shearing through irregularities

Figure 1.--Simplified diagrams (cross sections) showing the influence of surface irregularities on shear movement along a discontinuity (modified from Hoek and Bray, 1974, p. 82).

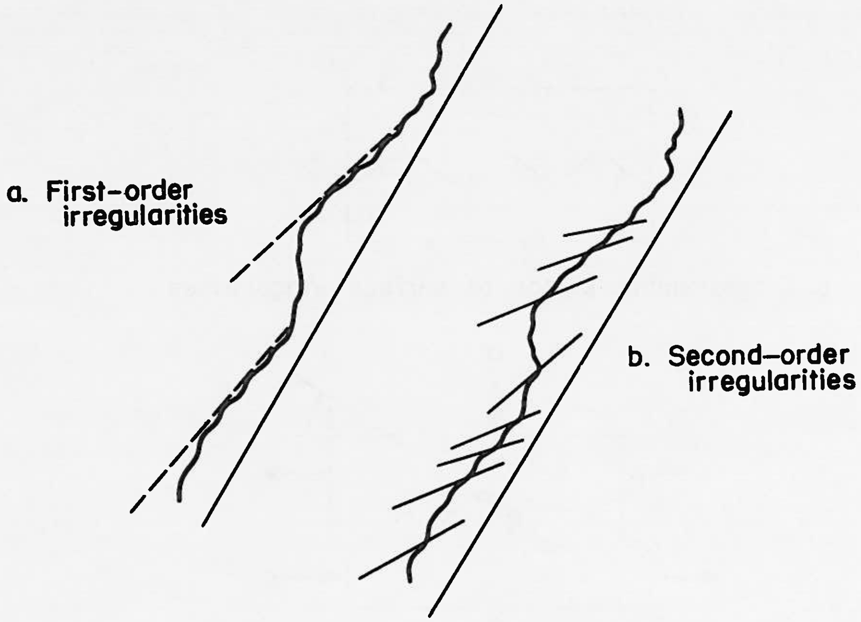


Figure 2.--Surface irregularities on a discontinuity in rock (modified from Patton and Deere, 1970, fig. 9).

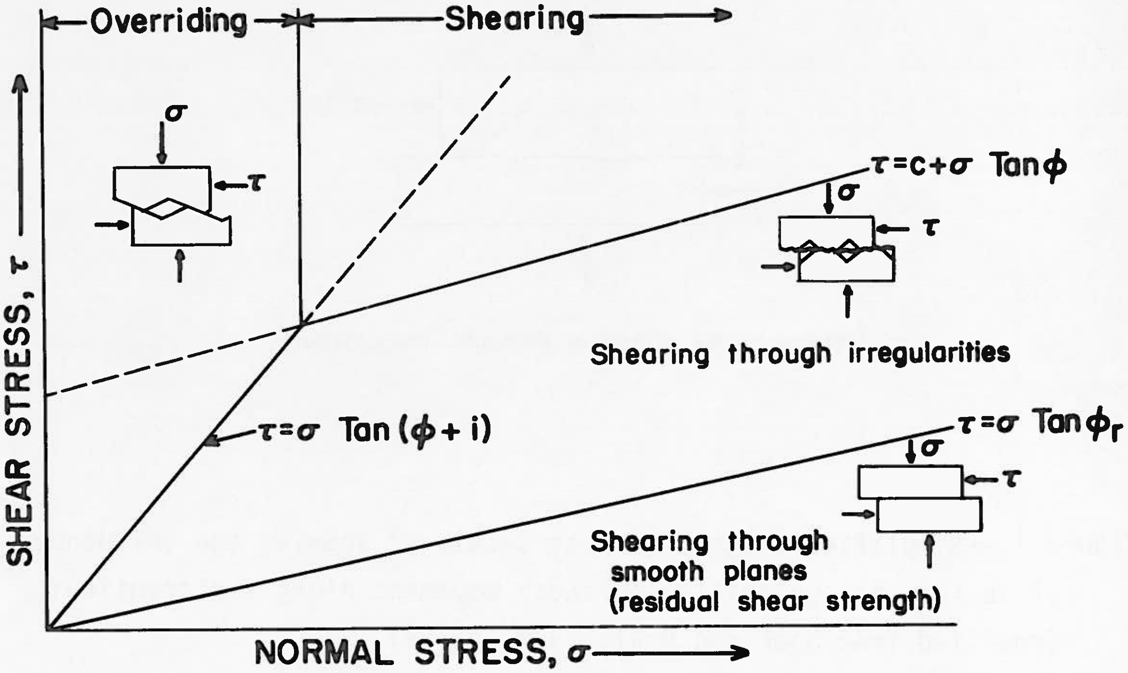


Figure 3.--Simplified Mohr envelopes for failure along discontinuities in rock (modified from Hoek and Bray, 1974, fig. 32).

(5) *Physical properties of adjacent rock.*--Because intact rock comprising surface irregularities commonly must be sheared off before movement can take place along a discontinuity, the shear strength of the rock may be very important to the stability of a rock slope. In those cases where movement occurs without shearing of irregularities, it is still necessary to overcome the frictional resistance developed between the opposing surfaces of the discontinuity. Different rock types and their alteration products develop different resistances to these shearing movements; thus, it is necessary to completely understand the pertinent physical properties of the rock involved, even though potential slope movement will occur along discontinuities rather than through intact rock.

(6) *Infilling materials.*--Infilling materials are defined here as any unconsolidated materials occurring between the walls of structural discontinuities. They include gouge developed from abrasion and shearing due to previous differential movement between the walls of the discontinuities, as well as sediment which has fallen, blown, or been washed or squeezed into the discontinuities.

The resistance to shear along a discontinuity containing infilling materials depends upon the physical properties and thickness of these materials, as well as on the characteristics of the walls of the discontinuity. For example, if the infilling material in a discontinuity is sufficiently thick, it is possible for the failure surface to occur exclusively within the infilling and for shearing resistance to be based entirely on the shear strength of the infilling material. Piteau (1970) presented four possible cases relating thickness and size of surface irregularities to possible shear failure along a discontinuity:

(a) The failure surface passes entirely through the infilling material; shear strength along the discontinuity is dependent on only the properties of the infilling material.

(b) The failure surface passes partly through infilling and partly through intact wallrock; shear strength is complex in that both infilling and wallrock will contribute to it.

(c) Infilling is present but is very thin; physical properties of the infilling material will only modify the shearing resistance developed between the rock walls of the discontinuity.

(d) No infilling is present; the shear strength along the discontinuity is dependent on only the properties of the wallrock.

Effects of ground water

The combined presence of ground water and discontinuities in rock masses causes most rock-slope failures. Hence, a thorough understanding of the character of the ground-water regime is necessary in any analysis of rock slopes, the most important factor being the pressure distribution of the ground water. According to Hoek and Bray (1974, p. 109), the presence of ground water in a rock slope can have the following detrimental effects on the slope:

(1) As is the case for soils, by far the most important effect of the presence of ground water in a rock mass is the reduction in shear strength resulting from fluid pressures along potential failure surfaces. Within a rock mass, the potential failure surfaces are the existing discontinuities; the water pressures act perpendicular to these surfaces. When there are many discontinuities with diverse orientations, the water pressure distribution within the rock mass can be treated in much the same way as it is for soil slopes. However, when the discontinuities have preferred orientations and when the spacing between them is large, abnormal distributions of water pressure result.

(2) As is the case for soils, presence of water increases the bulk unit weight of the rock mass and thus increases the driving forces which tend to cause slope failure.

(3) Freezing of ground water in open discontinuities can cause downslope movements due to expansion of the water. In addition, freezing of surface water on slopes can block surface drainage, resulting in buildup of water pressures within the slope.

(4) Erosion of infilling materials from open discontinuities can occur as a result of high-velocity flow of ground water. Removal of these materials can reduce the shearing resistance of the discontinuity.

ROCK-SLOPE MOVEMENTS AND THEIR RELATIONSHIP TO STRUCTURE

Many classifications of rock-slope movements have been proposed; for example, Sharpe (1938), Varnes (1958), Lacy (1963), Coates (1967), and Hoek and Bray (1974). In a widely adopted, comprehensive classification, Varnes (1958) divided slope movements into slides, falls, flows, and complex, depending on the kind and rate of movement, amount of water present, and the nature of the failure surface. Table 2 gives basic definitions and nomenclature from Varnes' classification. This paper recognizes some types of slope movements not included in Varnes' classification.

In rock-slope movements, structural discontinuities and their physical characteristics coupled with topographic expression determine whether failure will occur and, if so, the type of failure. The physical properties of the intact rock are often less important than the effects of structure. We now present examples of some of the most important types of rock-slope failures that are controlled by geologic discontinuities.

Sliding failures

The various types of sliding failures that are translational are controlled by structural discontinuities which are the major factors in determining the size, shape, and location of the moving rock mass. Most catastrophic landslides are in this class. The types of sliding failures dealt with here are block glides, rockslides, wedge failures, and slumps.

Block glides

Block glides, which are relatively uncommon, consist of large masses of rock that move down gently dipping discontinuities but still maintain internal integrity and approximate orientation. The sides of the blocks are generally controlled by fault planes or major joint sets. Braddock and Eicher (1962) presented examples of pre-Wisconsin block glides in the Dakota Group west of Fort Collins, Colorado.

Rockslides

If a gliding block undergoes internal deformation it may break apart and become a rockslide. The broken-up blocks may continue to slide very slowly along the slope or they may accelerate rapidly to form a rock avalanche, the most dramatic and potentially destructive type of mass wasting. For most avalanches it is probably impossible to tell (and makes

Table 2.--Classification of rock-slope movements (from Varnes, 1958)

I.	Falls	
	Mass in motion travels most of the distance through the air. Includes free fall, movement by leaps and bounds, and rolling of rock and debris fragments without much interaction of one fragment with another.	ROCKFALL
II.	Slides	
	Movement caused by finite shear failure along one or several surfaces which are visible or whose presence may be reasonably inferred.	
	A. Material not greatly deformed.	SLUMP (rotational) BLOCK GLIDE
	B. Material greatly deformed or consisting of many semi-independent units.	ROCKSLIDE (translational)
III.	Flows	
	Movements within displaced mass such that the form taken by moving material or the apparent distributions of velocities and displacements resemble those of viscous fluids. Slip surfaces within moving material are usually not visible or are short-lived. Boundary between moving mass and material in place may be sharp or a zone of distributed shear.	ROCK FRAGMENT FLOW ROCK AVALANCHE
IV.	Complex landslides	
	Movement is by a combination of two or more of the three principal types of movement described above. Many landslides are complex, although one type of movement generally predominates over the others at certain areas within a slide or at a particular time in the evolution of a slide.	

little difference) whether they are the result of large glide blocks breaking apart after moving short distances or of progressive failures proceeding rapidly from points of critical stress. Several case histories follow, showing the controlling influence of geologic structure.

Gros Ventre slide, Wyoming.--On June 23, 1925, 50 million yd^3 (38 million m^3) of rock moved 1 1/2 mi (2 1/2 km) along the dipslope of the Tensleep Sandstone on Sheep Mountain in northwestern Wyoming, crossed the Gros Ventre River, and rose 350 ft (107 m) up the opposite slope. This bedding-surface failure formed a dam 225 to 250 ft (69-76 m) high, and created a lake almost 5 mi (8 km) long. Two years later, on May 18, 1927, the lake overtopped the dam, causing a washout with serious flooding downstream.

The mass slid along the moderately dipping (18°-21°) bedding, probably on a bed of saturated clay shale (Legget, 1962; Sharpe, 1938). Figure 4 is a cross section of the Gros Ventre slide.

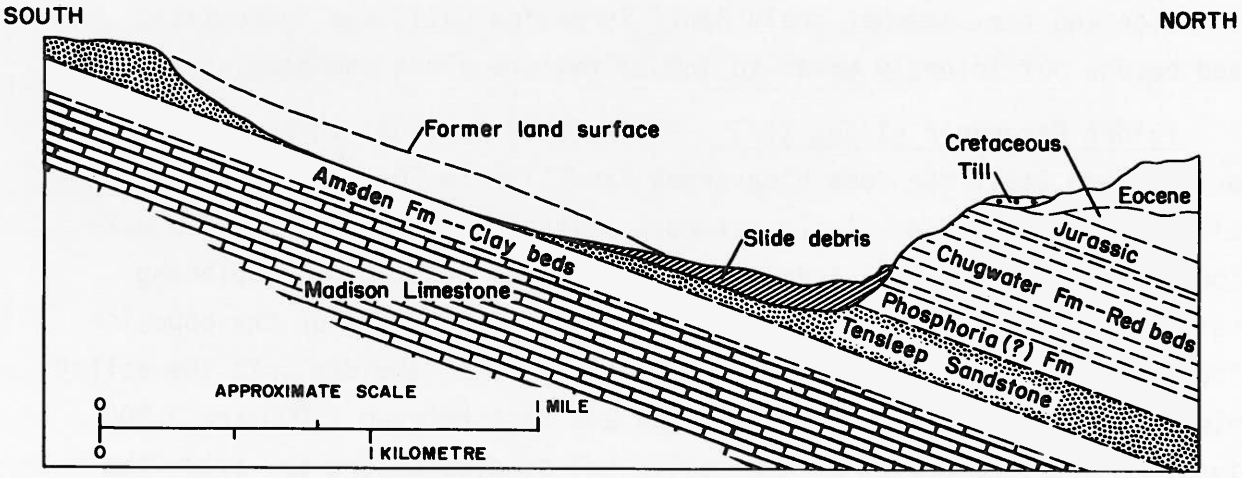


Figure 4.--Cross section of the Gros Ventre slide, Wyoming (from Alden, 1928, fig. 2).

Frank slide, Alberta.--On April 29, 1903, a large mass of rock broke loose from the south peak of Turtle Mountain in southern Alberta, rushed down the mountainside, destroyed much outlying property at the town of Frank, and killed about 70 persons. The slide was estimated to have a volume of 40 million yd^3 (30 million m^3) and to weigh 90 million tons (82 million metric tons) (McConnell and Brock, 1904).

For many years the Frank slide has been considered a classic example of a joint-controlled rockslide, the rock mass breaking at almost right angles to the bedding. This is based on the interpretation of Daly and others (1912) in their analysis of a potential failure of the north peak of Turtle Mountain (fig. 5*a*), said to be "identical" to conditions on the south peak. Contributing factors to the Frank slide were thought to be heavy spring rains in the preceding weeks and settlement caused by the mining of a coal seam at the base of the mountain. Coates (1967, p. 6-20 - 6-21) classified this slide as a *block flow*.

Recent mapping by Cruden and Krahn (1973) shows the structure of Turtle Mountain to be an anticline, and they have interpreted the movement as being principally along bedding planes (fig. 5*b*). On the basis of this interpretation, it seems likely to us that failure may have started along axial-plane fractures and proceeded along the bedding surfaces, which may have been infiltrated by rain. Another possibility is that runoff infiltrated axial-plane tension joints and the contact between the Livingstone Formation and the somewhat shaly Banff Formation until the hydrostatic head became sufficiently great to induce failure along the bedding surfaces.

Vaiont Reservoir slide, Italy.--On October 9, 1963, there occurred in northeastern Italy the most disastrous landslide in European history--the Vaiont Reservoir slide. A mass of rock having a volume of about 325 million yd^3 (250 million m^3) suddenly slid into the reservoir, displacing most of the water and sending a wave about 800 ft (260 m) up the opposite slope and at least 300 ft (100 m) over the crest of the dam into the valley below, where it destroyed five villages and took between 2,000 and 3,000 lives. A detailed review of the geological studies before and after the disaster was given by Müller (1964); other reviews have been made by Kiersch (1964) and Lane (1967). A remarkable point is that the thin-arch dam itself sustained little structural damage.

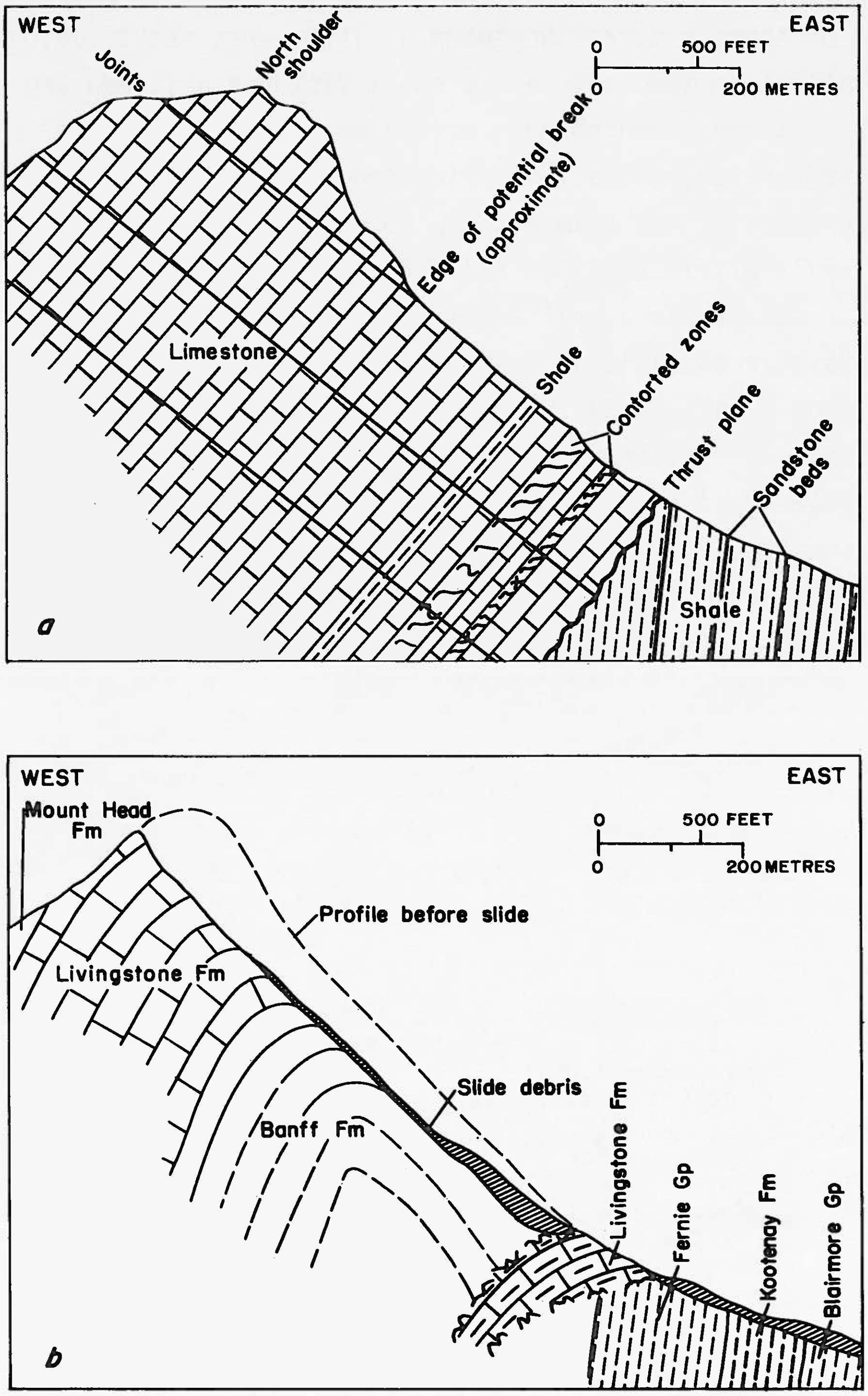


Figure 5.--Classical (*a*) and recent (*b*) interpretations of the geology of Turtle Mountain, Frank, Alberta (from Cruden and Krahn, 1973, figs. 1, 6).

The slopes above the reservoir had experienced other sizable failures earlier. One of these occurred November 4, 1960, when about 900,000 yd^3 (700,000 m^3) slid into the reservoir a short distance upstream from the dam. In 1960, a large creeping mass estimated at 260 million yd^3 (200 million m^3) was recognized, and a monitoring program was established to observe its progress; these observations were continued through three cycles of water-level fluctuations and until the disaster in 1963.

An adverse set of structural conditions controlled this slide. The Vaiont Gorge is near the axis of a syncline (fig. 6), and the sliding surface was largely defined by joints along the bedding surfaces. These bedding surfaces are bowl shaped and, because they intersect the valley wall, offer little resistance to gravity sliding (Kiersch, 1964, p. 34). A north-striking vertical joint set controls the eastern and western boundaries of the slide (Lo and others, 1972).

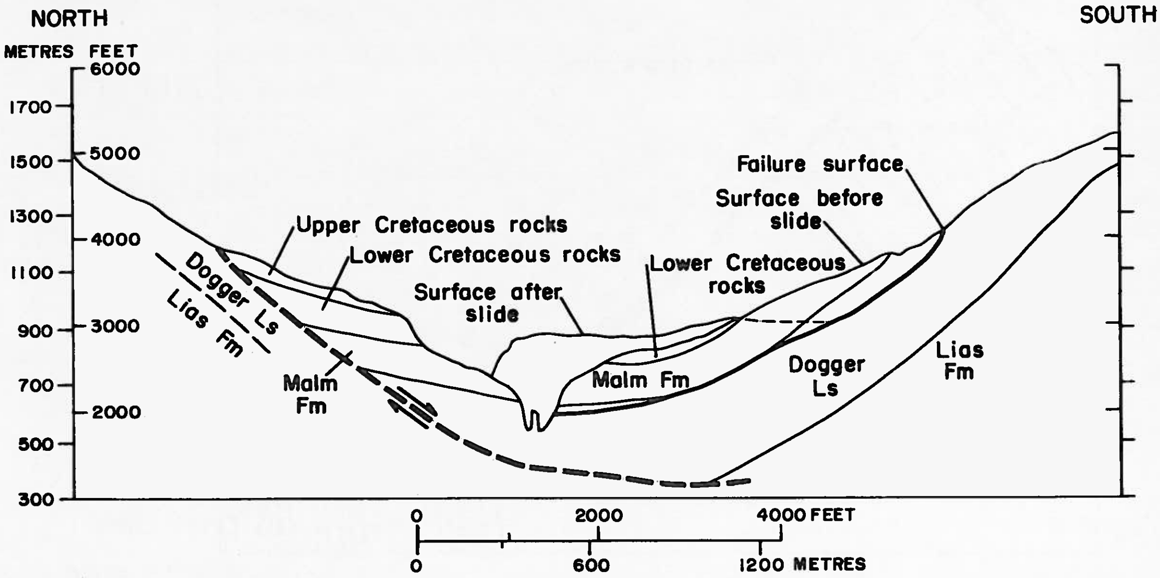


Figure 6.--Cross section of the Vaiont Reservoir slide (from Kiersch, 1964, fig. 5).

