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DEEP EXCAVATION TECHNIQUES

for

SHELTERS IN URBAN AREAS

(PRELIMINARY PLANNING AND COST DATA)

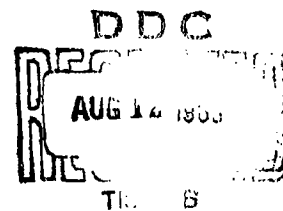
Prepared Under Contract

with the

Office of the Chief of Engineers, U. S. Army

for the

Office of Civil Defense, U. S. Department of Defense



JULY 1963

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FINANCIAL 8-0424

July 22, 1963

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Subject: Report on "Deep Excavation Techniques for
Shelters in Urban Areas (Preliminary Planning
and Cost Data)"

We are transmitting to you herewith the final report
entitled above in the number of copies shown on the attached
distribution list.

This transmittal is in accordance with the request of the
Office of Civil Defense, Department of Defense, and the Chief
of Engineers, Department of the Army.

This is for your information, use, and file.

Very truly yours,

DE LEUW, CATHER & COMPANY


R. H. Anderson
Vice President

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DEEP EXCAVATION TECHNIQUES
for
SHELTERS IN URBAN AREAS
(PRELIMINARY PLANNING AND COST DATA)

Prepared Under Contract
with the
Office of the Chief of Engineers, U. S. Army
for the
Office of Civil Defense, U. S. Department of Defense

Contract No. DA-49-129-ENG-507

This report was prepared for the Office of Civil Defense,
Department of Defense, under OCD-05-62-160 (Research
Subtask 1153C)

OCD REVIEW NOTICE

This report has been reviewed in the Office of Civil Defense
and is approved for publication. Approval does not signify
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JULY 1963

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165 WEST WACKER DRIVE
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FINANCIAL 6-0424

July 22, 1963

Office of the Chief of Engineers
U. S. Army
Washington, D. C.

Gentlemen:

In accordance with our Contract No. DA-49-129-ENG-507, dated 27 June 1962, we are pleased to submit herewith our report on Deep Excavation Techniques for Shelters for Urban Areas. This report covers techniques and cost studies for excavation in open cut and tunnels in earth and rock.

Specific applications are made to case studies which relate to the type, size and depth of excavations and ground conditions as defined in our contract and as further defined by subsequent conferences with your staff. By analysis of a range of these excavation parameters and types, general cost curves and data are derived and presented for guidance use in further allied studies.

New and promising developments in equipment and techniques are covered with respect to their application to the excavations under consideration.

We acknowledge the fine cooperation extended to us by your staff through the course of these studies, and wish to express our appreciation for the opportunity of being of service to you in this most interesting assignment.

Very truly yours,

DE LEUW, CATHER & COMPANY


R. H. Anderson
Vice President

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SCOPE OF WORK

This report, prepared for the Office of Chief of Engineers, Corps of Engineers, under Contract No. DA-49-129-ENG-507, presents the findings, evaluations and analyses developed from engineering studies of deep excavation techniques suitable for shelters in urban areas. The types of excavation, on which these studies are based, include open cuts, tunnels in earth, and tunnels in rock. Conditions are set forth herein relating to the sizes, depths, and other relevant geometry of these excavations.

The studies presented herein are designed to illustrate, in a broad sense, application of appropriate techniques of excavation to general subsurface conditions which occur in the continental United States. Specific subsurface conditions for open cut excavations and general parameters of conditions for tunnel excavations are established to provide a practicable range of study results.

These conditions are analyzed individually, with respect to the type and geometry of the excavation, for evaluation of an effective and economical technique of excavation, and from that evaluation the determination of related cost factors. Summation of these cost factors and further analysis is presented.

The results of review of the general field of excavation techniques as pertains to this report are also presented. These include description and evaluation of techniques, utilizing materials and equipment in common and special usage and proved to be of economic value, techniques under development which would be appropriate for consideration, and undeveloped techniques which might be considered promising with respect to excavations contemplated herein.

CONSIDERATIONS AFFECTING EXCAVATION METHODS

It is of prime importance, particularly considering the research rather than operational characteristics of these studies, to establish a perspective against which the assumptions, analyses and findings can be weighed, accepted or modified in application of this data.

Approach to the Problem

It is obvious that a definitive effort to set forth the proper technique of excavation to be applied to every possible subsurface condition to be encountered in the United States, establishing costs thereof, would be an expenditure of time and money not justifiable on a practical basis, even if such would be possible. It is viewed, however, that a realistic approach can be made to this problem by some generalization in the various conditions which might be encountered, sufficient to produce a range of cost results indicative of the interrelated effect of technique, subsurface conditions and type and size of excavations. These generalizations, resulting in specific study examples, are discussed in detail later in the report.

Limitations Imposed by Urban Conditions

The scope of work qualifies that these excavations, both open cut and tunnel type, would be situated in urban areas. It has been established that these sites would not be concentrated in the central business districts, but would more probably be located in and around the urban peripheries where a majority of the population would spend a majority of its time. No stipulations have been provided or established as to the character of the sites, whether in parks, on cleared land in connection with urban renewal or connected programs, on presently vacant land, or in street rights of way. However, it remains important to recognize that the urban conditions which would surround such sites impose requirements on even this general study. For example, it would not be realistic to assume that the removal of material from these excavations could be carried out with "off the road" equipment, since such transport would most probably be via urban thoroughfares. Further, conditions, which cannot be evaluated in these studies, would definitely be imposed on the real projects envisioned, such as the relocation and maintenance of utilities and shoring or underpinning of adjacent buildings. Each of these and other corresponding conditions would have considerable, and often great, economic effect on the excavation phase of the projects.

Subsurface Investigations

It is quite apparent that any program for construction of underground shelters would contain as a key phase studies relating to the proper selection of sites with respect to function and service to the populace. It is of equal importance, on a concurrent basis, to devote sufficient time and funds to preliminary subsurface exploration of all sites under consideration, with more detailed investigation devoted to the more favored sites. Subsurface conditions can readily be predicted to be of changing nature from block to block, not alone mile to mile, and any assumptions to the contrary could result in detrimental, if not disastrous, effect on the excavation operations at the selected site. It is the experience of the engineering and construction profession that even with adequate subsurface investigation, and with ample experience and planning devoted to the project, adverse conditions are encountered during construction which must be dealt with. Many times these conditions have resulted in major delays and increased costs, and, in some cases, caused a major change in the techniques and equipment initially utilized. Consequently, it is mandatory that a program involving excavations of the nature contemplated would include a subsurface investigation of adequate scope, coupled with engineering planning and design experience. Of no less vital importance are the contractor capabilities which would be called on to translate the planning into a construction project.

Cooperative Efforts

Recognition should be given to the potentialities of ingenuity in approach to unique construction problems, such as might be presented in a broad program envisioned herein. This ingenuity can be initiated by the engineer who plans the work, by the contractor who carries the design to reality, or by a joint and cooperative effort by both. However, it might be sometimes overlooked that real contributions can be made by the manufacturers of excavating equipment and tools in the formulation of approach to a difficult project. Every opportunity should be taken to allow the combination of these talents for that purpose.

Techniques and Equipment

For every project, it is possible, if not probable, that there is more than one economical and proper technique which could be utilized to achieve an identical end result. The selection of method should be left somewhat flexible to allow for an ingenious approach or use of available but suitable equipment. This proviso assumes that qualified engineers carry the design through supervision of construction for

proper control of techniques utilized. Although beyond the scope of this report, consideration should be given also to prequalification of contractors for the type of excavation considered to assure that award of work would be tantamount to a satisfactory construction project. This might also include the approval of techniques and equipment appropriate to the project.

Cost Considerations

The studies contained in this report are primarily based on the use of conventional techniques falling within the normal capabilities of the average heavy construction contractor of this country. It is anticipated that a program of this scope would be placed in action by the usual practice of award of contract to a successful bidder after public invitation of bids. Costs of contract work would depend on the location, the country's economic picture at the time, and volume of work available, as well as on the character of the work. It is evident that these outside effects could produce a range of costs which would override the differential in costs estimated using two dissimilar but workable techniques.

Construction Time

The studies in this report do not embody time as a factor, since no criteria is available for that guidance. An effort has been made to provide a reasonable relationship between work and equipment to produce a median cost with the factors contained herein as a base. However, at time of national emergency, it is recognized that time would probably be of the most vital significance in the construction of shelters.

FACTORS AFFECTING ECONOMY AND TECHNIQUE OF EXCAVATION

OPEN CUT EXCAVATION

Geological Conditions

The subsurface materials and conditions at a selected site directly affect the excavation techniques to be utilized and, consequently, the costs of excavation. The importance of this fact warrants, as background, a thorough study of the physiographic province in which the site is located. A program of test borings should be designed to determine as closely as possible the actual conditions which would be encountered, with recourse to any information of value relating to previous investigations or construction at nearby locations.