Wedge failures

If two roughly planar discontinuities join in an adverse orientation, such that the line of intersection of the two surfaces is inclined out of the slope, the possibility of a wedge failure exists. Such failures are relatively common in open-pit mines and may involve as much as several million cubic yards of rock.

Wedge failures lend themselves to mechanical analysis better than most other failure types. If the strike and dip of two planar structures are known, the bearing and plunge of the line of intersection and the dihedral angle between the planes can be readily calculated. By knowing the volume of a potential slide and the shear strengths of the surfaces involved, a complete stability analysis can be made. Detailed formulas and procedures for such an analysis were presented by Hoek and Bray (1974).

Slumps

Slumping is the classic failure mode in cohesive soils. Slump failures are rare in hard rock; however, they occur occasionally in flat-lying, weaker rocks without well-defined structural controls other than bedding. Indeed, it is often difficult in such cases to classify the material by behavior as either "rock" or "soil." Structures such as joints or faults may control, in part, the boundaries of slump areas.

A noteworthy example of a possible slump failure is the Downie slide on the Columbia River in British Columbia, about 170 mi (275 km) north of the U.S.-Canadian border (Piteau and others, 1975). This prehistoric slide has an estimated volume of 1 to 2 billion yd^3 (750-1,500 million m^3).

The toe of the slide extends along the west bank of the river for about 2 mi (3 km) between Priest Rapids and Twelve Mile Rapids, and the slide extends almost 3,400 ft (1,030 m) up the west bank. This is a massive slide about which little information has as yet been published, but it has been the subject of an extensive investigation during the last decade, partly because it is only 1 1/4 mi (2 km) downstream from the proposed Downie Dam. This is probably a complex slump failure with some controls by bedding and foliation surfaces (D. R. Piteau, oral commun., 1975).

Rockfalls

Rockfalls occur where individual blocks of loose rock fall or roll down the face and are probably the most common type of rock-slope "failure." However, such occurrences are usually more a nuisance than a major engineering problem. Rockfalls occur on almost all steep slopes in jointed rock, especially in formations where there are strong and weak layers. A common example is a resistant sandstone bed in a shaly unit, where the weaker material has been eroded by stream or wave action or slope wash, leaving an overhanging ledge which eventually breaks off and tumbles down the slope.

The controlling structural parameter in most actual or potential rockfalls is the joint spacing, as this determines the size of the blocks which may fall. Also, the danger of rockfalls increases if one of the joint sets controlling the blocks dips out of the face. Most steep slopes, especially those excavations subjected to the effects of blasting, experience some rockfalls from time to time, but simple corrective measures such as scaling loose rock and providing catch benches will lessen the chance of injury or damage to persons or facilities below.

A dramatic example of a major rockfall in hard schist occurred July 9, 1958, at Lituya Bay, Alaska (Miller, 1960). During or immediately following a strong earthquake, 40 million yd^3 (30 million m^3) of rock plunged a maximum of 3,000 ft (900 m) down the steep valley wall into a small arm at the head of the T-shaped bay. This fall caused a surge of water which destroyed the forest on the wall opposite the slide to a maximum elevation of 1,720 ft (525 m) and generated a gravity wave perhaps 250 ft (75 m) high which moved toward the mouth of the bay at a speed of at least 100 mph (160 km/h). This wave destroyed two of three boats in the bay, and killed two persons on one boat.

Topples

A less common mode of failure is that of toppling, where a column or slab of rock separates along discontinuities and simply overturns, owing to gravity; usually such failures are triggered by additional forces such as wind, water, and earthquakes. Erosion and creep of underlying strata may be contributing factors.

Such a failure mode has been proposed by Hoek and Bray (1974, p. 28-32) for the initial failure of the Frank, Alberta, slide and for part of the Vaiont, Italy, failure of November 1960.

Structural features that permit toppling failures are steeply dipping joint sets, with wide to moderate spacing in fairly competent rock, which allow slab and column development. If the joint spacing is close or the rock weak, failure will usually take place through small, isolated rockfalls.

CONCLUSIONS

Failures of rock slopes and the resulting loss of life and destruction of property can often be prevented. In order to achieve safer, more stable rock slopes, the major geologic structural features must be identified and evaluated with respect to the proposed construction. The engineer must appreciate and allow for the weakening effects of discontinuities in his design. Past experience has shown that the performance of rock slopes should be monitored *after* construction so that timely corrective measures may be taken as needed.

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CUT AND FILL ORDINANCE AS ADOPTED BY THE CITY OF CINCINNATI

by

JAMES R. KRUSLING, P.E.

INTRODUCTION

On October 30, 1974, Cincinnati's City Council passed Ordinance No. 447-1974, our "Cut and Fill" Ordinance. Cincinnati's cut and fill ordinance is by no means unique as many other cities including Madeira have ordinances regulating excavations and fills. The Uniform Building Code also contains provisions on this subject.

On October 6, 1965 City Council enacted legislation requiring a permit for filling. The primary purpose of this ordinance was to prevent obstruction of natural watercourses and to prevent damage to the sewer system. This ordinance served its intended purpose very well, however it soon became apparent that it was wholly inadequate to control fills causing hillside instability and the regulation of excavations was not covered at all. This was painfully brought to our attention by numerous landslides that occurred as the result of irresponsible fills and excavations.

In 1967 work began on an ordinance regulating both cuts and fills, however progress was slow due to the press of other work. The City Planning Commission's Hillside Preservation Studies and the evergrowing concern for the environment caused renewed interest in the project. Several committees produced numerous drafts of an ordinance. The most successful was a special Task Force of the Engineering Society of Cincinnati who in the Fall of 1973 were requested by the City to spearhead efforts to finalize a cut and fill ordinance. After numerous meetings, a draft of an ordinance was submitted to the City, March of 1974. This draft received the approval of the City Planning Commission on April 19, 1974 and was referred to City Council. The Council referred the ordinance to the Administration for comment and after several public hearings and much soul searching passed the ordinance with some slight modifications. To overcome some objections, largely from the homebuilders, the ordinance was amended by Council on April 9, 1975.

REQUIREMENTS OF THE PRESENT ORDINANCE:

A permit is required for any excavation or fill with the following exceptions:

1. Normal cemetery operations of opening or closing of graves.
2. Public work performed by or under the control of the Director of Public Works, except for excavation and filling beyond the project work limits. All other City Departments and other public agencies are required to go through the permit process but are not subject to the fee. While the Department of Public Works is exempt as the administering agency, the Director has ordered all Divisions in his Department to comply with the provisions of the ordinance.
3. Exploratory excavations under the direction of a registered professional engineer, soil engineer, engineering geologist, soil scientist, or architect where incidental to the practice of architecture, and exploratory excavations by a contractor or builder provided they are not made in a slope steeper than 5:1 and are promptly and properly filled.
4. Temporary excavations for wells, tanks, vaults, tunnels, sign foundations, and trenches for sewers, water lines, gas lines, electric lines, and other underground utilities.
5. Except where it is necessary to modify a portion of the existing sewer system any excavation that does not exceed 5 ft. in vertical depth at

its deepest point or 100 cubic yards per 5000 square feet of site area, whichever is more restrictive.

6. Except where it is necessary to modify a portion of the existing sewer system any fill that is not deeper than five feet nor more than 100 cubic yards per 5000 square feet of area and is placed on natural terrain with a slope flatter than 5:1 and does not result in a slope steeper than 3:1.
7. Except where it is necessary to modify a portion of the existing sewer system, any excavation for a basement and footings of a building authorized by a valid building permit provided the excavation is not greater than 8 feet deep nor 350 cubic yards per 5000 square feet of site area and is made in existing terrain with a slope flatter than 10:1 and the subsequent filling with this excavated material on the same site, provided the fill is not deeper than 5 feet nor 350 cubic yards per 5000 square feet of site area and is placed on existing terrain with a slope less than 10:1 and does not result in a finished slope steeper than 3:1. This provision eliminates most one and two family residences from the provisions of the ordinance, except when they are on hillsides.

You cannot be relieved of the responsibility of getting a permit by making a big job a series of small jobs falling within the above exceptions. In any event whether a permit is required or not, no excavation or fill shall cause any slope to become unstable, impose loads which may affect the safety of structures or slopes, interfere with drainage, adversely affect lawful sewers, cause a stagnant pond of water, or cause erosion or sedimentation.

If any excavation, fill, slope or other condition has become a hazard to public or private property, orders shall be issued to the

Property owner to correct this condition. The owner is to apply for a permit within ten days after receipt of the notice or else he is subject to the penalty provisions. If an engineer has been engaged within this ten day period we grant the owner a reasonable time to apply for a permit.

The person who wants to make a cut or fill not covered by the exclusions is required to submit plans and specifications showing what is proposed. Unless waived, the plans shall be prepared by a registered professional engineer. Since state law permits architects to practice engineering where it is incidental to the practice of architecture, they would be allowed to prepare the plans if the cut and fill were incidental to their architectural project. Unless waived, the plans and specifications submitted with the permit application shall:

1. Include the owner's name and address; if the applicant is not the owner of the property upon which the excavation or filling is to be done, he must submit a letter from the owner to the applicant granting permission for the work;
2. Include a plot plan, drawn to scale, showing the location of the proposed work;
3. Include a contour map of the affected area showing the existing and proposed contours at 5-foot intervals;
4. Show the proposed amount of excavation or fill in cubic yards;
5. Show the location of any existing and proposed streets;
6. Show the location of any existing and proposed buildings or structures on the subject property and within 15 feet of subject property;
7. Show the location of any existing watercourses, drainage, and sewer systems serving the property;
8. Show existing and proposed drainage structures, walls, cribbing and

surface protection, and any necessary temporary earth restraining installations;

9. Show a plan for temporary and permanent drainage of the property, including any new or altered sewer systems;
10. Describe the proposed method for the protection of the soils from erosion;
11. Show additional information as may reasonably be required by the director.

The following requirements may be waived or modified if the information submitted above is considered sufficient to adequately evaluate the application:

1. Include a report showing the results of surface and sub-surface exploration, conditions of the land, and procedures for performing the operation;
2. Show plans of all drainge provisions which shall be of such design to adequately handle the surface run-off, together with a map showing the drainage area of all land tributary to the site, and estimated cubic foot per second runoff of the area served by any drain computed in accordance with current acceptable standards;
3. Include a description of the borrow material, and the method to be used for an the degree of its proposed compaction;
4. Show proposed preparation of existing ground surface to receive fill;
5. Show proposed terraces and ditches where necessary to control surface drainage and debris;
6. Show proposed sub-surface drainage if necessary for stability;
7. Show plans for all retaining walls, cribbing, vegetative provisions, erosions and sediment control measures, together with location of

temporary and/or permanent fencing and other protective devices to be constructed in connection with, or as a part of the proposed work;

8. Show a timing schedule and sequence indicating the anticipated starting and completion dates of the development sequence--stripping and/or clearing, rough grading and construction, final grading and vegetative establishment, and maintenance and the time of exposure of each area prior to the completion of effective erosion and sediment control measures.

Unless waived, excavation and fill in the field shall be supervised by a registered professional engineer, an architect where the excavation and fill is incidental to the practice of architecture, or their representatives. The registered professional engineer or an architect shall submit a summary report to the director upon completion of operations.

The requirements for plans and specifications, and field supervision and summary report may be waived or modified if the application for a permit or a certification in writing of a registered professional engineer or an architect where incidental to the practice of architecture states, to the satisfaction of the director, that the proposed excavation or fill will not:

(a) Interfere with adequate drainage for the site area and the drainage area of land tributary to the site.

(b) Obstruct, damage, or adversely affect existing sewerage or drainage, public or private;

(c) Cause a stagnant pond of water to form;

(d) Create slope stability problems on subject and adjacent property;

(e) Cause detrimental erosion or sedimentation; or

(f) That the proposed excavation or fill is in an isolated, self-contained area and that there is no apparent danger to adjacent public or private property.

The ordinance, in effect, mandates what a reasonably prudent owner or developer would do on his own. That is, get competent technical advice.

Excavation and fill slopes cannot be made steeper than 3:1 unless there is appropriate exploration and analysis and a written opinion by a registered professional engineer that the resulting slope will not be hazardous or result in erosion and sedimentation problems.

The owner may be required to modify the existing sewer system at his own expense if found necessary to provide adequate drainage or provide a one-year bond to cover replacement cost if the sewer is of doubtful strength.

A bond may also be required during construction if hazards are determined to exist during this time to cover the potential damages.

The fee charged for the permit is $10.00 for the first 100 cubic yards plus $10.00 for each 1000 cubic yards above the first 100. The fee schedule is arbitrary because of the difficulty of projecting the number of permits to be issued within a given period. After we have the benefit of some experience the permit fees will probably be modified.

The permit is valid for one year and may be extended for two additional one-year periods at no additional cost at the discretion of the Director. The extension provision is to cover large projects that take longer than a single construction season.

The permit holder is responsible for providing notice of start and completion of operations and operations may be suspended if it is determined that the work is endangering public health and safety. The suspension would remain in effect until the hazardous conditions are corrected. The work may also be suspended when the operations are contrary to the terms of the permit.

No building permits can be issued until the owner has complied with these regulations.

Since discretionary powers are granted in the administration of the Ordinance, an appeals procedure has been established. The grounds for appeal are:

(a) That the action of the director was erroneous or constituted an erroneous application of the provisions of this ordinance, related laws and ordinances, or was otherwise contrary to law;

(b) That the action of the director imposes an undue hardship on the complainant, and a modified application or an alternative arrangement is available and feasible; whereby the hardship can be relieved without defeating the purpose and intent of the provisions of this ordinance.

These appeals are to be heard by an Engineering Board of Appeals as established in the City's Administrative Code. The Board is composed of seven members; an engineering geologist, two registered professional engineers, experienced in the practice of soil engineering, a builder, a registered architect and two persons selected from the general public. The appointments are made by the City Manager.

Violation of the ordinance can subject one to a fine not less than five dollars nor more than five hundred dollars. Each day's violation constitutues a separate offense. The original ordinance called for a fine or imprisonment up to six months.

ADMINISTRATION OF THE ORDINANCE

The detailed administration of the ordinance is the responsibility of the Soils Engineering Office of The Engineering Division of the Department of Public Works.

In the eight months since passage of the ordinance, over 230 Building Plans have been reviewed by this office and 110 excavation-fill permits for work ranging from minor grading to construction of major retaining walls, have been issued. Prior to issuing many of the permits, there have been numerous contacts and meetings with contractors, owners, engineers, architects and their representatives to bring about the necessary plan revisions to insure compliance with the ordinance. In general, we have enjoyed an amicable working relationship with the building industry, however, in some instances, we have had to be quite insistant on certain plan revisions before issuing the permit. This is especially true where additional cost is involved. Apparently we haven't been too arbitrary since no one has asked for a review by the Engineering Board of Appeals.

In addition to reviewing plans for private projects this office has also reviewed plans for public improvements such as highways, Metropolitan Sewer District and Urban Development projects. The increased surveillance by the Department of Buildings and Inspections in their effort to eliminate housing blight has resulted in the demolition of many condemned buildings. We have been following these demolitions to insure that the removal of the buildings and debris from the premises is in an orderly manner and that the discarded material is deposited in a designated area, and that the basements are filled in a manner to preclude future erosion and slide problems. Periodic checks are also made of the work sites to insure compliance with the terms of the permit.

The widespread coverage by the news media has aroused the concern of many citizens about landslides. This has served to generate numerous complaints, especially during periods of heavy rain. Most complaints are of a minor nature and are resolved by field investigation and corrections suggested by our personnel on the site. More involved problems require the services of an engineer. At the request of private property owners, the Hamilton County Soil and Water Conservation District have made field investigations and recommendations regarding problems that are related to erosion. The services of the district have also been utilized by our personnel in evaluating permit applications, particularly in the area of erosion.

Frequent contacts are made with property owners where conditions hazardous to public or private property exist in order to get them corrected. Most property owners are cooperative and comply with the orders. Occasionally the orders are ignored and the matter is turned over to the City Solicitor for appropriate action. In most instances, after receiving a threatening letter from the Solicitor, the property owner decides to cooperate. Unfortunately, certain public agencies, the Federal Housing Administration and the Cincinnati Metropolitan Housing Authority, have failed to comply with orders dealing with hazardous conditions on their property. The question of whether or not the ordinance is enforcable against other governmental agencies remains to be resolved.

While the ordinance permits architects to submit plans where the excavation or fill is incidental to the practice of architecture, we have recommended that they engage an engineer when problems are anticipated.

The workload of this office, consisting of reviewing plans for building permits, excavation-fill permits, plans for public improvements, investigating existing slides and complaints and monitoring the work done under existing permits continues to grow daily. The permit fees collected in the eight months since passage of the ordinance approximately equals $4,000.00, which is much less than the cost of administration. The cost differential is more than justified when you consider the cost of damages to public and private property due to irresponsible actions in the past. I have no illusions that this ordinance will prevent all future landslides, erosion and sedimentation problems in Cincinnati. I feel the enforcement of this ordinance will drastically reduce these damages.

I would like to express my appreciation to James W. Johns, P.E., Head of the Structures Section for his efforts in the development of the ordinance and its administration, and to Ram Jindal, P.E., our Soils Engineer, for making it work.

DRILLED PIER RETAINING WALLS

BY

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INTRODUCTION

Much has been published in the texts and journals on the subject of "Laterally Loaded Piles". The purpose of this presentation is not to convey a "state of the art" summary on the subject, but rather to present a local application of the principles involved and how they have been put to use in solving particular types of "soil" slope stability problems.

DISCUSSION

Driven timber and steel piling have long been used by Highway Departments and others as a means of increasing the shear strength of a failing mass by multiple driving of individual units into a slide area. This has been successful in arresting further movement in many cases. Where the materials contained rock slabs or boulders or were underlain by bedrock or hardpan, rarely were the piling able to penetrate the failure surface and thereby utilize the available resisting strength of the underlying stable mass, as needed to create stability. Under such circumstances sliding continued to occur. With recent development of heavy duty, high torque, pier drilling equipment and a wide variety of rock bits. the capability emerged for installing a reinforced concrete structural cantilever which could be designed to penetrate the stable underlying layer a sufficient depth to create stability within the overlying failing mass. This presentation limits itself to this form of structural restraint. However, the same principles can also be applied to other forms of structure; i.e., intrusion grout reinforced piling, predrilled sockets for steel rail or pile sections, tied back members and others.

Drilled piers have been used in the correction of a wide variety of slope stability problems. Locally they were first used on an experimental basis, for correction of highway berm (fill) slope failures, drilling being accomplished from the remnant edge of paving or along a temporary access road. In recent years their use has included:

1. providing temporary restraint of an open excavation for construction of a structure

2. installation behind a failing 20 ft. high masonry gravity wall to arrest movement of a continuing failed slope above

3. provide "toe" restraint below a failing fill and/or natural slope immediately adjacent to and below occupied structures or traveled roadways

4. provide restraint of sloping overburden so that an immediately adjacent 50 ft. deep near vertical rock cut (unrestrained) could be made and later backfilled following construction of the structure --- plus others.

The suitability of using drilled piers on a given project is dependent upon a number of factors. These include:

1. relative economics in terms of other alternate types of structures or grading schemes

2. limitations in accessibility for the equipment needed for both drilling and concreting

3. supplemental measures necessary for restoration of grade "upslope" from the piers

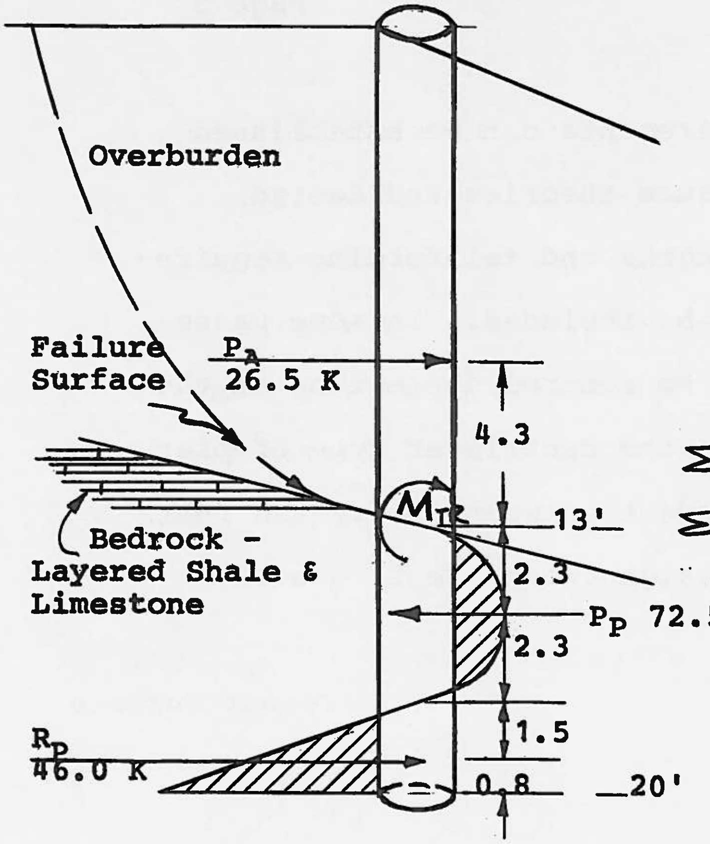
4. features downslope which would be influenced by a continuation in movement of the materials below the piers

5. aesthetic requirements of the immediate and surrounding vicinity.