The number of test borings and the depth of penetration would be determined by the characteristics of the material, whether somewhat uniform or considerably variable, and by the intended area and depth of the excavation. The type of sampling procedure would be designed to provide the best information regarding the nature and depth of earth materials and rock and the location of the water table or presence of artesian water. This information would be properly shown in the form of a soils profile so that each geological stratum and expected water condition could be readily seen and studied in the planning of the excavation and the construction to follow. It is of particular importance that any information relative to the quantity of flow of water which could be expected during excavation and construction should be obtained.

The appropriate excavation techniques can then be readily determined by the analysis of the soils profile. There will be few instances where the excavations envisioned herein will proceed through uniform materials throughout their depth and areas, allowing simplicity of technique in attack. The quantities of each type of material to be encountered will govern to a considerable degree the techniques adopted for the entire excavation. Obviously, the presence of rock in the volume of material to be excavated will cause the adoption of at least two separate modes of attack although the supporting equipment may remain the same.

Some comments on the characteristics of general classifications of subsurface materials are presented below with respect to their effect on the method of excavation. Considerations relating to dewatering or the stabilization of these materials are covered in a following section.

Sands and Gravels. These materials are easily excavated, and may be removed by almost any feasible method, appropriate to the location and conditions affecting the excavation process. The consideration of using conveyors as a means for removing spoil from the larger, deeper excavations is particularly appropriate where large quantities of these materials are present.

A suitable factor of safety of stability may be had in these materials using 1:1 slopes,

Practical and economical excavation techniques require that a method of dewatering be employed when excavations of sand or gravel must proceed below the water table. This requirement will exert great influence over the cost factors, not only of excavation, but construction and operation of the intended facility. After proper dewatering, the excavation could proceed in the conventional manner.

Silty or Non-Cohesive Soils. When found in a dry state, soils of a fine-grained, non-cohesive nature could be excavated economically with scrapers or shovels, with stable slopes in the range of 1:1.

However, when these materials are to be excavated below the water table, many diverse problems complicate the excavation procedure. If, for example, the material below the water table is a fine sand, it will exhibit the tendency to flow into the excavation unless steps were taken to halt this flow. Dewatering or similar stabilizing techniques usually prove satisfactory in this or similar material having some degree of permeability. Finer grained materials will cause even more costly measures to be taken to stabilize the excavation, due to the low degree of porosity, and in many cases, will require the use of sheeting or similar retainage device to allow the excavation to proceed to completion.

Ground water under hydrostatic head, as might be encountered in an aquifer confined below a non-permeable stratum, will bring about a "quick" condition in the excavation, wherein the weight of the materials is balanced by the upward flow. An extensive occurrence of this condition can make open cut excavation in this stratum an impractical matter. Considering the problem of flotation and handling this flow for the duration of the project life, it would be appropriate to consider abandonment of any site found to be subject to this condition.

It is again emphasized that subsurface water plays an important, and sometimes deciding, role in the techniques, methods and costs of excavation, not only by its presence, but by its flow characteristics, whether upward, horizontal, or merely static.

Clays. When dry to moist clays are encountered above the water table, there is seldom any problem in their excavation. If they are extremely dry, it may be necessary to use rippers or similar devices for loosening them before loading into transporting equipment. Clays containing some moisture can be handled effectively by scrapers or shovels.

Clays under optimum conditions will stand vertically to as much as 20 and sometimes 30 feet of depth, on a temporary basis.

When clays are encountered below the water table or when they have an extremely high moisture content, their excavation becomes quite difficult. Soft clays cannot be drained; hence usual water control methods become unworkable. Stabilization could involve such techniques as the use of braced sheet piling or cofferdams constructed around the periphery of the excavation. It is obvious that such techniques are extremely costly and practicable only within certain depth ranges.

Certain soft clays exist which would prohibit an open excavation of greater than a nominal depth, for example, of 15 to 20 feet. Some soft clays exert tremendous lateral pressures, and unless the material can be prevented from squeezing into the excavation, detrimental settlements to adjacent areas may result. Some soft clays exhibit swelling or heaving

characteristics, resulting from removal of the overburden. In fact, these clays will move up from the bottom into an excavation. This phenomenon is also often accompanied by settlement in adjacent areas.

These very soft unstable clays would probably be best removed by dragline from outside the excavation area where shallow excavations will suffice for the purpose intended.

Reference is made to Exhibit 1, Unified Soils Classification System, which further defines the major soils divisions and groups and the characteristics of each.

Igneous Rock is that material which has solidified from the mass which made up the earth in its original molten state. Intrusive igneous rocks, such as granite or diorite, are those that intruded into the lower voids of the earth's crust, cooling under pressure from being confined. Extrusive igneous rock, for example basalt or diabase, was formed when the molten rock burst through the earth's surface, cooling rapidly without pressure.