DESIGN

Drilled pier retaining walls rely upon the principle of "soil arching" between piers to enable a series of spaced cylindrical columns or shafts to resist soil movement through the space in between. Refer to attached Plates I and II. Pier diameters have varied from 18 to 30" where the depth to the failure surface is generally less than 20 ft. Pier diameters as great as 10 ft. have been considered for depths approaching 50 ft. The spacing between piers is a function of the type of fill or natural materials being restrained, ground water conditions, slope ratio and other factors. For average conditions involving cohesive or mixed soils adequate arching develops between 24" or 30" diameter piers spaced 5 ft. on center.

Longitudinal reinforcing requirements can be established using conventional earth pressure theories and design methods. To minimize pier lengths and reinforcing requirements, anchor "ties" can also be included. In some cases "ties" may actually be needed to control deflection at the top. Current experience using the cantilever type of pier wall suggests reasonable agreement between theory and performance when the following design procedure is used.



Given: 24" Ø Piers (cantilever)
5' c. to c., 20' long
7' rock penetration

Find: Moment at top of rock, rock stress and F.S.

$P_A = ka \gamma_t \frac{H^2}{2} =$

$0.5 \times .125 \frac{\times 13^2}{2} = 5.3$ K/ft.

@ 5' c. to c. = $\underline{26.5^K}$

$M_{tr} = 26.5 \times 4.3 = 114'$ K

$\Sigma M_P = 0 \quad 26.5(6.6) - R(3.8) = 0$

$\underline{R = 46.0^K}$

$\Sigma F_x = 0 \quad P_A + R_P - P_P = 0$

$\underline{P_P = 72.5^K}$

P avg. = $\frac{72.5}{4.6} \times 2.0 = 7.8$ KSF

P max. = 1.5 x P avg. = 11.8 KSF

@P qu = 15 tsf (from Lab. Tests)

Passive Pressure = $2c + \gamma H = 2 \times 15 + .130 \times 15.3 = 32.0$ KSF

$F_S = \frac{P_P}{P \text{ max.}} = \frac{32}{11.8} = 2.7$ (OK)

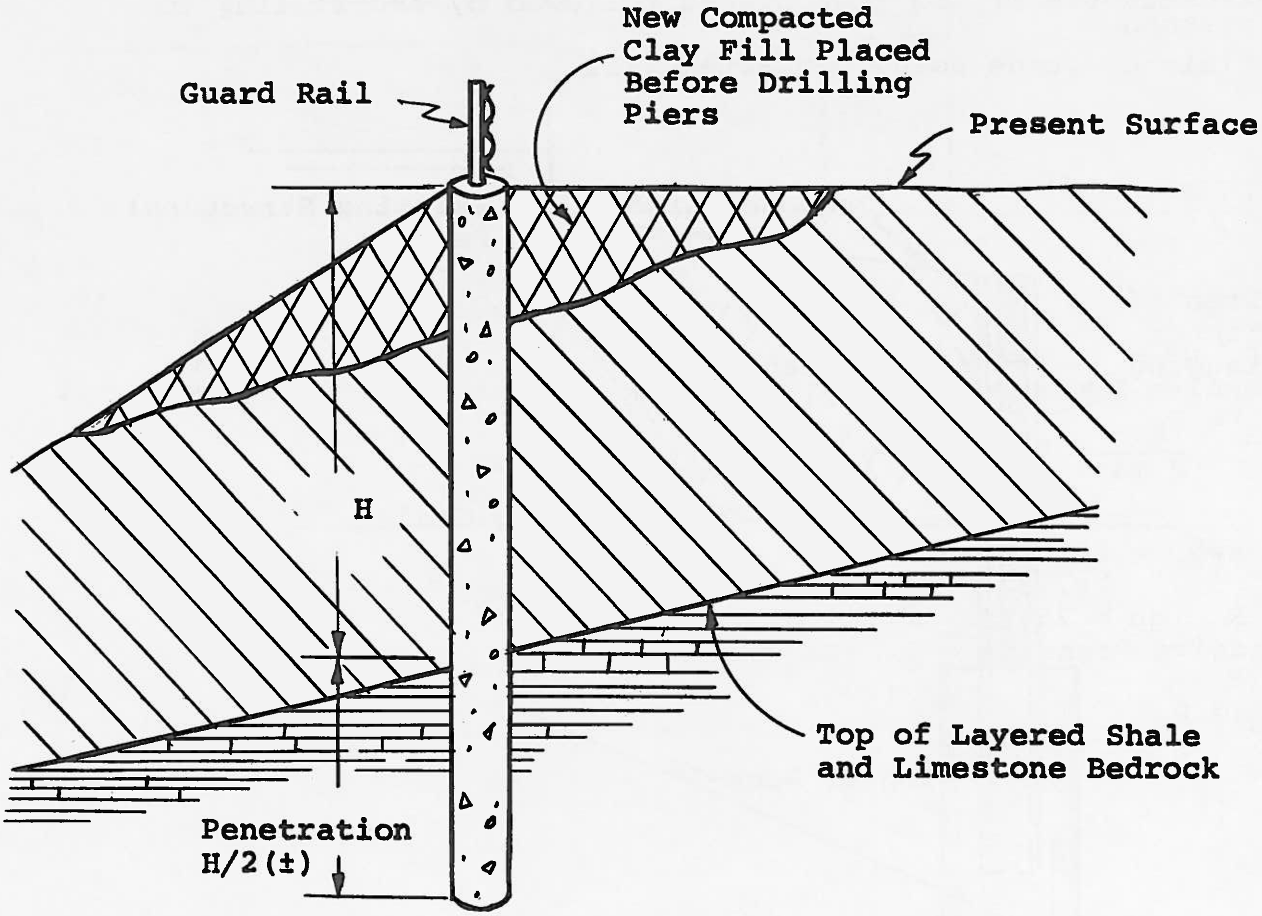
R avg. = $\frac{46.0}{2 \times 2.3} = 10$ KSF R max. = 2 R avg. = 20 KSF

@ R qu = 25 TSF

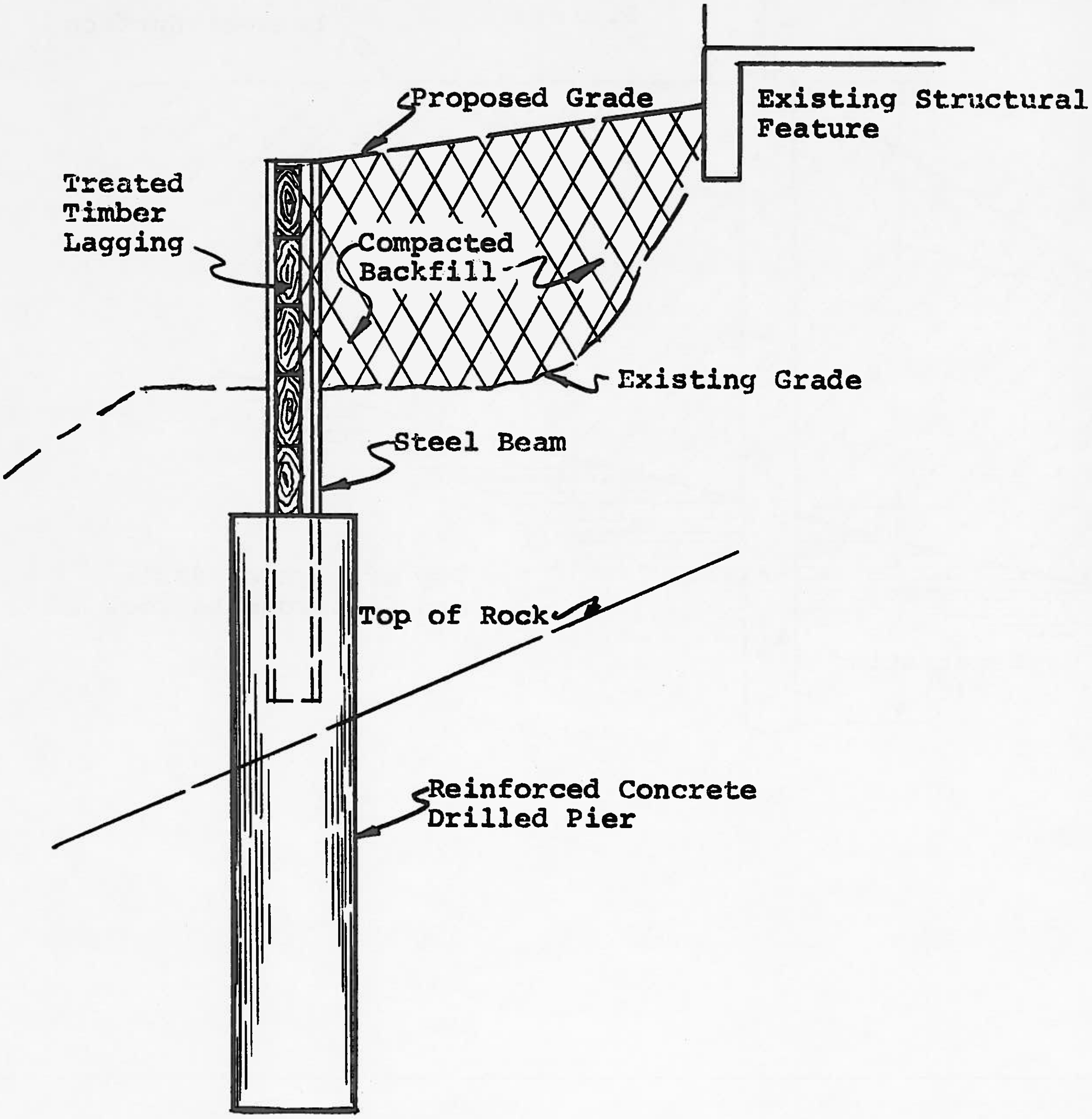
Passive Pressure = $2_C + \gamma H = 2 \times 25 + .130(19.1) = 52.5$ KSF

$F_S = \frac{P_P}{R \text{ max.}} = \frac{52.5}{20.0} = 2.6$ (OK)

Various means have been used to restore the grade adjacent to and upslope from the pier wall structure. Ideally, compacted cohesive fill is first placed along the center line of piers to approximate finished grade. A single line of spaced unconnected reinforced concrete piers are then installed, guardrail support sometimes being inserted in the fresh concrete.



Where the piers are installed at some lower elevation on the slope and must have backfill placed upslope to create finished grade, then the piers are drilled from a temporary access road or bench and extended above the grade. The projective portion can consist of formed reinforced concrete or structural steel sections embedded in the concrete poured below grade. Lagging between piers, either precast concrete or treated timber are then placed followed by backfilling to finished grade using compacted fill.



COST

Current escalation in the cost of construction makes the estimation of unit prices for a given design and project requirements difficult at best. Primarily due to the high cost of reinforcing steel and its fabrication, pier walls using 30" diameter shafts have been the same cost or cheaper than those using a 24" diameter alternate. A typical comparative estimate is as follows. This does not include cost of grading or appurtenant structure.

ITEM	QUANTITY	COST	
		24"Ø	30"Ø
Drilling & Concrete In-place (Soil & Rock)	709 L.F.	$17/L.F. - $12,053	$24/L.F. - $17,016
Deformed Bar Reinforcing In-place	50,750 lb.	$0.40/Lb. - $20,300	
	37,950 lb.		$0.40 lb. -$15,180
TOTAL COST		$32,353	$32,196
Cost/Cu.Yd. In-place*		$ 392.39	$ 249.72

*Actual job costs suggest a range of $250 to $300/cu. yd. in-place depending upon project size, mobilization difficulties, etc.

CONCLUSIONS

Drilled piers have been effectively used for correction of failed slopes where site conditions are appropriate for this form of earth restraining structure. They have an advantage over conventional gravity structures in that:

1. the need for preliminary excavation to expose the bearing surface is eliminated

2. the concept permits the development of resistance within more competent rock at greater depth and does not limit the stability by the shear strengths along weak planes that are typically found near the surface of shale formations.

3. the quantity of backfill in comparison is significantly less

4. restraint of the temporary cut slope or worry over its stability until backfill can be placed is eliminated

5. construction time is usually one-half or less

6. total cost is usually 1/2 to 2/3 that of a gravity (crib wall) structure.

DRILLED SHAFTS (PIERS) FOR STABILIZING FAILED SLOPES

(TYPICAL SECTION)

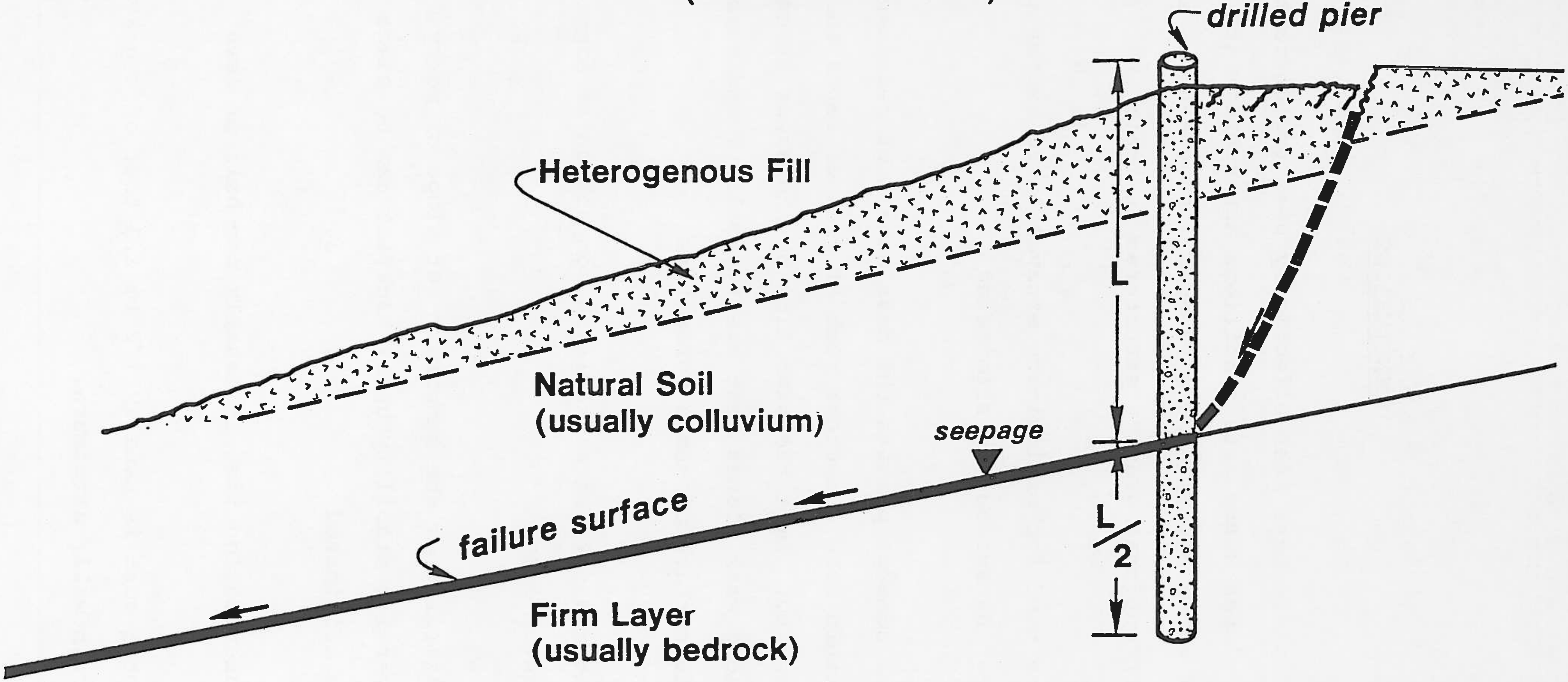


PLATE I

LONGITUDINAL PIER SPACING

(TYPICAL)

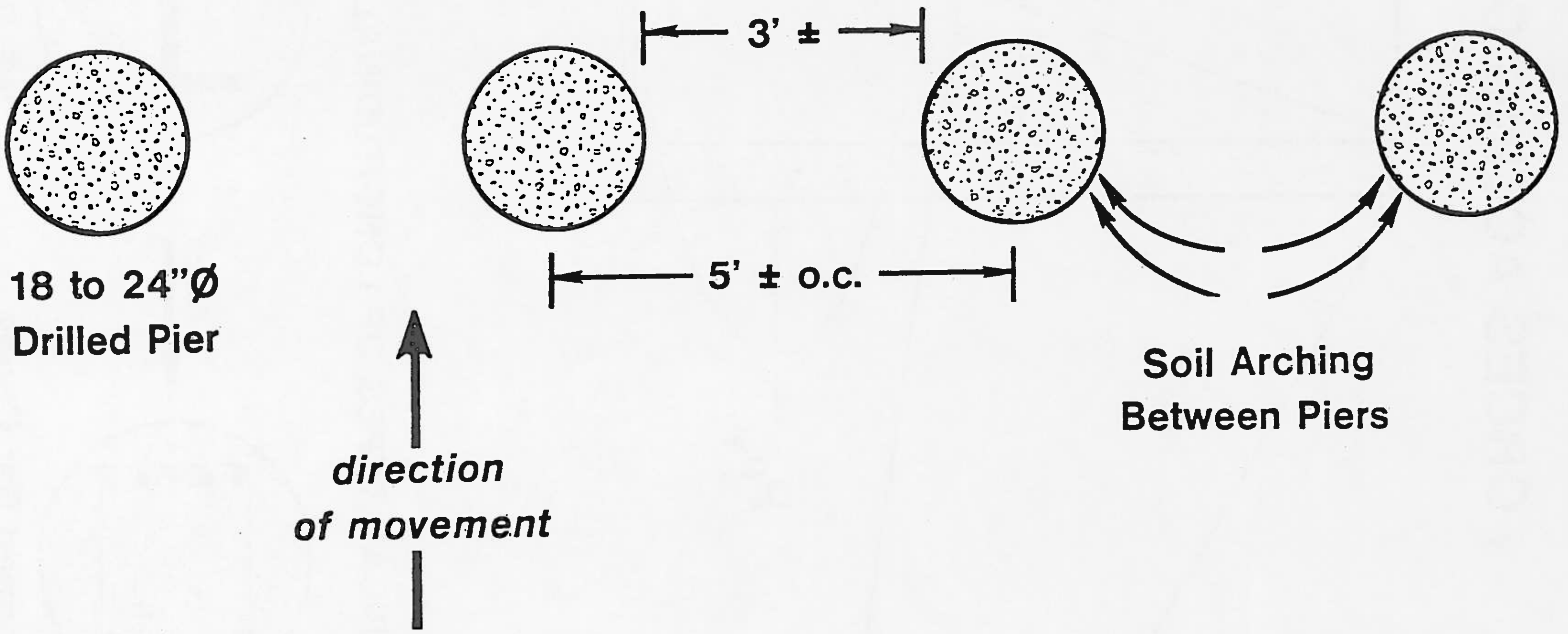
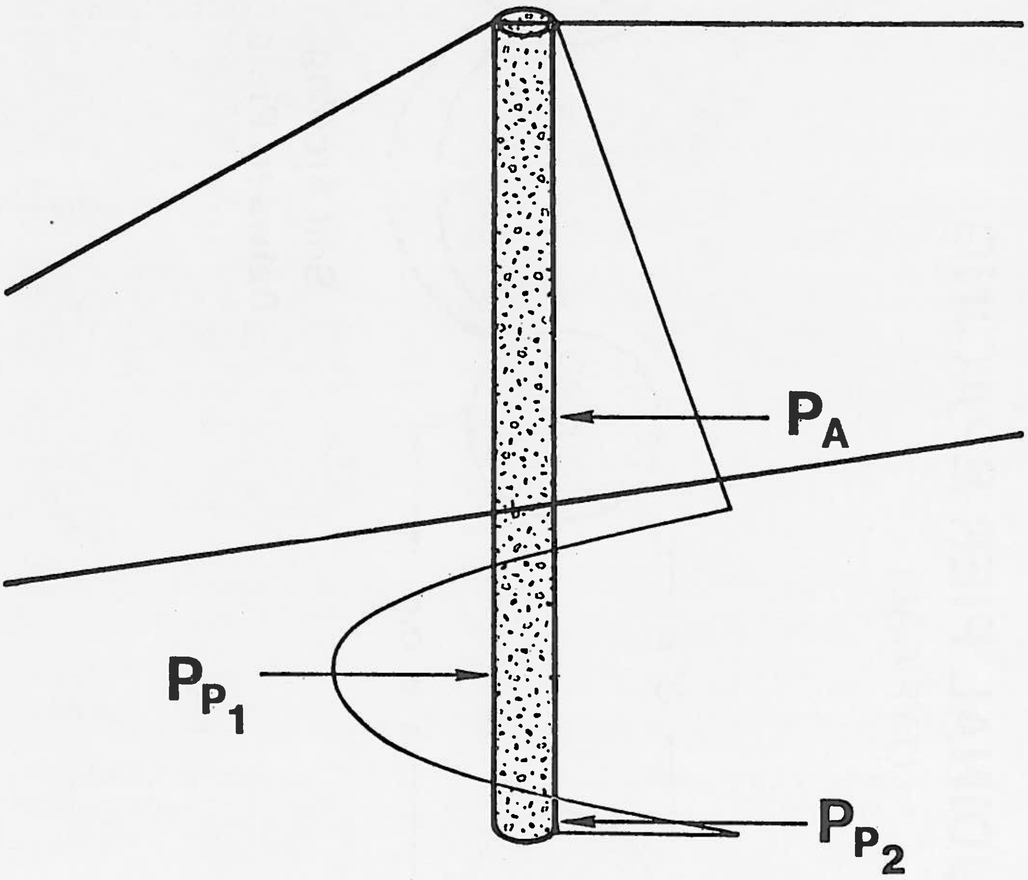
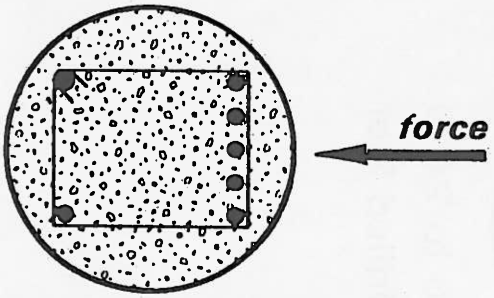


PLATE II

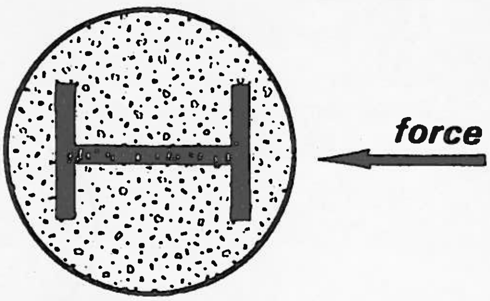
FORCES ACTING ON PIER



TYPICAL TYPES OF LONGITUDINAL REINFORCING



Deformed Bar Cages



WF Beams

PLATE III

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REGRADING FAILED SLOPES

by

H. A. Mathis

The subject project is located in Campbell County, Kentucky approximately .25 miles south of I-275 and relocated KY 9 Interchange. The portion of the project under investigation consists of relocating and upgrading KY 9 to a connector road for I-275. The existing KY 9, prior to any construction, was a two-lane road located in a side-hill condition in an area with a history of instability. The upgrading and relocation of the roadway consisted of moving the roadway into the hill and building a four-lane facility. This would require an additional twenty feet of fill into the existing fill of KY 9.