Igneous rock is nearly always very hard and requires considerable amount of energy to dislodge from the excavation. In almost all cases, its removal will require blasting. Normally after blasting, loading would be performed by shovels into trucks, or, if the method of blasting produced a fine enough rock, it could be moved from the excavation by conveyor belts.

Excavation in igneous rock below the water table would not prove difficult unless the rock has been subjected to severe earth pressures causing faulting or jointing of the rock which would permit large quantities of water to enter the excavation. In most cases, the normal flow of water could be handled by the appropriate use of surface ditches and sump pumps in the excavation area. Stable slopes in igneous rock can almost always be made vertical, or at the maximum not flatter than 1/4 : 1. However, benching is usually necessary to prevent the danger of rocks from falling into the excavation.

Sedimentary Rocks. In a broad sense, these rocks may be described as those materials cemented together after being precipitated in or evaporated from water after being disintegrated from other rock types or resulting from the deposit of

CIVIL DEFENSE SHELTERS
IN URBAN AREAS
UNIFIED SOIL CLASSIFICATION SYSTEM
DE LEUW, CATHIER & COMPANY - CONSULTING ENGINEERS - CHICAGO

1	2	3	4	5	6	7	8
MAJOR DIVISIONS	SOIL GROUPS & TYPICAL NAMES	GROUP SYMBOLS	DRY STRENGTH	OBSERVATIONS AND TESTS RELATING TO MATERIAL IN PLACE	PRINCIPAL CLASSIFICATION TESTS ON DISTURBED SAMPLES	SHRINKAGE, EXPANSION, ELASTICITY	DRAINAGE CHARACTERISTICS
GRAVEL AND GRAVELLY SOILS	BELL GRADED GRAVEL & GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	GM	NONE	DRY UNIT WEIGHT OR VOID RATIO DEGREE OF COMPACTION, OR RELATIVE DENSITY CEMENTATION, DURABILITY OF GRAINS, STRATIFICATION & DRAINAGE CHARACTERISTICS, GROUND WATER CONDITIONS.	MECHANICAL ANALYSIS	ALMOST NONE	EXCELLENT
	BELL GRADED GRAVEL-SAND-CLAY MIXTURES, EXCELLENT BINDER	GC	MEDIUM TO HIGH		MECHANICAL ANALYSIS, LIQUID & PLASTIC LIMITS ON BINDER	VERY SLIGHT	PRACTICALLY IMPERVIOUS
	POORLY GRADED GRAVEL & GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	GP	NONE		MECHANICAL ANALYSIS	ALMOST NONE	EXCELLENT
	GRAVEL WITH FINES, VERY SILTY GRAVEL-CLAYEY GRAVEL, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES	GF	VERY SLIGHT TO HIGH		MECHANICAL ANALYSIS, LIQUID & PLASTIC LIMITS ON BINDER IF APPLICABLE	ALMOST NONE TO SLIGHT	FAIR TO PRACTICALLY IMPERVIOUS
SANDS AND SANDY SOILS	BELL GRADED SANDS & GRAVELLY SANDS, LITTLE OR NO FINES	SM	NONE	DRY UNIT WEIGHT, WATER CONTENT & VOID RATIO, CONSISTENCY - UNDISTURBED & REMOLDED, STRATIFICATION, ROOT HOLES, FISSURES, ETC., DRAINAGE & GROUND WATER CONDITIONS, COMPRESSION TESTS	MECHANICAL ANALYSIS	ALMOST NONE	EXCELLENT
	BELL GRADED SAND-CLAY MIXTURES, EXCELLENT BINDER	SC	MEDIUM TO HIGH		MECHANICAL ANALYSIS, LIQUID & PLASTIC LIMITS ON BINDER	VERY SLIGHT	PRACTICALLY IMPERVIOUS
	POORLY GRADED SANDS, LITTLE OR NO FINES	SP	NONE		MECHANICAL ANALYSIS	ALMOST NONE	EXCELLENT
	SAND WITH FINES, VERY SILTY SANDS, CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES	SF	VERY SLIGHT TO HIGH		MECHANICAL ANALYSIS, LIQUID & PLASTIC LIMITS ON BINDER IF APPLICABLE	ALMOST NONE TO MEDIUM	FAIR TO PRACTICALLY IMPERVIOUS
FINE GRAINED SOILS HAVING LOW TO MEDIUM COMPRESSIBILITY	SILTS (INORGANIC) & VERY FINE SANDS, NO. 60, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS WITH SLIGHT PLASTICITY	ML	VERY SLIGHT TO MEDIUM	DRY UNIT WEIGHT, WATER CONTENT & VOID RATIO, CONSISTENCY - UNDISTURBED & REMOLDED, STRATIFICATION, ROOT HOLES, FISSURES, ETC., DRAINAGE & GROUND WATER CONDITIONS, COMPRESSION TESTS	MECHANICAL ANALYSIS, LIQUID & PLASTIC LIMITS IF APPLICABLE	SLIGHT TO MEDIUM	FAIR TO POOR
	CLAYS (INORGANIC) OF LOW TO MEDIUM PLASTICITY, SANDY CLAYS, SILTY CLAYS, LEAM CLAYS	CL	MEDIUM TO HIGH		LIQUID & PLASTIC LIMITS	MEDIUM	PRACTICALLY IMPERVIOUS
	ORGANIC SILTS & ORGANIC SILT-CLAYS OF LOW PLASTICITY	OL	SLIGHT TO MEDIUM		LIQUID & PLASTIC LIMITS FROM NATURAL CONDITION & AFTER OVEN DRYING	MEDIUM TO HIGH	POOR
	MICACEOUS OR DIATOMACEOUS FINE SANDY & SILTY SOILS, ELASTIC SILTS	MH	VERY SLIGHT TO MEDIUM		MECHANICAL ANALYSIS, LIQUID & PLASTIC LIMITS IF APPLICABLE	HIGH	FAIR TO POOR
FINE GRAINED SOILS HAVING HIGH COMPRESSIBILITY	CLAYS (INORGANIC) OF HIGH PLASTICITY, FAT CLAYS	CH	HIGH	CONSISTENCY, TEXTURE AND NATURAL WATER CONTENT	LIQUID & PLASTIC LIMITS	HIGH	PRACTICALLY IMPERVIOUS
	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY	OH	HIGH		LIQUID & PLASTIC LIMITS FROM NATURAL CONDITION & AFTER OVEN DRYING	HIGH	PRACTICALLY IMPERVIOUS
	PEAT AND OTHER HIGHLY ORGANIC SWAMP SOILS	PT	SEADILY IDENTIFIED			VERY HIGH	FAIR TO POOR
	FIBROUS ORGANIC SOILS WITH VERY HIGH COMPRESSIBILITY						

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the remains of great quantities of organisms. Sedimentary rocks present a wide variety of physical characteristics, varying from the extremely hard sandstones and limestones to the soft shales and loosely cemented conglomerates. It is, therefore, expected that excavation techniques in this material will vary considerably, depending upon the nature of the rock and its physical characteristics. Conglomerates and soft sandstone can usually be loosened with a ripper and excavated similar to equivalent soils. Limestones and hard sandstones will invariably have to be blasted for removal. Appropriate excavation slopes in the harder materials may vary from vertical to 1/2 : 1 dependent on the characteristics of the rock. Sedimentary rock encountered below the water table can be excavated in the same manner as carried out above the water table. In the case of some limestones or sandstones which have a high porosity, it would be necessary to control the water from outside the excavation area. If the porosity is extremely low and the rock is neither faulted nor jointed, any water that would be encountered could usually be handled with sump pumps in the interior of the excavation.

Perhaps the most troublesome type of sedimentary rock would be the shales. These rocks are made up of clays which have been compressed, reducing the void ratio and moisture content. As a consequence of removing overburden or side protection such as in an open excavation where water may come in contact with the shale and pressures are removed, it is normal that these shales will expand. This swelling characteristic endangers excavation slopes. The site soils testing program should include laboratory tests on any shales encountered relating to their swelling tendencies.

Metamorphic Rocks. These rocks represent the results of a process of recrystallization of rocks or earth taking place under high temperature and pressure conditions. The properties of the product depend on the nature of the original material and, to a considerable extent, on the deformation associated with the process.

Metamorphic rocks are usually hard and require blasting prior to excavation. They would be treated in essentially the

manner as igneous rock. Some metamorphic rock, such as slate, which were derived from shales, will exhibit swelling characteristics.

The most adverse characteristic of metamorphic rock would relate to the stability of slopes. These rocks normally are uplifted and faulted, introducing the problem of spalling and slope instability in deep excavations. Satisfactory treatment would include benches in the slopes at 30 to 50 foot intervals vertically and barricades to prevent the fall of rock into the excavation.

When these rocks are encountered below the water table, the same problems and techniques of excavation used are encountered as with igneous and sedimentary rock.

Water Control

Although physically part of the geological conditions, the presence and control of ground water is probably the most important single factor influencing methods and costs of open cut excavation, and consequently warrants a separate discussion. For the open cut excavation envisioned herein, there will be only a limited number of sites where ground water will be located at levels 150 feet or more below the surface of the ground. It will be vitally important, therefore, in the program for deep shelters envisioned herein to give priority attention to the matter of water control.