Construction of relocated KY 9 began in the summer of 1973. During construction much water was encountered and perforated pipe was installed in these areas. There were no rock benches constructed, only soil benches. When the new embankment was constructed to approximate final grade, large cracks developed in existing KY 9 and in the new embankment.

An inspection was held at the subject slide on August 28, 1973 by Division of Materials personnel. It was the opinion of those present that a sub-surface investigation would be necessary prior to recommending any corrective methods for the landslide.

In order to determine the magnitude and probable cause of the slide, a total of seventeen sample borings were obtained throughout the slide area. Thin walled tube and standard penetration test samples were obtained from these borings. A total of five slope inclinometers were installed on two critical cross sections to determine the magnitude and direction of the slide. Three of the inclinometers were installed in the portion of the slide that was obviously moving more rapidly, and two were installed in an area of the slide expected to be moving more slowly. Movement was detected in all five slope inclinometers and in all cases the bottom of the movement was located in soft soil-like shale coinciding with the existing water table.

From slope inclinometer data the failure was determined to be a wedge type failure. Slope stability calculations were made using the sliding wedge method for critical sections at stations 36+50 and 38+50. The critical cross sections along with the failure surfaces are on the cross section sheet included in this report.

Initially, a shearing resistance angle (Ø) of 22^{o} was assumed from triaxial test results. This yielded a factor of safety of 2.2 for the section at station 38+50. Since the slope is unstable, this factor of safety is obviously in error which indicates the peak strength from triaxial tests is not applicable in this case.

After a surface of sliding forms and extensive movement occurs the bonds between soil particles are destroyed and the particles along the surface of sliding assume a parallel orientation, favorable to low resistance to shear along the surface. The shearing resistance after very large displacement is known as the residual strength. It cannot be determined from conventional triaxial tests. Residual strength can be determined from direct shear tests or, in cases such as this where the failure surface is known, by varying the strength in stability analysis until a factor of safety of 1.0 is obtained by trial and error. In this case a Ø angle of 11^{o} yielded a factor of safety of 1.0.

A shear key consisting of granular material with the bottom of the key benched into rock was proposed as a method of correction

Various locations, configurations, depths, and widths of the shear key were analyzed on the critical sections. A cohesion of zero was used in all cases due to the long term type of analysis. For the existing fill and original ground a shearing resistance angle (Ø) of 11° and a unit weight of 130 pcf was used. A Ø of 20° and a unit weight of 130 pcf was used for that portion of the correction to be of compacted soil. For the granular material, a Ø of 40 degrees and a unit weight of 140 pcf was assumed.

The failure surface on the critical sections was determined from slope inclinometer data. The subsurface exploration indicated the minimum depth to rock was at approximately 180 feet left of centerline. After several analysds, the best shear key configuration was found to be the one shown on the cross sections. The bottom of the key extends from 176 feet to 196 feet left of centerline with side slopes of 1:1.

The critical failure surface found for both shear key sections analyzed is shown on the critical sections. The minimum factor of safety obtained was 1.4 and 1.6 for stations 36+50 and 38+50 respectively, assuming the natural ground down slope from the key will fail thus adding no stability to the embankment. This may be conservative since the shear key will tend to lower the water table and add stability to the lower portion of the landslide.

The slope to the left of the shear key was analyzed along the original failure surface which yielded safety factors of 1.2 and 1.3 for stations 36+50 and 38+50 respectively.

In addition to the shear key method, four other possible correction methods were considered as follows:

1. The complete removal of the affected portion of relocated KY 9 embankment was first considered. Benches would be constructed in rock and the fill replaced on a flatter slope with a drainage blanket. The estimated cost of this method was so great that the idea was abandoned.

2. Another means of correction examined was the use of railroad rails or H-piles as a type of retaining wall. The rails would be placed in pre-augered holes and concreted in rock to a depth of 10 feet. The estimated cost was considerably greater than the shear key. Due to the greater cost and the lack of reliable information concerning the design and performance of rails, the method was discarded.

3. A retaining structure consisting of reinforced earth was considered. However, this would involve either excavating a portion of the embankment or placing the wall at the location of the proposed shear key where it would also function as a shear key. Thus in this case a reinforced earth structure would be inefficient.

4. A crib type retaining structure was also examined. For this situation a cribwall varying in height from 26 feet to 46 feet would be required.

Detailed cost estimates were not prepared for any of the four alternates. However, approximate calculations indicated that the cost of any one significantly exceeded the cost of the shear key.

The shear key was discussed and the four alternate corrective methods were examined in a meeting held with the Division of Construction. It was the opinion of those present that the recommende shear key type correction would be the most effective and economica

Since the side slopes of the shear key were to be on a 1:1 slope and the bottom of the key would be below the existing failure plane, certain special construction techniques were stipulated in the recommendations.

The shear key was designed to be 600 feet long and it was recommended that it be constructed in three 200 feet sections.

This was to prevent the whole trench from being excavated at one time, thus trying to prevent complete failure of the embankment above. In all three stages the excavation and backfilling of the trench should proceed in a continuous operation. This meant that one section should be completed before opening up another section. Lateral drains should be constructed in a manner as to allow drainage for the open excavation at all times. It was highly recommended by Soil Section personnel that continuous operation should mean twenty-four hours a day, seven days a week. However, due to the high cost of labor in the area, the risk of failure by leaving the trench open over night or over the weekend would have to be taken.

The contract was awarded to R. C. Durr and Company for approximately $476,000 and construction began October 7, 1974. Construction of the shear key proceeded as expected until the existing failure surface was encountered. When the bottom of the trench reached the failure surface the 1:1 backslope on the up hill side of the trench started moving very rapidly and falling into the trench. The trench could be kept open for a short period of time, however, when left open over night the slope above the trench would, in most instances, move into and completely fill the trench. Finally the backslope throughout the shear key had to be flattened to a stable slope.

CONCLUSION:

The original amount of excavation calculated for this job was approximately 58,000 cubic yards. The final amount after having to flatten the backslope of the shear key was approximately $95,000. The final cost of correcting the slide was approximately

$663,000. As can be reflected in the original cost and yardage versus the final, there were numerous problems encountered in constructing the shear key. However, if the excavation and backfilling of the shear key would have proceeded in a twenty-four hour, seven-day a week operation, this excessive amount of overrun could have been prevented.

The project was completed August 8, 1975 and the shear key has stabilized the original landslide. The lateral drains from the bottom of the shear key have water flowing through them all the time and other than the excessive amount of overrun and additional cost, the shear key is a success.

PROGRESSIVE FAILURE IN SHALE
(CANNELTON DAM STAGE I COFFERDAM FAILURE)

By Claude A. Fetzer[1]

Cannelton Locks and Dam, a U. S. Army Corps of Engineers' project, are located on the Ohio River at river mile 720.7, about 3 miles upstream of Cannelton, Indiana. See Exhibit 1. The locks are located on the Indiana shore and consist of one main lock chamber on the river side 110 feet wide by 1200 feet long and one auxiliary lock chamber on the land side, 110 feet wide by 600 feet long; these were constructed during the period of 1962 to 1965. See Exhibit 2. The dam is non-navigable, and consists of two sections. The main portion is a concrete structure, 1412 feet long with twelve tainter gates. Each gate is 42 feet high and 100 feet long, and is supported between 15-foot wide piers. The second section of the dam is a 195-foot long concrete weir or walls that extends from the end of the gated section to the Kentucky shore. See Exhibit 3. The normal upper pool elevation after the dam was completed was to be 383.0 feet mean sea level, and the normal lower pool was to be elevation 358.0 feet making an operating differential head of 25 feet. During flood flows the dam gates are raised out of the water and the dam has no effect on flood stages. The maximum flood of record (1937) had a water surface of El. 408 at the site. The normal pool at the site in 1967 was elevation 358.

The plans for the dam required it to be constructed in three stages starting from the Kentucky shore and proceeding towards the completed locks on the Indiana shore. The plans required that the pier bases be constructed as caissons sunk from the existing river bed level of about elevation 326. The caissons were either single caissons having a length of 115 feet and a width of 50 feet, or double caissons at the cofferdam tie-ins having a length of 120 feet and a width of 140 feet. See Exhibit 4. The wells in the caissons were approximately 20-foot square. It was expected that the caissons could be sunk to top of the Waltersburg shale. After the caissons reached top of the Waltersburg Shale, the excavation through the shale to the Vienna limestone was required to be made in a checkerboard fashion through the wells with each excavation backfilled with concrete prior to excavating the adjacent well. The caissons were required to be protected by a cofferdam constructed to El. 395.

The contract for construction of the dam was awarded to J. A. Jones Co. on 30 June 1965. The specifications permitted the contractor to submit alternative construction schemes. The contractor proposed changing the construction scheme to an open excavation extending to top of rock. His proposal, which was later formalized under a contract modification, was to construct a two-stage cofferdam starting from the Kentucky shore with the first stage. The first stage was designed to enclose the fixed weir and piers 7 through 13 of the main dam. The second stage was designed to tie into pier 7 and continue on to the locks. See Exhibit 5.

This paper is only concerned with the first-stage cofferdam. The first-stage cofferdam consisted of an outer ring of 37 sand-filled steel sheet-pile cells 60.48 feet in diameter which was backed up by a sand berm. The sand berm was retained by an inner ring of 28 tub cells. The sheets on the outer cells extended from El. 395 to top of rock (El. 247-260) except that the inboard sheets

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[1] Chief, Foundations and Materials Branch, Corps of Engineers, Ohio River Division, Cincinnati, Ohio

on the upstream and downstream arms were only driven a few feet below the top of the sand berm at El. 326. The sheets on the tub cells extended from El. 326 to top of rock. The tub cells were theoretically located from 3.71 to 10.91 feet away from the pier edges. Excavations for single piers (120 feet by 50 feet) and for the double pier (120 feet by 140 feet) were to be made by open cut. The seepage was controlled by 23 deep-well pumps to rock located along the Kentucky shore and just inboard of the outer row of cells.

Construction of the cofferdam was started in 1966 and it was substantially completed by July 1967. See Exhibit 6. Excavation and dewatering of the overburden within the tub area from El. 326 to top of rock (El. 260 ±) at the riverward end was completed during July through September 1967. Cleanup to top of rock was completed in the pier 7-pier 8 area by Oct 67, when drilling for presplit holes through the Waltersburg shale around pier 8 was started. See Exhibit 7. The first half of pier 8 perimeter was shot on 12 Oct and the second half on 14 October. The arc between cells 139 and 167 at the downstream Indiana corner was pulled and redriven during this period, and repair work on several cells continued. Drilling for the primary blast holes for pier 8 was started on 15 October The center area of pier 8 was shot on 16 October and the ends were shot on 17 October. Excavation of pier 8 was started on 17 October and by 18 October was down to within 2 or 3 feet of grade at the downstream end; the shale in the downstream Indiana corner was breaking up and sloughing into the excavation, and much water was leaking out of the shale at the downstream end of pier 8 from below the tub cells. See Exhibit 8. On 19 October the Kentucky side of pier 7 perimeter was pre-split. Spalling of the upper 4 feet of shale around pier 8 excavation continued. On 20 October the remainder of pier 7 perimeter was pre-split except for 7 holes in the downstream Indiana corner. Sloughing in the downstream end of pier 8 continued until it reached the toe of cell 166 on 21 October. See Exhibit 9 Cracks were observed in the gatebay 7 area (the unexcavated area between piers 7 and 8) and at toe of cell 143 at 1600 hours on 21 October. At 1920 hours on 21 October the last 7 pre-split holes were shot in pier 7, and at 2000 hours the shale in gatebay 7 moved laterally 2 or 3 feet into pier 8 excavation; movement was primarily along the previously observed cracks. See Exhibits 10 and 11. The crack at the toe of cell 143 had extended to arc 143-144 and had opened up to a width of 2 to 4 inches.

On 22 October, pier 8 excavation was flooded to stop caving of sides. On 24 October the crack at toe of cell 143 widened and had a depth of 6 to 8 feet. During the period of 25 thru 27 October, the contractor installed 20-foot diameter sand-filled, sheet-pile cells in both ends of pier 8, and on 27 October started sticking and driving sheet piling along the pier 7 perimeter pre-split line. Spalling was visible along the Kentucky wall of pier 8. See Exhibits 12 and 13.

On 29 and 30 October the water was pumped out of pier 8. See Exhibit 14. Sloughing continued along the Kentucky side of pier 8 and blocks were falling out of the Indiana side. On 31 October a large block of shale fell out of the Indiana side and covered an end loader. Contractor decided to slope Indiana wall to stop sloughing of shale blocks. On 31 October the Indiana wall was sloped and sheet pile driving around pier 7 was continued. Spalling continued along Kentucky wall of pier 8 excavation. See Exhibits 15 and 16. At 1500 hours the crack at the toe of cofferdam had extended to cell 144 and to arc 142-143. The inspector ordered the stairwell moved from cell 143 to cell 142. See Exhibit 17.

At 0430 hours on 1 November, cell 143 failed suddenly by ripping along the Kentucky side T connection to arc 143-144. The sand in the cell spilled into the pier 8 excavation. See Exhibit 18. Cells 144 through 149 moved laterally in the downstream direction; the maximum movement from the theoretical original location was 11 feet (for cell 145). In addition there was a 50-foot wide subsidences with a maximum depth of 12 - 15 feet in the berm fill behind cells 144 to 149. There was also an upheaved zone extending diagonally across the foundation from cell 149 of the upstream arm to cell 163 of the downstream arm. See Exhibit 19. Fortunately, the graveyard work force was on their meal break, and no one was in the tub area when the failure occurred. This failure was described in the Engineering News Record (1).

After the failure, flooding of the tub area was started almost immediately. On 3 November the contractor started backfilling the tub area with river sand to El. 285. See Exhibit 20. Internally braced sub-cofferdams were constructed for each pier base through the sand backfill. Z-piling (PZ 27) was installed around the edge of each pier base; it was driven through the sand backfill and through the shale to limestone. Heavy internal bracing was installed as the material was excavated from within the sub-cofferdam. The pier bases were completed in this manner without any major problems. See Exhibit 21.

Investigations after the failure indicated that the failure occurred near the shale-limestone contact with the heave area between Cells 149 and 163 representing the end of the rupture area. See Exhibit 22. The failure could be classed as a simple block and wedge failure; however, a review of the events from 1 October to 1 November indicates that failure started when the excavation for pier 8 was started on 18 October. Excavation of the original overburden within the sub-coffer area with the subsequent horizontal loading of the inner cells caused a reversal of stresses in the shale. According to Bjerrum (2), stress reversal in stiff clay and clay shale could possibly reduce its strength to residual strength.

The effects of stress reversal in clay shale and for the Waltersburg's shale in this case are not known. In its pre-construction stage the Waltersburg shale had a vertical load of 66 feet of saturated sand and gravel with very little known horizontal stress; this disregards unknown stresses due to tectonic or regional geologic stresses. Immediately after the sand and gravel was excavated along with removal of water to top of rock, the apparent stresses in the shale in the tub area, i.e., between the inner cells, changed from a high vertical stress with low horizontal stress to a very low vertical stress with a high horizontal stress (due to the cofferdam loading). The exact apparent vertical stress can not be computed as there were no piezometers in the shale to measure the uplift. Assuming full pressure relief during drawdown, the computed changes in stresses are as follows:

Before excavation Σ_V = 4,950 psf at top 6,690 psf at bottom
Σ_H = ?
After excavation Σ_V = 0 at top, 1,740 psf at bottom
Σ_H = 6,400 psf ave. + ?

In this particular case the shale in the pier 7-8 area was subject to horizontal stresses from the upstream and downstream arms and to an additional horizontal stress from the river arm. By cutting a hole in the shale for the pier 8 the shale was no longer a contiguous body. Instead, the excavation acted like a cutout in a concrete wall where stresses are concentrated. See Exhibit 23. The remaining shale at the ends of the pier 8 excavation were not only subjected to high horizontal loads due to the adjacent cells; the continuous sloughing at the downstream end and the movement of the shale away from cell 143, i.e., the continuous widening of the crack at the toe of cell 143) were undoubtedly partly due to a bearing capacity failure. In fact the upstream row of cells was acting as a beam to hold the shale at cell 143 from moving into the excavation; the ripping of the cell on 1 November relieved the high tension that had built up in this cell. The crumbling of the Indiana and Kentucky side walls was indicative of the high horizontal stresses in the shale. The shale block between the pier 8 excavation and the river arm of the cofferdam are computed on Exhibits 24 and 25. The exact direction of the resultant force is not known. The upstream and downstream arms were tending to buckle the shale block in compression with a combined force 43,400 kips, whereas the river arm was trying to push the shale block into the pier 8 excavation with a computed force of 20,900 kips. The tendency for the block to fail into the excavation was aggravated by the pre-splitting around pier 7. The final movement of the block was triggered by the shooting of the last 7 pre-split holes at the downstream corner on 21 October; however, the stress cracks were noticed prior to this final shot. The buoyant weight of the block in gatebay 7 involved in the movement was computed to be 21,620 kips; if the shale at the base had a strength of Ø = 13 Ø ; c = o, a force of 5,000 kips would have been required to move the block as a wedge, which is a much greater force than would have developed from the 7 pre-split holes. However, this type of analysis is not considered applicable to the failure of 21 October as the failure was more like a tilting failure of a vertical deck of cards with little if any movement at the base and considerable movement at the top. Even though the base probably did not move, the upper part of the rock was broken and had lost most of its strength. The continued high stress in the side walls was evident by the continuous sloughing of both side walls after pumpdown during the period of 29 October - 1 November. The continued widening of the cracks in front of Cell 143 indicated that the rock at the upstream end was moving into the excavation even though the 20-foot diameter sand-filled cells had been placed to hold this face of the excavation.

When the tension in Cell 143 exceeded the strength of the structural tee, the beam snapped and triggered the movement of Cells 144 - 149 on 1 November. Using extremely simplifying assumptions the back computed shear strength for the block moved by the slide indicates Ø = 13° and c = 0 at shale-limestone contact. See Exhibit 25. Samples of the slide plane could not be recovered. Tests conducted on intact Waltersburg Shale had strengths on 6-inch diameter direct-shear samples ranging

from	Ø = 16°	c = 0
to	Ø = 24	c = 4.3 tons per sq ft

Repeated shear tests conducted on presplit 3-inch by 3-inch by ½-inch direct shear samples had values ranging from $\phi = 6^\circ$ $c = 0$

to $\phi = 8^\circ$ $c = 0$

Conclusions

1. The excavation methods caused a reversal of stresses in the Waltersburg Shale, but the effects on the shear strength of the stress reversal are unknown.

2. The layout of the cofferdam resulted in high horizontal stresses in the Waltersburg Shale.

3. Progressive failure of the shale surrounding the pier 8 excavation occurred.

4. The upstream arm of the inner row of cofferdam cells restrained movement of the upstream end of pier 8 excavation until the strength of Cell 143 was exceeded. Splitting of Cell 143 triggered the movement of Cells 144-149.

5. To avoid progressive failure extreme caution is required when making cuts into clay shale. An overall assessment of the in-situ geotechnical conditions should be made, and the effects of the unloading and imposition of new stresses on the weaker members of the formation should be carefully analyzed. Where soft seams or presheared surfaces due to faulting are found, extremely low shear strengths approaching the residual strength should be used in the analyses. Unless 100 percent core recovery is achieved in the shale members, the presence of a soft or presheared seam should be assumed where the core is missing. The uplift and pore pressures in all members must be measured, and used in the analyses. During construction if signs of movement or distress are observed quick countermeasures are required if failure is to be averted.