Selection of the proper method of control of water in an earth-work operation requires some knowledge of the interaction between soils and water, and more specifically with reference to various soil grain sizes. The very fine grain soils, such as clays, will be affected only slightly by the presence of a very small amount of water. In fact, in the geological processes of the past, when water was removed by overburden pressures and the clay particles compressed, the clays were changed into shale. As the moisture content of clay increases to approximately 20 to 30 percent of the dry weight, the clay changes from a stiff to a more plastic material, decreasing the inherent stability of the material.

Silt particles, the next larger grain size, remain relatively unchanged with the introduction of a small amount of moisture, except

possibly in the case of deep cuts when the material could become unstable. Dry loess, which is a wind blown deposit of silt size particles, can be excavated on a vertical slope; however, as the water content increases, the problems increase substantially. The grain size of silt is such that water does not flow readily through it, and when it does flow, it has a tendency to remove particles at the interface of more porous media, presenting an unstable condition due to erosion.

Fine sands will exhibit a very small amount of cohesion if they contain a very low moisture content. Here again, the permeability, although greater than exhibited by silt, is still relatively low.

Coarse sands and gravels are relatively unaffected from the standpoint of stability, and normally are considered free draining even under minimum moisture conditions.

Water encountered in soils can be considered in two states: adsorbed water and free water. Adsorbed water is that portion of the moisture content in the soil that clings tightly to each individual soil particle. It can seldom be removed even under the most favorable drainage conditions. Free water is that part of the moisture content which is separated from the soil particles at such a distance that there no longer occurs an electro-chemical attraction. Free water can flow provided there is sufficient space between the particles and enough head imposed on the water to provide pressure.

From the above, it can be seen that it is of extreme importance in any subsurface investigation leading to an excavation project to determine not only the grain size of the soil particles, but also an indication of the amount of water that can be anticipated in the finer grain soils, usually referred to as the moisture content.

It is extremely desirable in the coarser grain soils to make a determination of the quantity of flow which might be encountered. It is preferable that this information be gained through pumping tests, and if this is not economically feasible, this determination could be approximated by calculations based on the relationship between grain size of the native soils to the permeability coefficient of those soils.

After the type of material and quantity of water has been determined, the procedures and techniques for control may be selected. These techniques can be divided into two categories, both of which are designed to provide stable soil conditions. The first technique is to

promote stability by removal of the water as subsequently discussed under Dewatering Systems. The second technique is to promote stability by the use of an additive to the soil-water complex which strengthens the soil or reduces the permeability or both without removing the water.

Dewatering Systems

Wellpoints. Probably the most common technique for dewatering soils involves the use of wellpoints. This system employs a steel casing equipped with a strainer at the bottom, located in the water bearing soil. The top of the casing would be connected to a header pipe leading to a centrifugal pump which draws the water from the ground adjacent to the casing. Water in the vicinity of the wellpoint would be forced toward the intake by atmospheric pressure.

Removal of the water by this technique depresses the normal water table to the desired elevation so that excavation can be carried out in the dry. The maximum suction lift that can be attained in a single stage wellpoint system is about 22 feet. However, in coarse grain soils where the water volumes per wellpoint exceed five gpm, the attainable suction lift may be less because of friction. In the finer grain soils, such as fine silts or sands, special vacuum equipment can be provided in the pumping system increasing the suction lift to as much as 26 feet. Where necessary to remove water from levels deeper than the above, it becomes mandatory to utilize a double or triple stage wellpoint system, or resort to other types of equipment.

Water Ejectors. Water ejectors or eductors are utilized for lifting water for greater heights than is possible with wellpoints. This high lift system consists of two pipes placed into the ground with a parallel header system. One header pipe contains water flowing downward under pressures up to 150 psi. The other is the collector or ejector main. The high pressure water is directed to the point of the wellpoint screen where it is forced upward through an orifice, causing a suction on the water in the surrounding soil. This system is capable of lifting water to a height of 50 feet. Due to power considerations, the

maximum practical capacity of an ejector-operated high lift system is 1500 to 2000 gpm.

Deep Wells. When the material is very coarse and the excavation must extend to a considerable depth below the water table, it becomes desirable to consider the use of deep wells. Another condition appropriate for such consideration might occur where artesian water confined below an impervious layer would have to be controlled and removed from the excavation site. These deep wells are, in effect, a conventional water well extending below the bottom of the excavation accepting the ground water which is removed from the surface by a submersible pump at the base of the well.

Electro-Osmosis. Some soils, such as silts, clayey silts, and fine clayey-silty sands that will not drain by gravity can be drained by wellpoints provided that there is some assistance in attracting the moisture or free water to the wellpoint. One method involves the electro-osmotic, or electrical drainage, process, and is accomplished by driving two electrodes into saturated soil. When an electric current is passed between the two electrodes, water will flow from the positive electrode to the negative electrode. By making the cathode--or negative electrode--a wellpoint, water may be removed by pumping.

Open Drainage and Sump Pumps. When the amount of water which will enter an excavation is determined to be limited in quantity without damage to slopes, it can be channeled into surface ditches draining into a sump. Water collected at several points is then removed from the excavation with sump pumps.

Filter Blankets. In the use of open drainage and sump pumps, care should be taken to assure that the finer particles are not being removed from the soils strata. If this condition occurs, consideration should be given to the installation of a filter blanket placed over the slope made up of increasingly coarser materials permitting the water to pass through the blanket, yet holding the finer soils in place. Water permeating through the filter blanket can be drained down the slope into the open drainage system.

Recharging Ground Water. In combination with a dewatering system, a recent innovation was successful in the construction of apartment houses with deep foundations on sand. This was a novel method which consisted of recharging the ground water by diffusion of pumped water to prevent settlement damages to existing nearby buildings while the ground water table was lowered about 12 feet to permit foundation construction in the dry.

Methods of Stabilization

In the broader sense, soil stabilization may be considered as the process of altering soil properties to improve their engineering performance. However, this portion of the report pertains only to those stabilization techniques, methods, and procedures that will be applicable to aiding the engineer or contractor in excavating below the ground surface or in holding back water as a means of water control. It should be pointed out that all of the methods described above under "Dewatering" would, in practical application, add an element of stability to the material after the water has been removed.

Thermal Stabilization. The heating or freezing of a soil can cause considerable change in its physical properties. In the clay soils, the temperature required to cause complete stability is probably too costly to be of practical significance, although for limited applications, it could have favorable results. An interesting innovation, used in the Soviet Union, involves the injection into the ground of a controlled mixture of liquid fuel and air at a pressure of approximately .3 atmosphere through a network of pipes at a typical spacing of three meters. The mixture is fired for a period of 10 to 12 days and produces a cylinder of solidified soil about nine feet in diameter. The maximum depth to which stabilization has been effected is about 40 feet. This injection process of hot gases is applicable only in non-saturated soils.

The most important method of thermal stabilization involves the freezing of the pore water or the free water in soil. Water in cohesionless soil freezes at or slightly below zero degrees centigrade. However, water in clays must be cooled lower than zero degrees to be frozen. Freezing is normally a

temporary stabilization process and once the soil has been frozen, it must be maintained in that condition. Consideration should also be given to heaving of the soil as a result of freezing.

Cement Grouting. Grouting with cement can be utilized in granular soils to make the soil stable and prevent them from moving into an excavation or, in the form of a light grout, to reduce the permeability of the soil and thus aid in the water control problem. There are many references related to cement grouting, and before such a procedure is used, the best technical advice should be obtained. One disadvantage of cement grouting relates to the lack of control on setting up time.

Chemical Grouting. There is a considerable variety of chemical compounds which have been proposed for use in grouting. The selection of the appropriate type of chemical would depend upon the requirements involved, as well as the type of materials into which the chemical will be grouted.

Chemical grouts can be utilized under flowing water conditions. They require close control of grouting operations because of their characteristics regarding time of setting.

The progress report of the Task Committee on Chemical Grouting of the Committee on Grouting of the Soils Mechanics Foundation Division of the ASCE constitutes the best available information on chemical grouting.

Soil Grouting. In general, soil grouting employing a clay slurry is usually the least expensive type of grouting. Its application is limited, however, because the material is difficult to inject, has very little cementing action and the amount of reduction in the permeability is very small.

Exhibit 2 indicates a comparison of a few appropriate methods of stabilizing or dewatering soils as related to soil grain size.

Dangers of Dewatering

One of the principal sources of danger in connection with dewatering involves the loss of buoyancy in granular materials, automatically increasing the effective stress, and, in turn, causing settlement in cases where soft layers occur within the effective zone beneath the foundations. It is therefore imperative before dewatering techniques are employed in urban areas that the

possibility of settlements to adjacent structures be thoroughly investigated.

Another source of considerable danger in the use of wellpoints is that the extreme fines can be filtered through the pumping system, causing a loss of ground and subsequent settlement. If it is found that fines are passing through a pumping system, this may be corrected by placement of a graded filter at the wellpoints. The problem of losing ground, however, is more prevalent in the gravity flow of water into sumps.