APPENDIX. - REFERENCES

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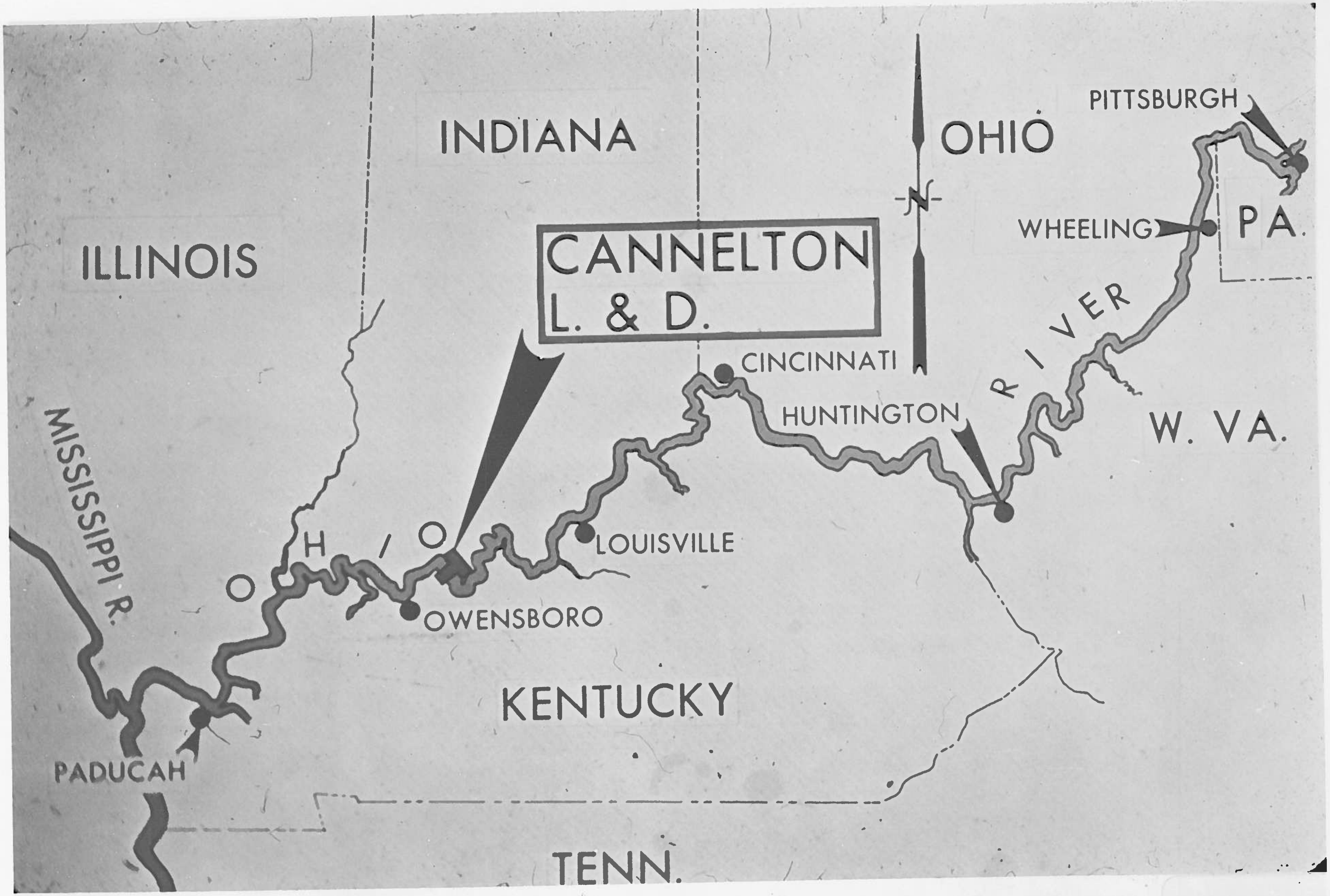
ACKNOWLEDGEMENTS.

The chronology was developed from project records by Cecil E. Dodson, who was the Corps of Engineers' resident geologist on the project starting 20 May 1968. He also collected the available slides and made drawings showing the sequence of events. The contract was administered by the Louisville District, Corps of Engineers. During the period of Sept-Nov 1967 COL R. R. Wessels was District Engineer, R. H. Russell was Chief, Construction Division, and A. K. Boyle was Resident Engineer. The photographs on Exhibits 20 and 21 were used by permission of Engineering News-Record. Some slides used in the lecture were furnished by Robert E. Grayman.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RIVER
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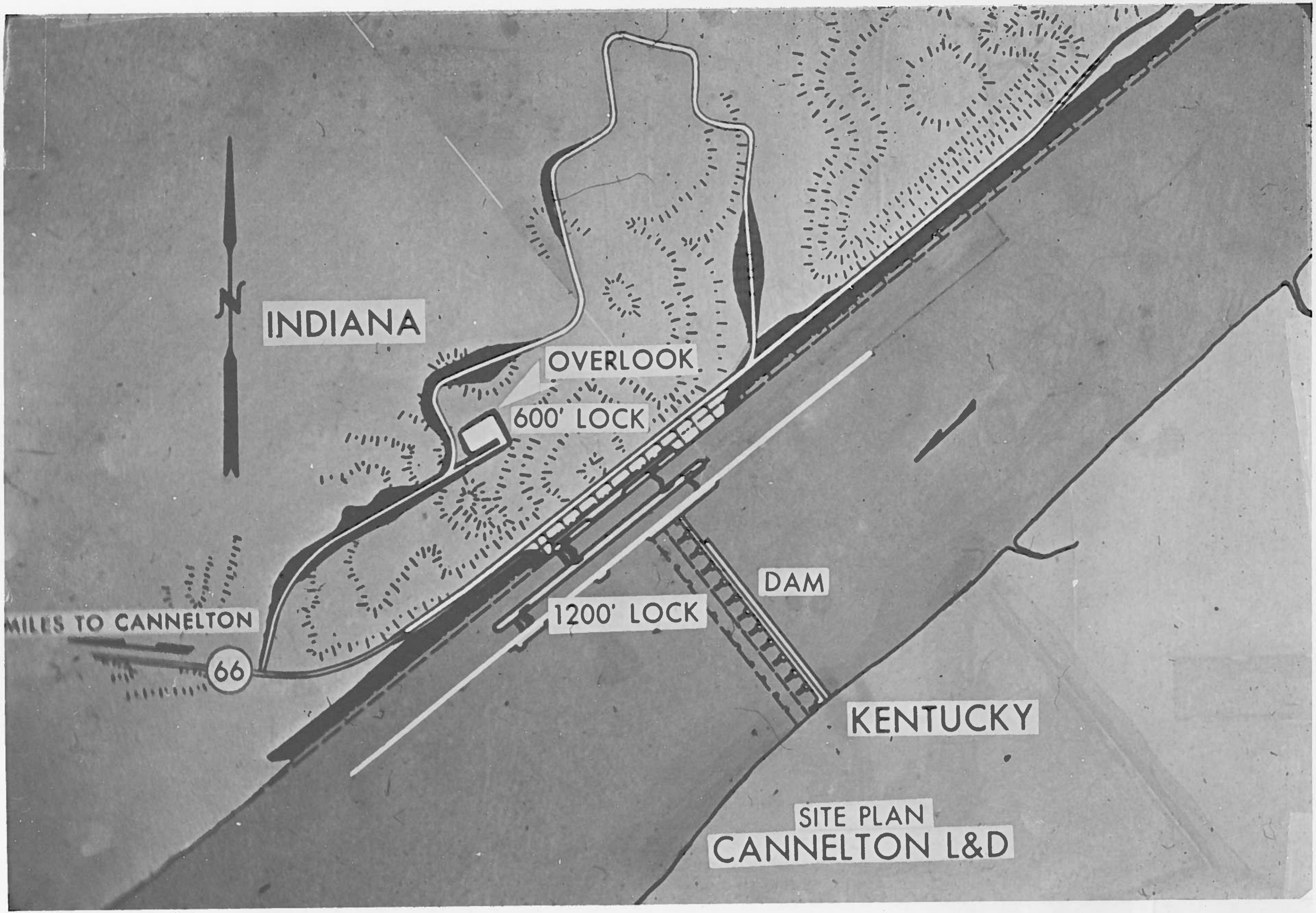
HUNTINGTON
W. VA.
MISSISSIPPI R.
O H I O
LOUISVILLE
OWENSBORO
KENTUCKY
PADUCAH
TENN.

EXHIBIT 1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66
DAM
1200' LOCK
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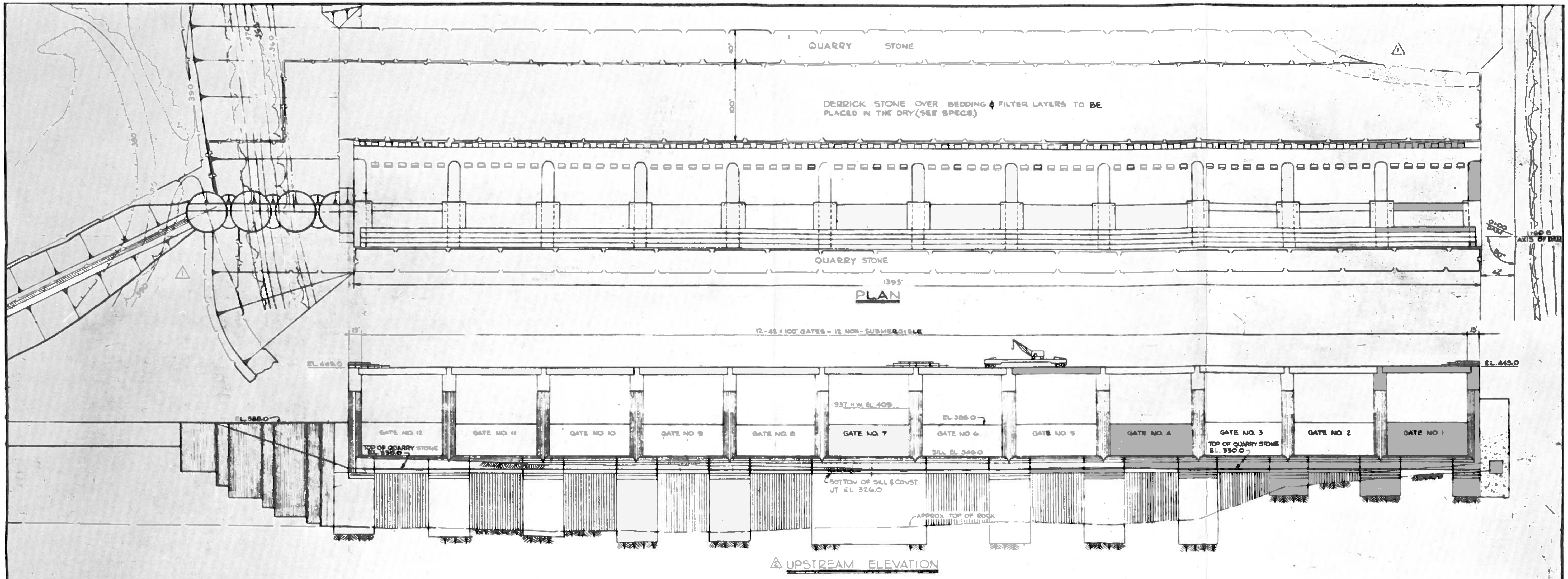
SITE PLAN
CANNELTON L&D

EXHIBIT 2



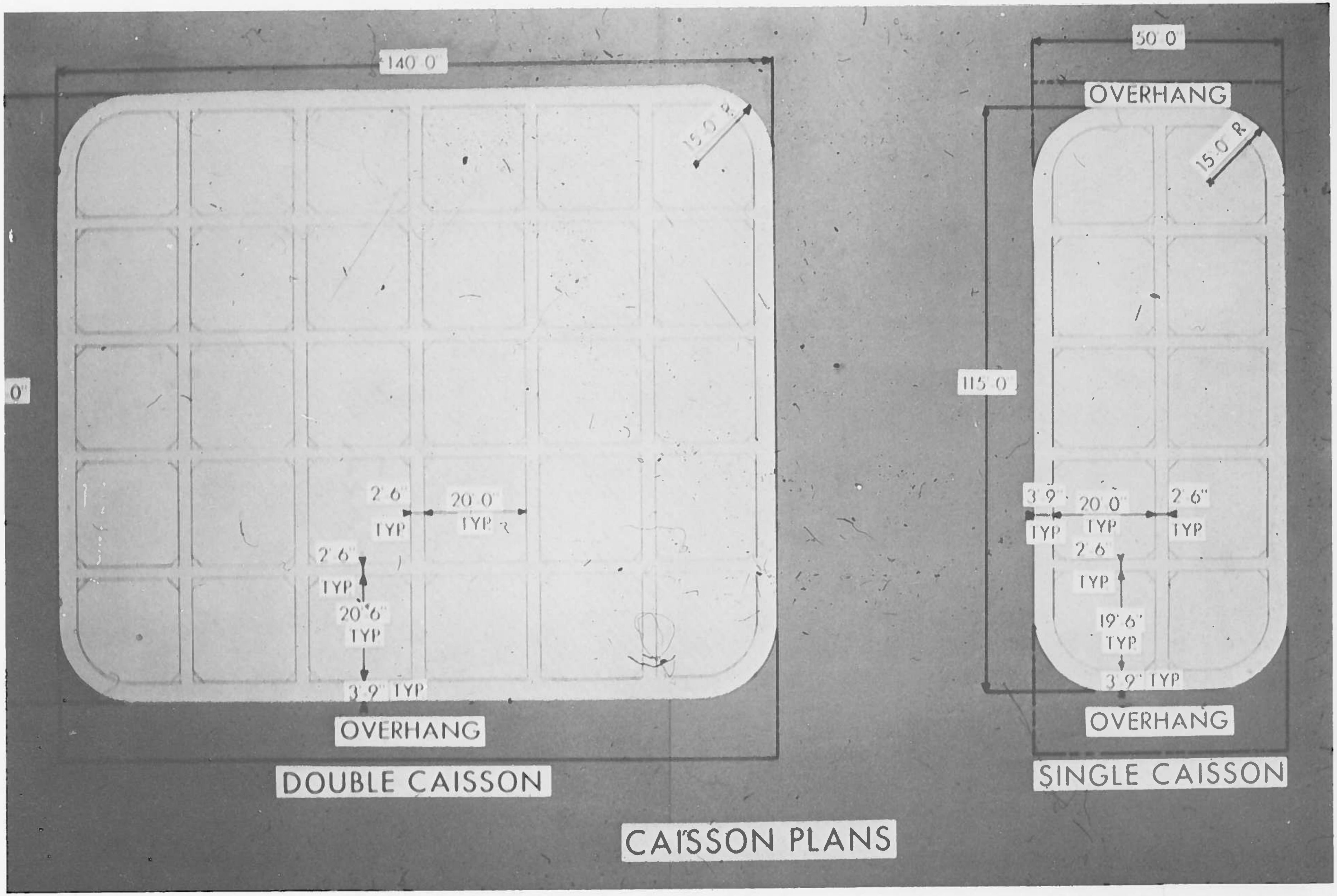
REVISION	DATE	DESCRIPTION	BY
2	12 JAN 66	REVISED FOOTINGS FOR PIERS 6, 7, 8, 11 & 12 (CONT MOD)	CHB
1	15 JULY 65	REVISED PER AMDT NO. 1 & 5	SFF

U. S. ARMY ENGINEER DISTRICT, LOUISVILLE
CORPS OF ENGINEERS
LOUISVILLE, KENTUCKY

OHIO RIVER
CANNELTON LOCKS AND DAM
DAM
PLAN AND PROFILE

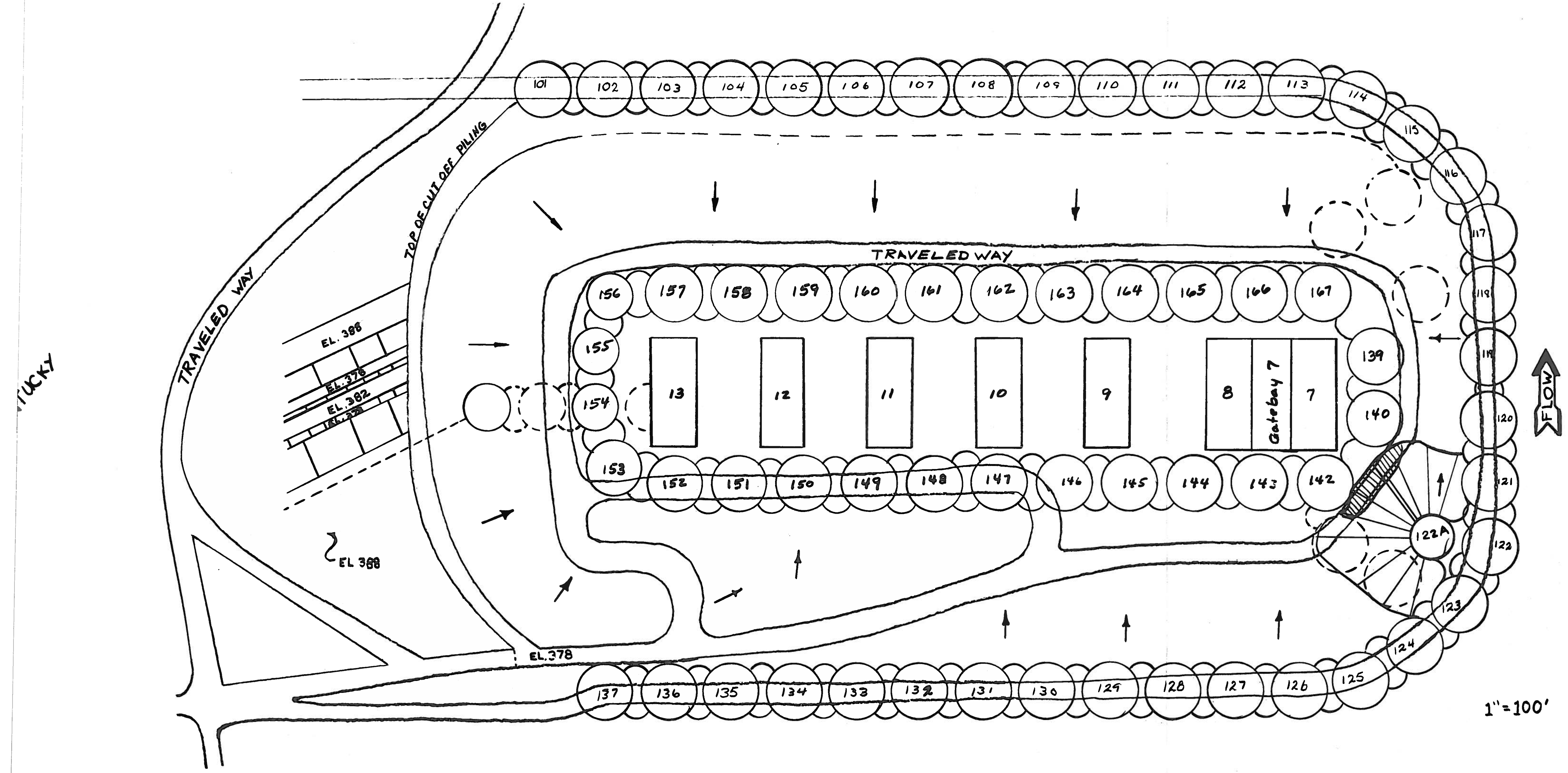
DESIGNED:
DRAWN: T.F.M.
TRACED: C.A.H.
CHECKED: R.E.L.
SUBMITTED:
APPROVAL RECM: R. E. Kasken
ASST. CHIEF ENGINEERING DIV.
APPROVED:
COL. CORPS OF ENGINEERS
DISTRICT ENGINEER
DATE: APRIL [illegible]
APPROVED:
CHIEF, ENGINEERING DIVISION
SCALE: 1" = 50'

EXHIBIT 3



CAISSON PLANS

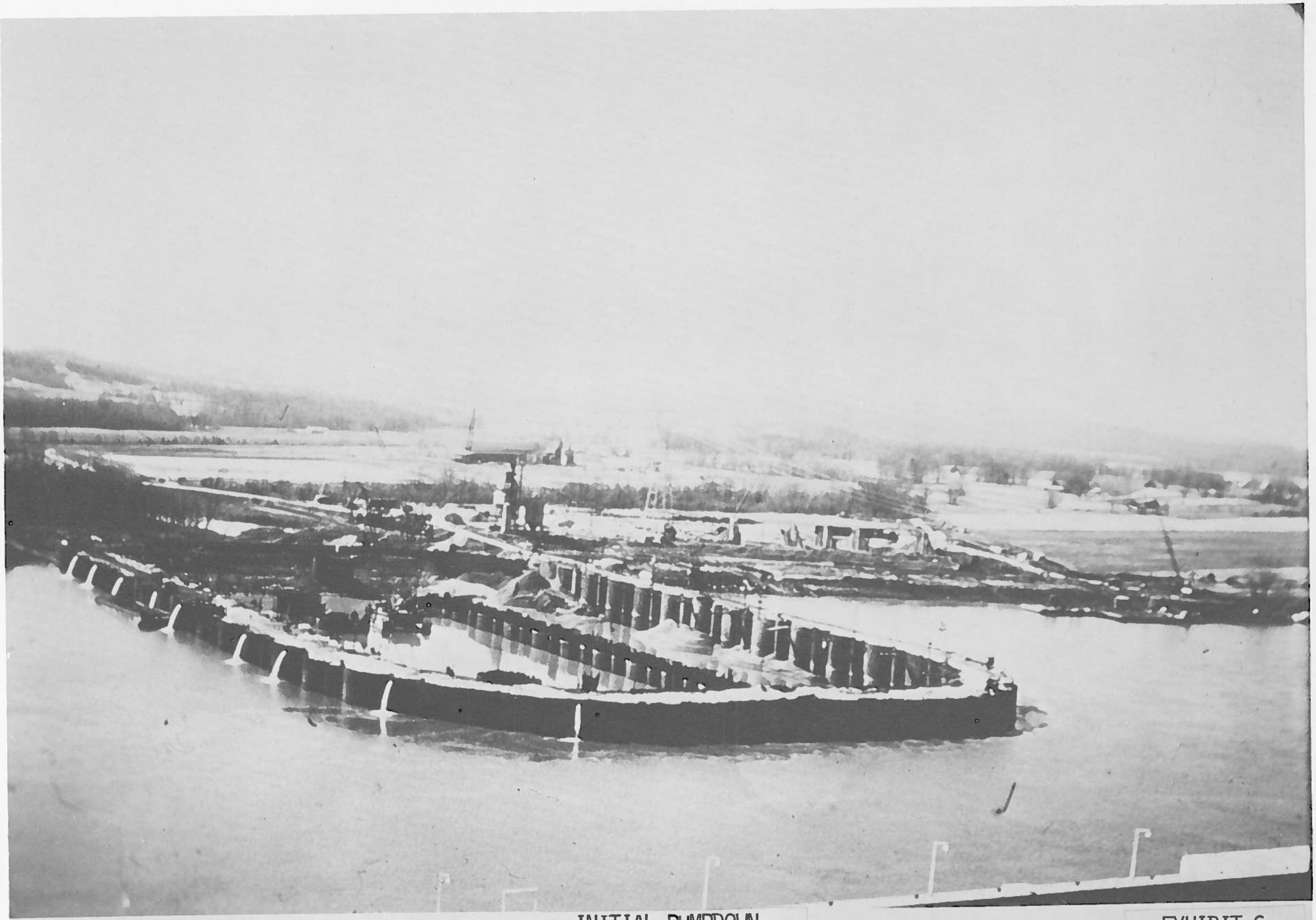
EXHIBIT 4



STAGE 1 COFFERDAM LAYOUT

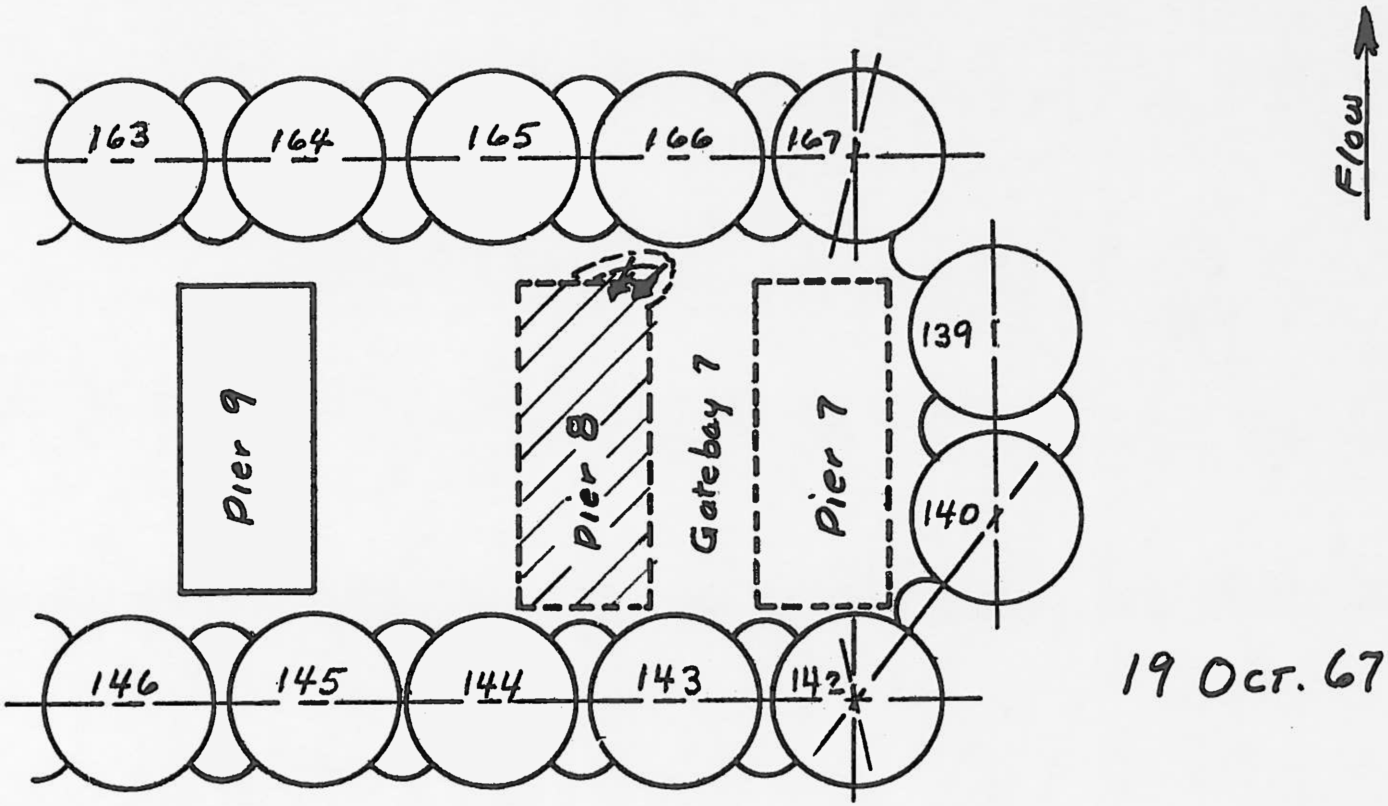
FOR SECTIONS SEE EXHIBITS 22 & 24

EXHIBIT 5



INITIAL PUMPDOWN

EXHIBIT 6

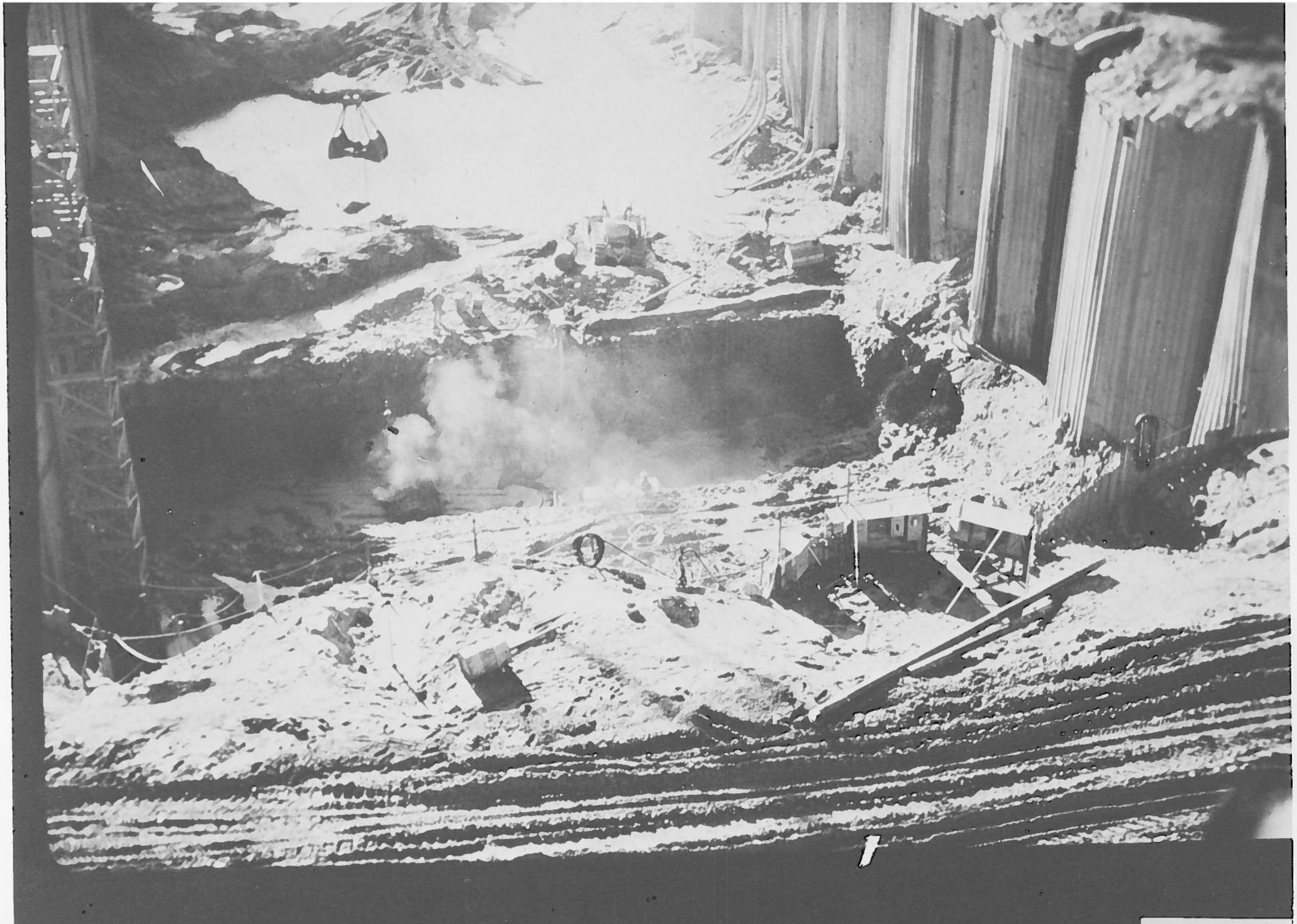


19 Oct. 67

Sloughing in downstream - Indiana corner continues nearing Cell 166. Water entering cracks in shale at toe of Cell 166. Pre-split Kentucky side of Pier 7; continued pre-split drilling. Spalling from top 4 ft. of shale around Pier 8 excavation. Water moving thru shale 4 ft. below surface."

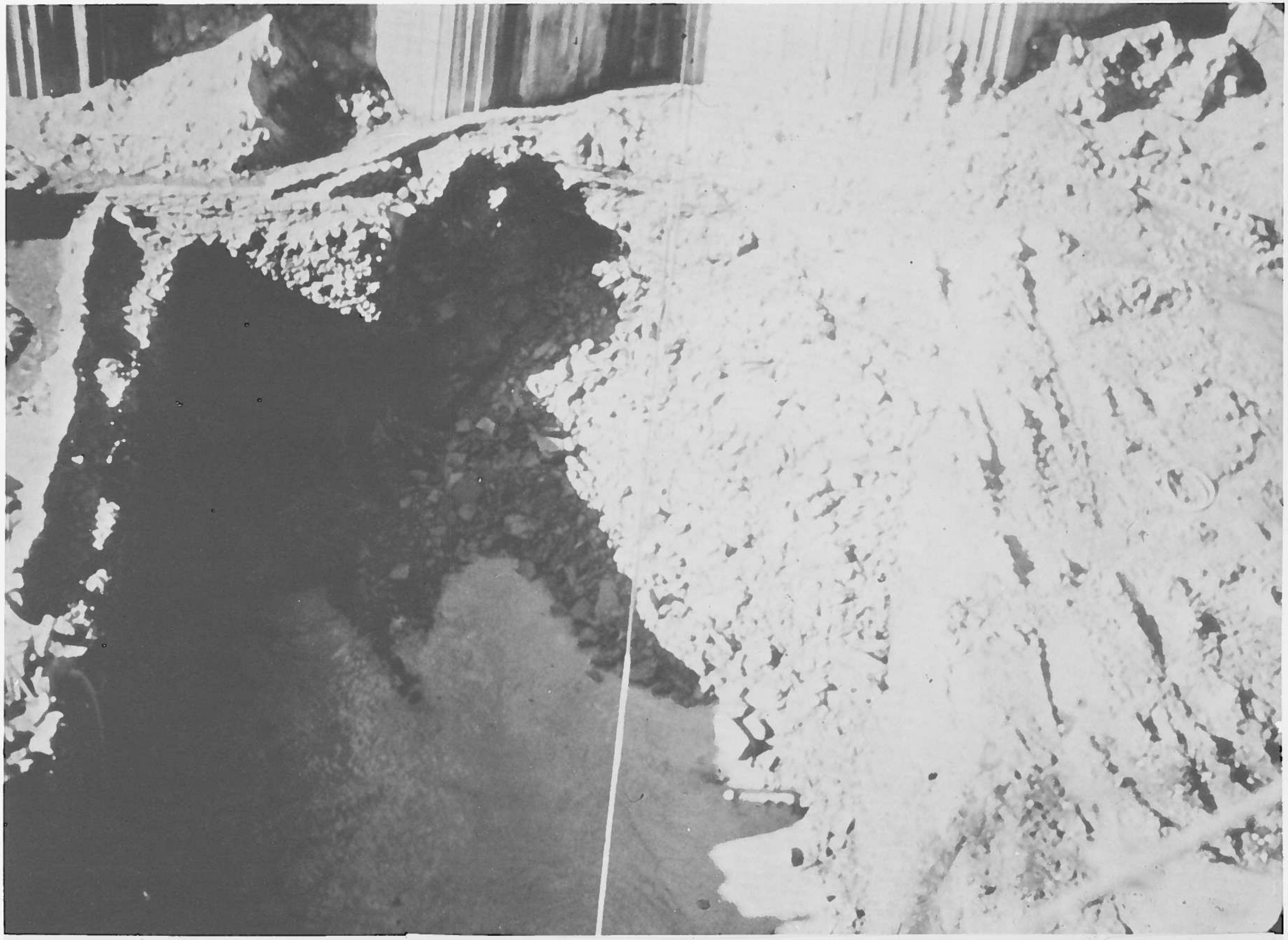
After C.E. Dodson

EXHIBIT 7



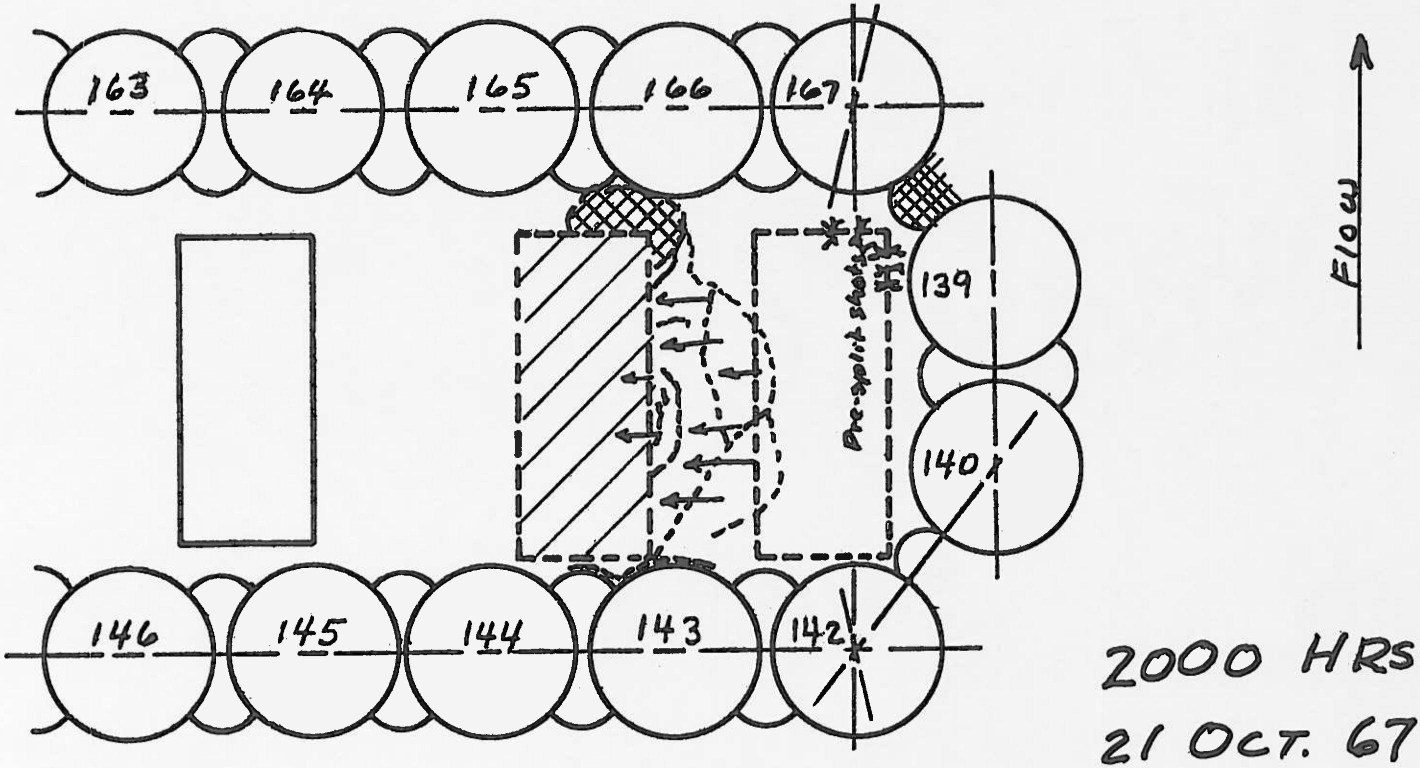
EXCAVATION PIER 8   19 OCT 67

EXHIBIT 8



SLOUGHING IN DOWNSTREAM INDIANA CORNER
21 OCT 67

EXHIBIT 9



2000 HRS.
21 OCT. 67

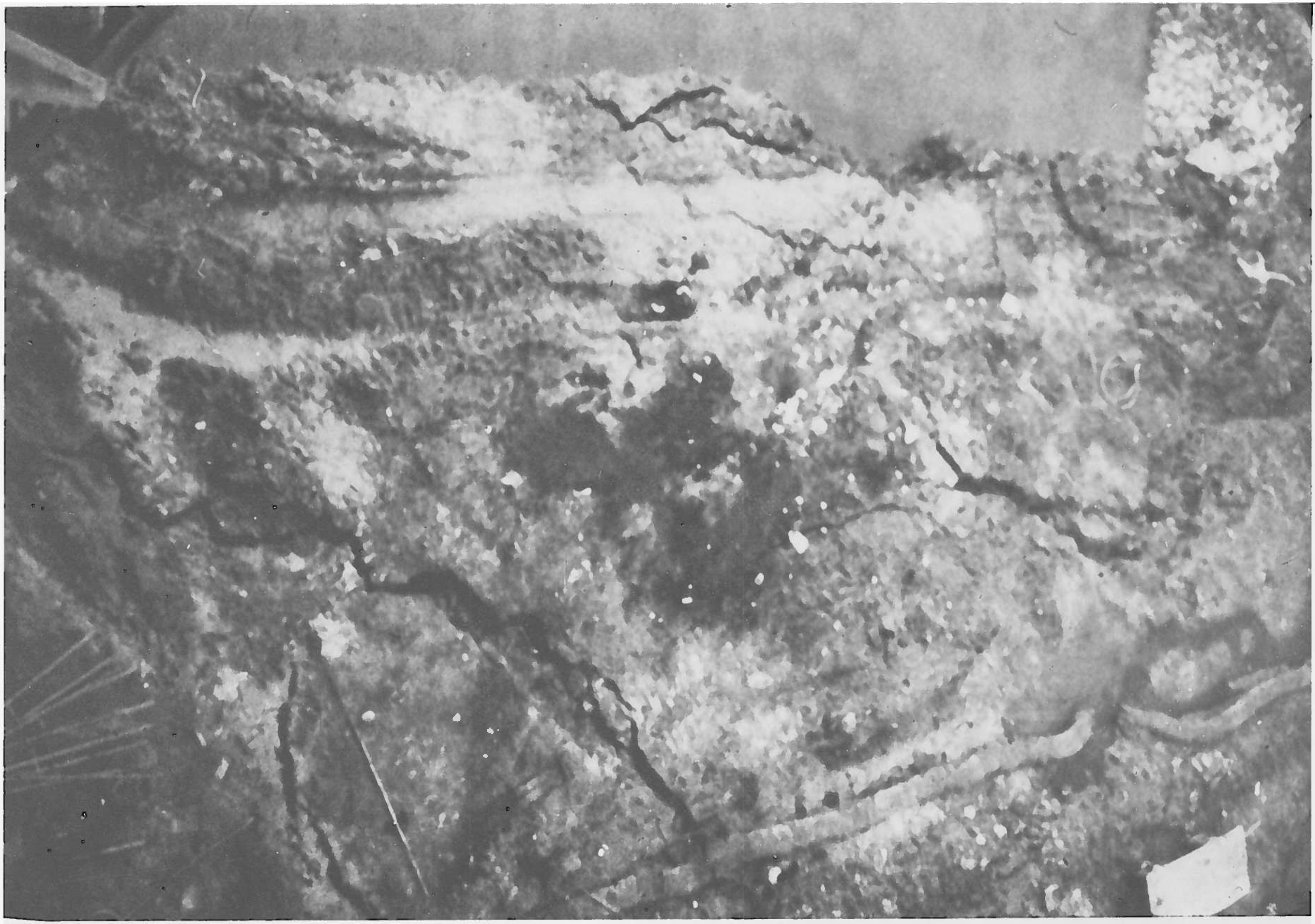
1920 Hours: Shot last 7 pre-split
holes in Pier 7 in downstream Indiana corner.

2000 Hours: Shale of Gatebay 7 moved into Pier 8
Excavation 2-3 ft. Movement occurred
along previous observed cracks.

Crack at toe of Cell 143 extended to arc 143-144
during afternoon, and had opened up to 2-4 in.

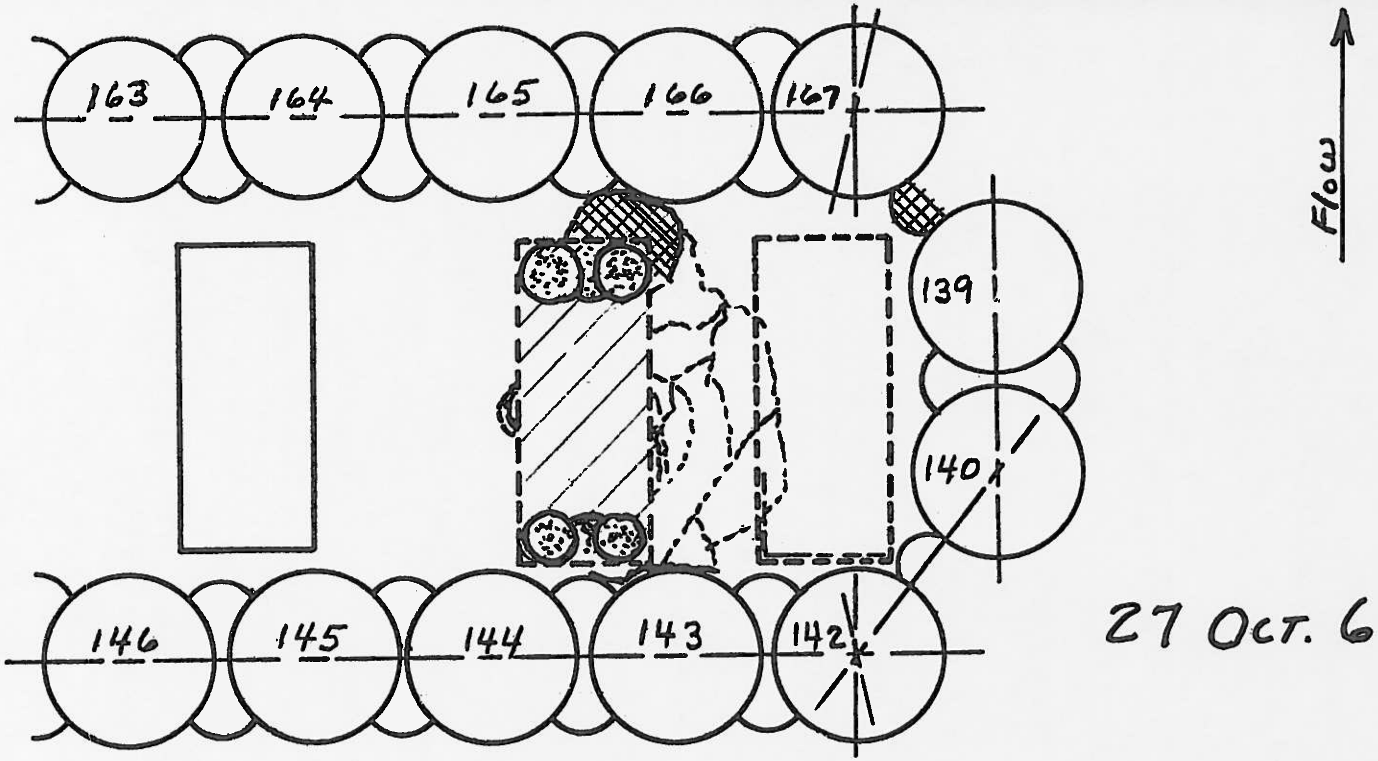
After C.E. Dodson

EXHIBIT 10



CRACKS IN GATEBAY 7 22 OCT 67

EXHIBIT 11



27 Oct. 67

22 Oct.: Pier 8 excavation flooded.

24 Oct.: Cracks at toe of Cell 143 widening.
Inspector noted that some sheets of Cell 143 driven into shale.

25-27 Oct.: Contractor constructed 20-ft. diameter cells in both ends of Pier 8 excavation.

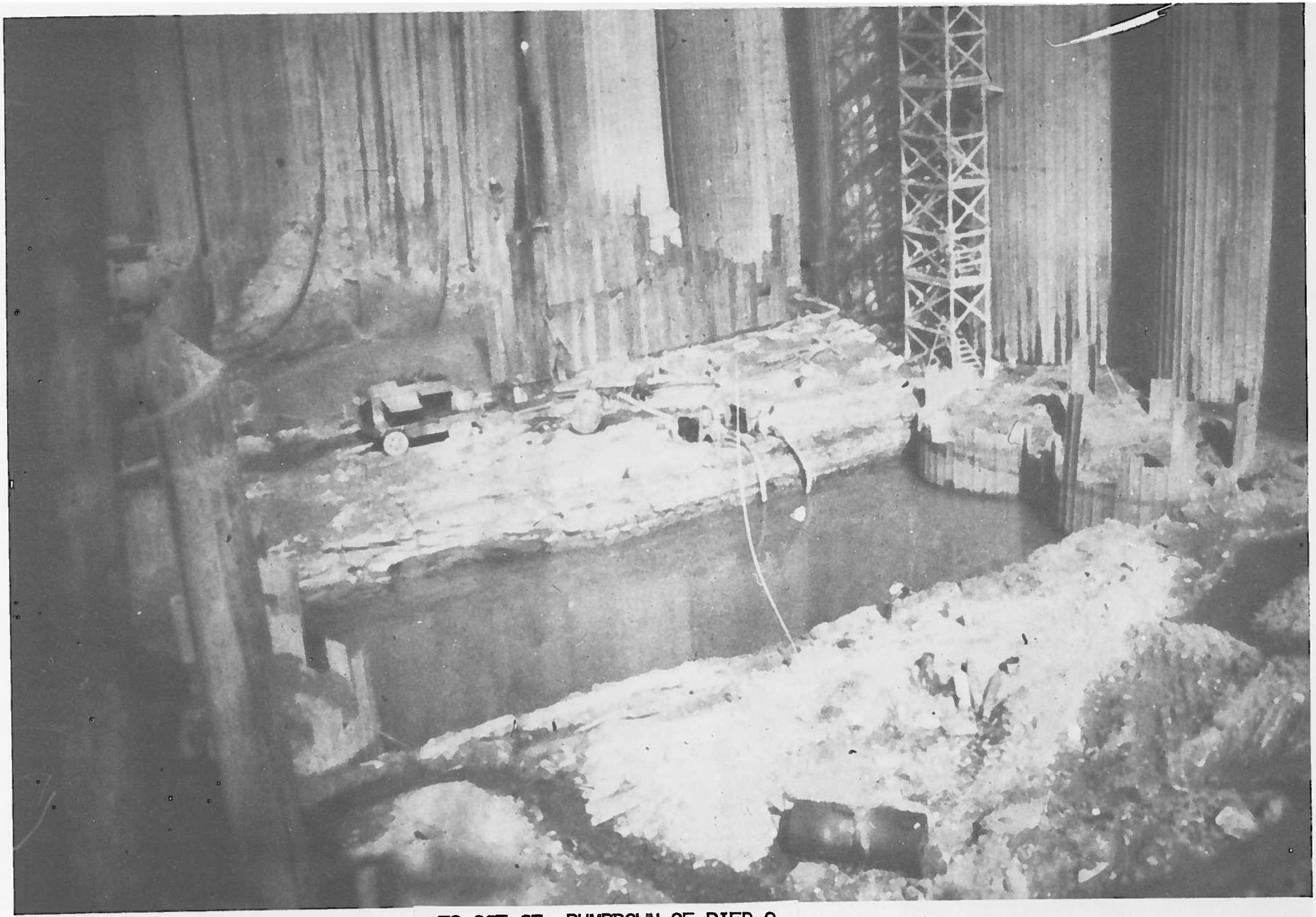
27 Oct.: Contractor started sticking and driving sheet piling in Pier 7 pre-split line. Spalling visible in center of Ky. wall of Pier 8.

After C.E. Dodson

EXHIBIT 12

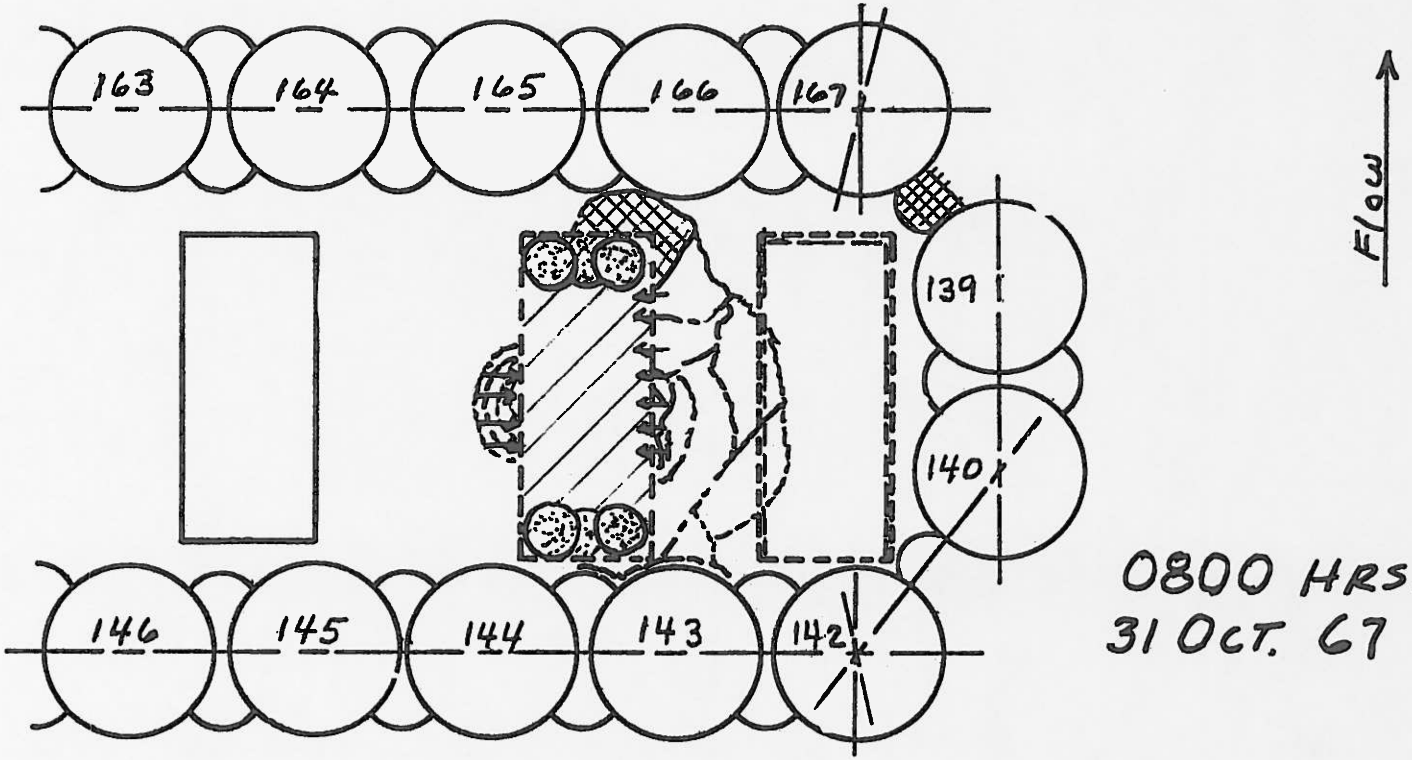
25 OCT 65 SETTING 20-FT DIAM. CELLS IN PIER 3 EXCAVATION

EXHIBIT 13



30 OCT 67 PUMPDOWN OF PIER 8
EXCAVATION

EXHIBIT 14



0800 HRS.
31 OCT. 67

27-31 Oct. : Contractor continued to stick & drive sheet piling in Pier 7 pre-split line.

30 Oct. : Sloughing and spalling continues in Ky. wall; curved configuration obvious. Pumped Pier 8 down & started cleanup. Blocks falling from Indiana wall.

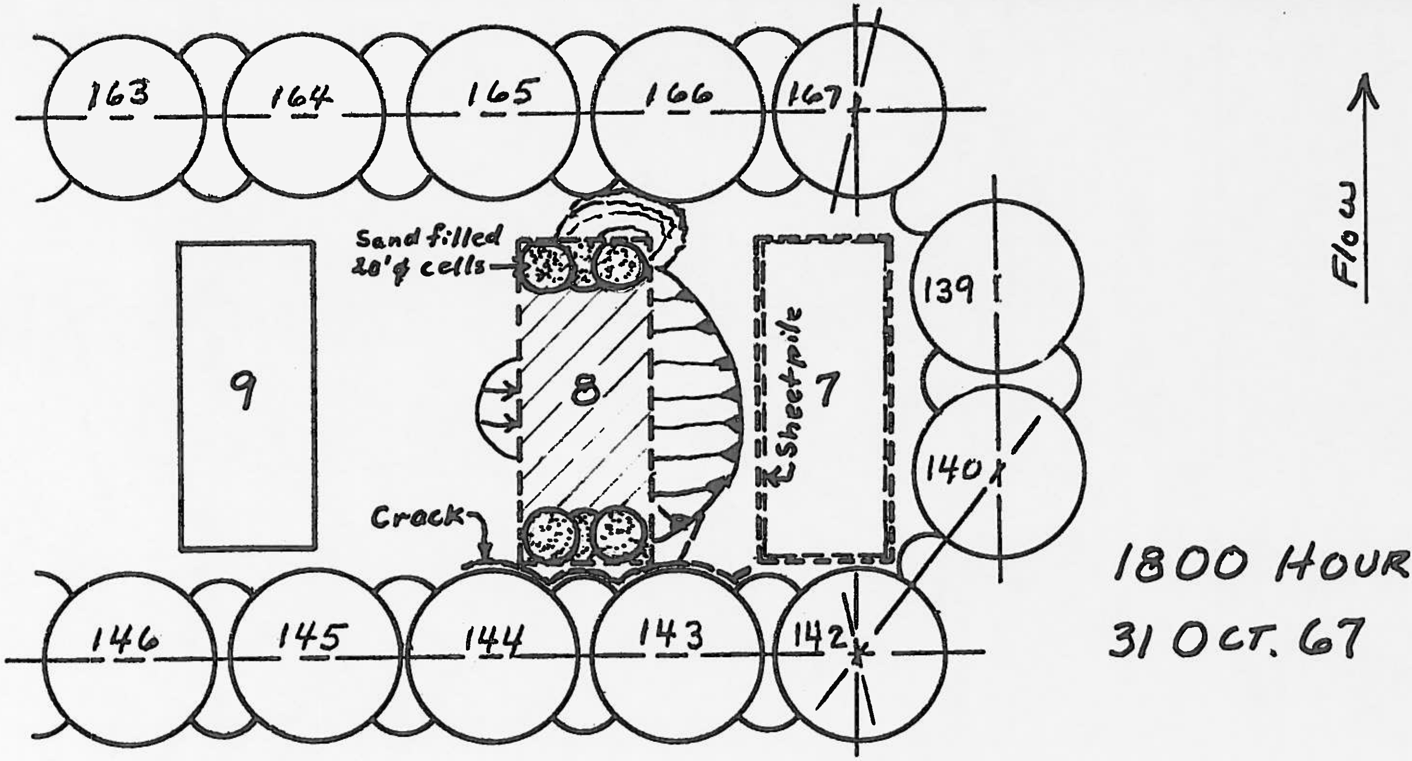
31 Oct. : Sloughing continues in Ky. wall. At 0800 Hrs., a block of shale from Ind. wall fell on end loader. Contractor decides to slope Ind. wall to stop sloughing of shale blocks.

After C. E. Dodson

EXHIBIT 15

31 OCT 67 FALLOUT IN BOTH SIDES OF PIER 8

EXHIBIT 16



1800 HOURS
31 OCT. 67

Shale in Gatebay 7 sloped to Pier 8 excavation.

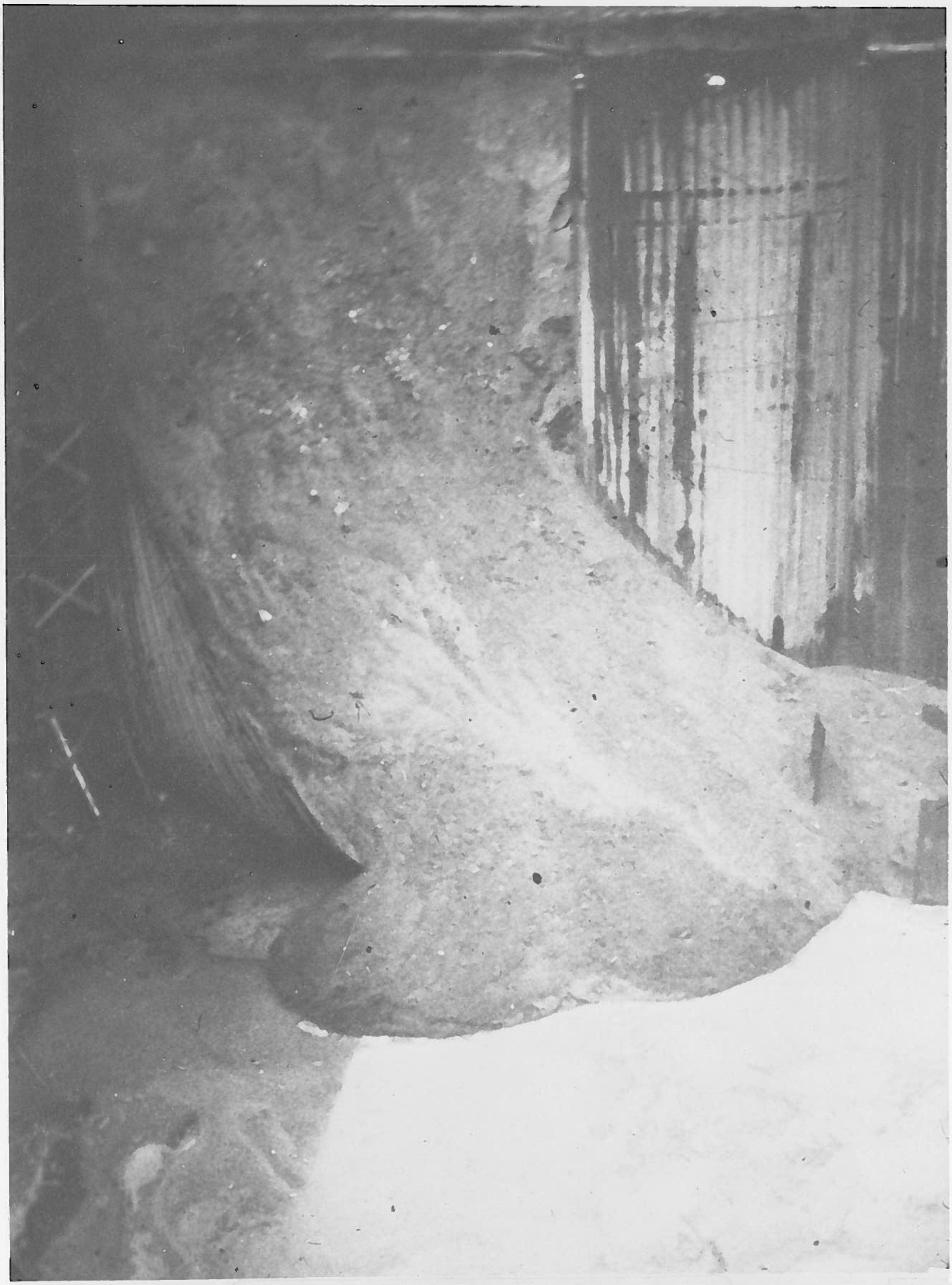
Contractor continues driving sheet pile into pier pre-split line; spalling continues in Kentucky wall.

At 1500 Hrs. the crack at the upstream toe had extended to cell 144 and to arc 142-143 and into Gatebay 7.

Inspector ordered stairwell moved from Cell 143.

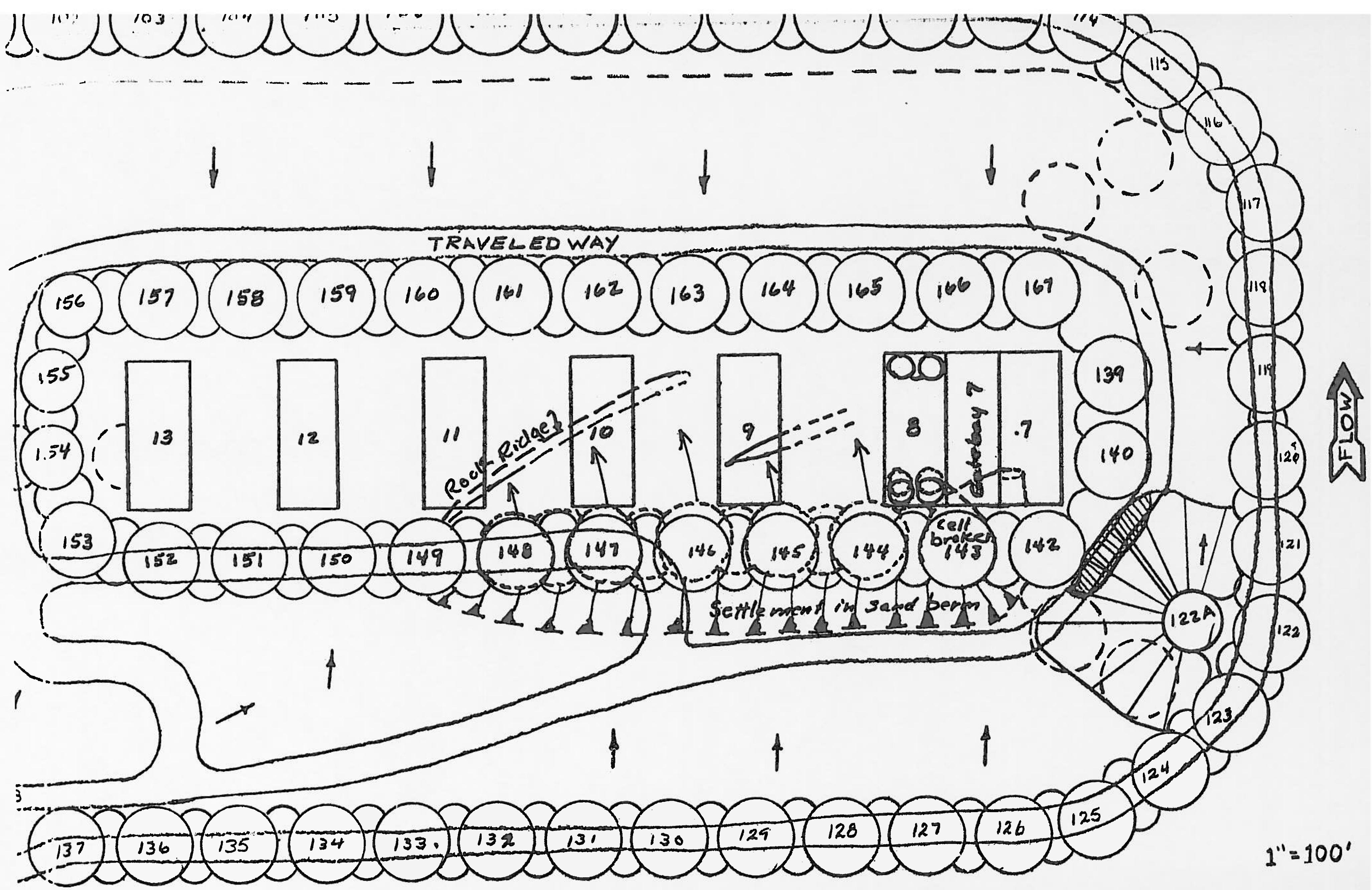
After C. E. Dodson

EXHIBIT 17



1 NOV 67 SAND SPILL FROM CELL 143

EXHIBIT 18



VISIBLE EXTENT OF FAILURE

EXHIBIT 19

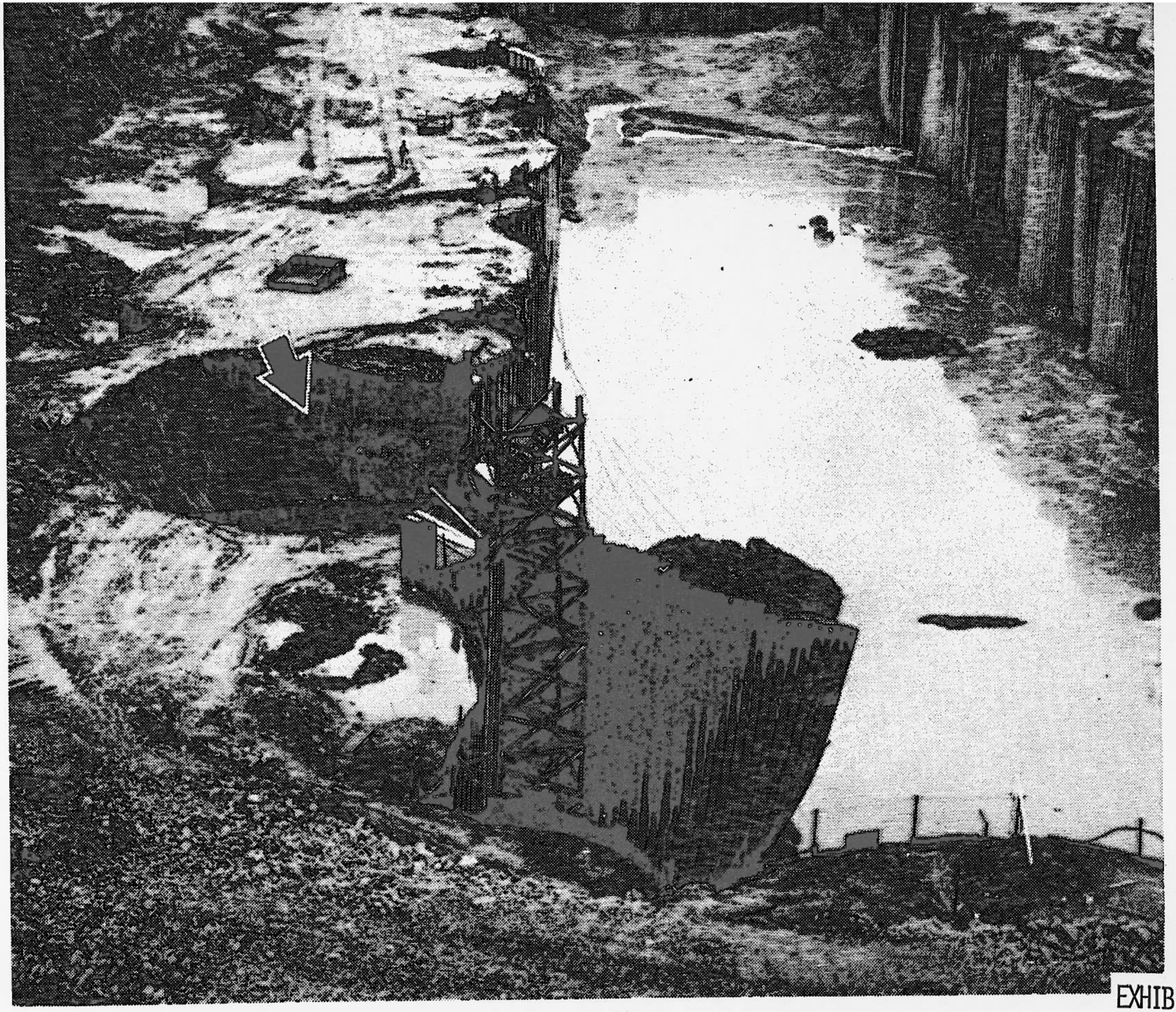
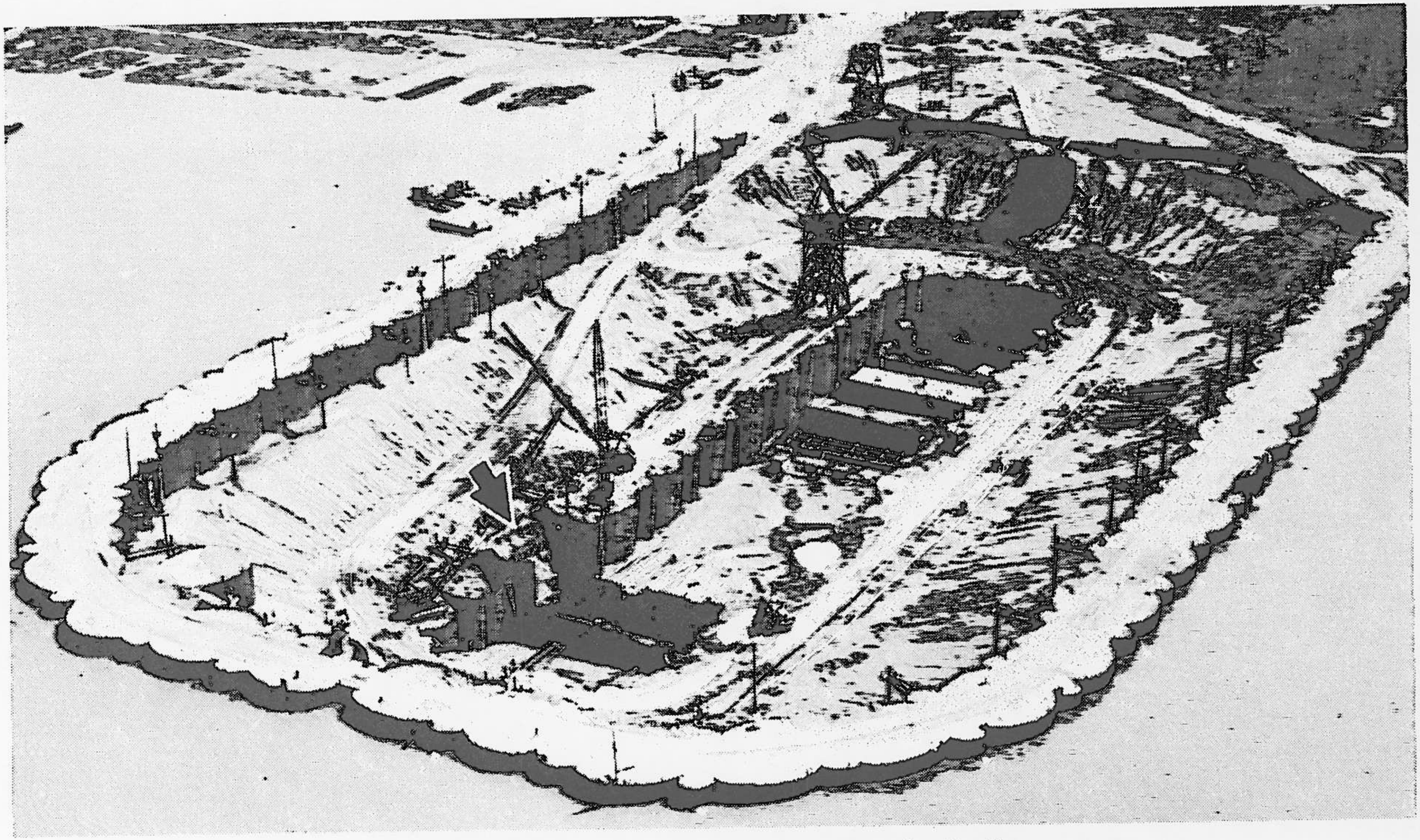


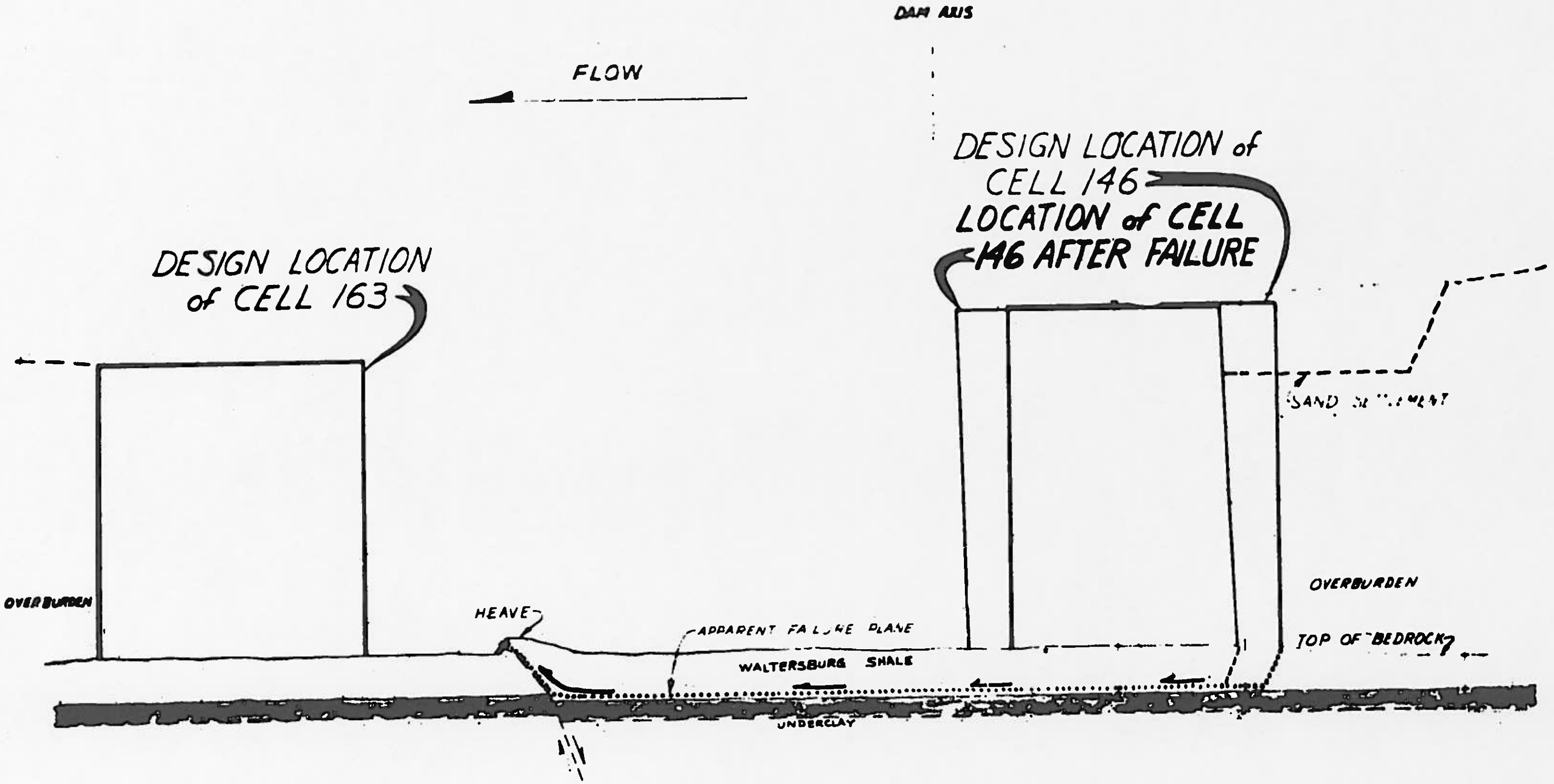
EXHIBIT 20

FAILURE AFTERMATH (arrow). Flooding and sand dumping stabilized cells.



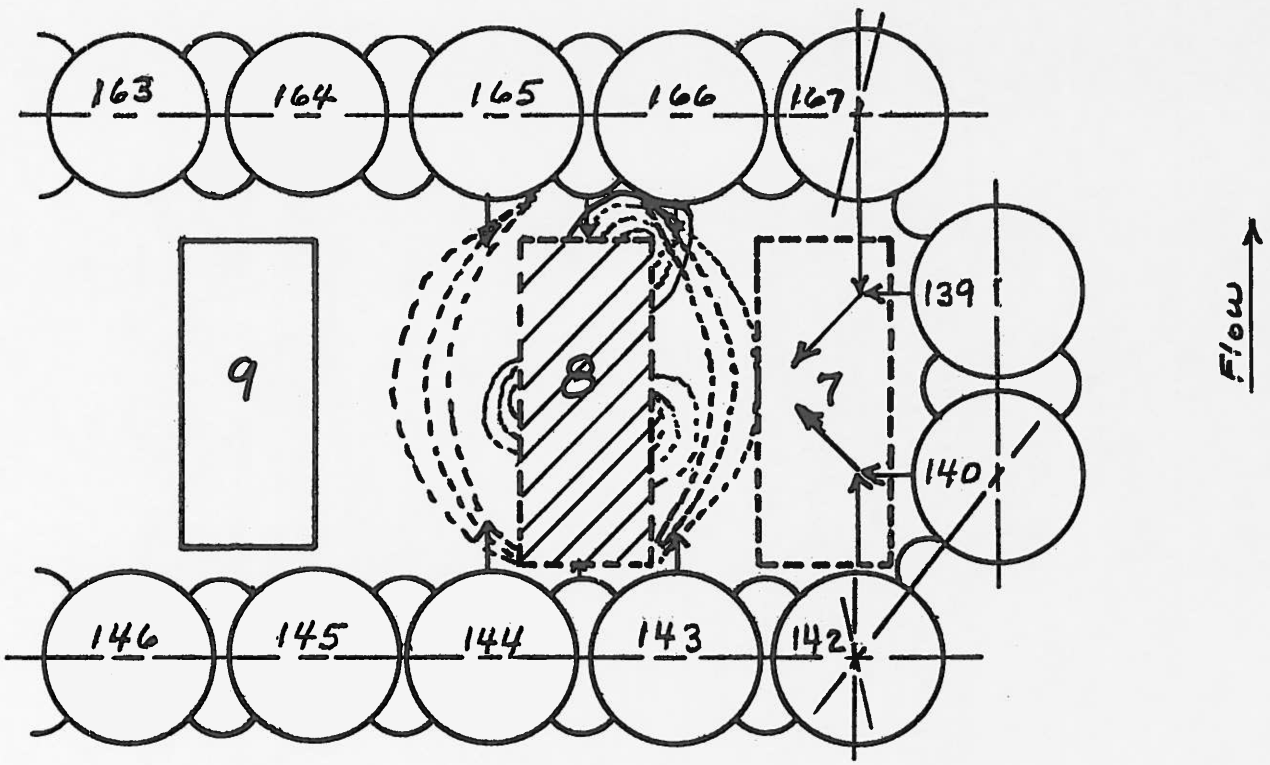
COFFERDAM in the Ohio River is a 20-acre, 160-ft hole; arrow marks ruptured cell, slippage area.

EXHIBIT 21



SECTION THROUGH FAILURE AT CELL 146
SCALE: 1"~10

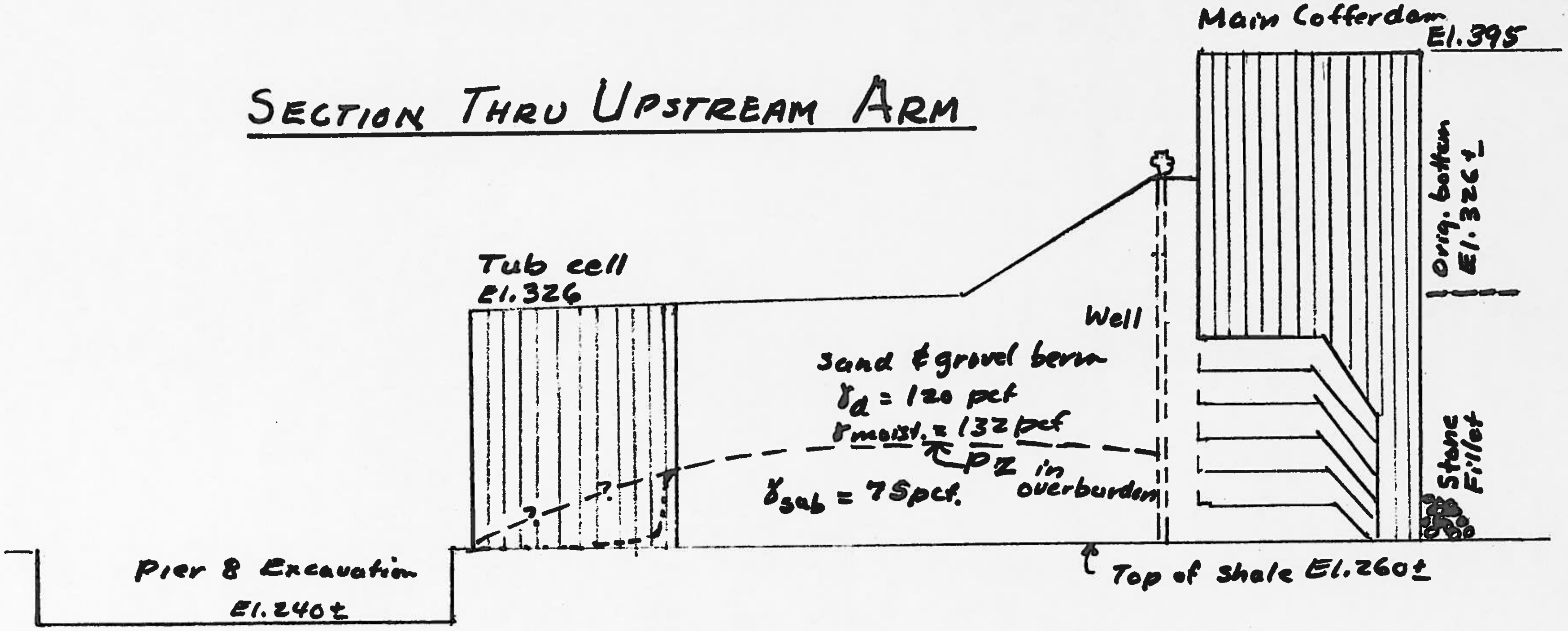
EXHIBIT 22



LOADING STRESS

EXHIBIT 23

SECTION THRU UPSTREAM ARM



Compute thrust on tub cell: Assume earth pressure at rest $K = 0.5$

$P_1 = \frac{1}{2} K \gamma H^2 = \frac{1}{2} \times .132 \times .5 \times (42)^2 = 58^K$ Moist S&G

$P_2 = \frac{1}{2} \times .5 \times .075 \times (24)^2 = 10.9^K$ sub. S&G

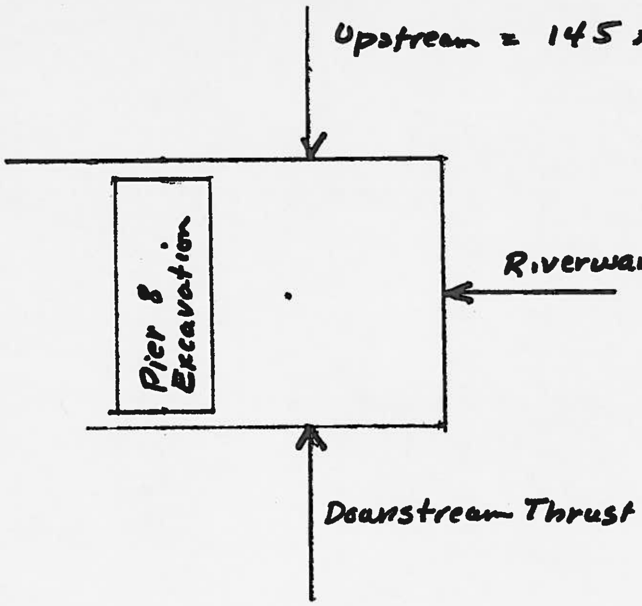
$P_3 = \frac{1}{2} K \gamma h \cdot h_1 = \frac{1}{2} \times .5 \times .132 \times 42 \times 24 = 66.9^K$

$P_4 = \frac{1}{2} \gamma H_1^2 = \frac{1}{2} \times .0625 \times \overline{24}^2 = 18.0^K$ water pressure

total 149.6^K per ft.

EXHIBIT 24

Compute thrust on Shale block riverward of Pier 8

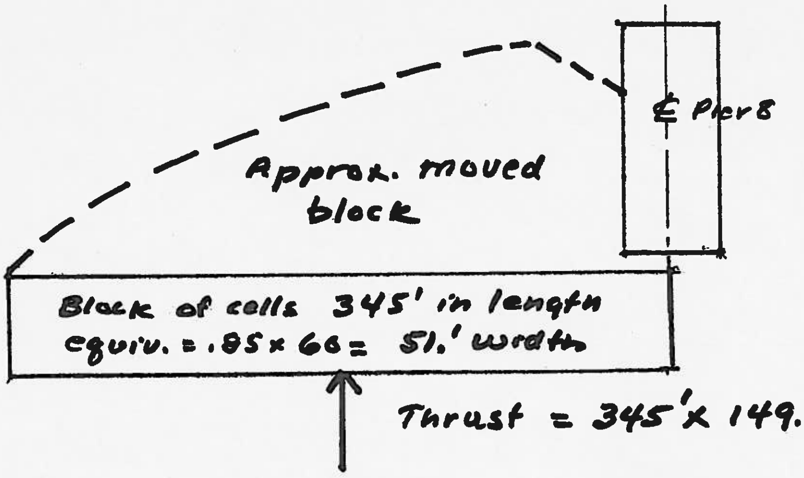


Upstream = 145 × 149.6 = 21,700 Kips

Riverward = 140 × 149.6 = 20,900 kips

Downstream Thrust = 145' × 149.k/' = 21,700 Kips

Compute shear strength of failed block (1 Nov 67)



Bouyant wt. of block	= 30,000 k
Wt. of cells	= 175,000 k
Wt of shale under cells	= 16,130 k
Total	221,130 Kips

Thrust = 345' × 149.6 k/' = 51,600 Kips

Assume cohesion = 0 compute tan⁻¹ for S.F. = 1.0

$$\tan^{-1} = \frac{\Sigma H}{\Sigma V} = \frac{51,600}{221,130} = .233 \quad \phi = 13.1^\circ$$

say $\phi = 13^\circ$

EXHIBIT 25