Costs

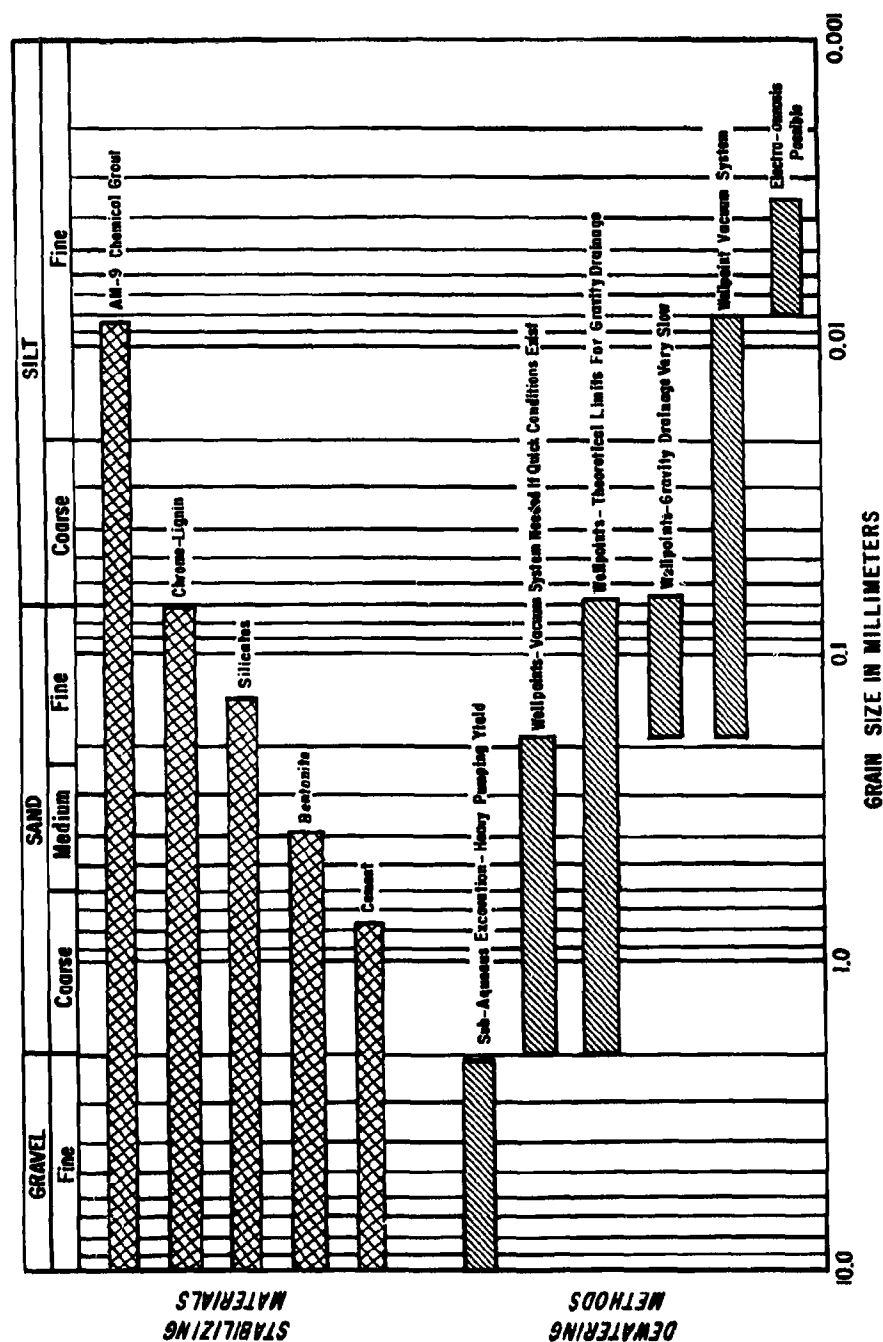
The costs of the above methods vary considerably dependent upon the type of material and the type of dewatering or stabilizing technique that is used. There is no standard or even average price for any of the methods mentioned, and it would be necessary in each instance to appraise the potential costs with the necessary basic information at hand.

Sheet Piling and Cofferdams

In cases where the dewatering or stabilization could not increase stability sufficiently to permit stable slopes, it may be necessary to use sheet piling or similar retainage device, for instance in soft clays where stabilizing or dewatering techniques are ineffective.

Site, Design and Construction

Sites selected for "cut-and-cover" shelter construction requiring open cut excavations could be bounded by residential units, light commercial establishments, or heavy office and industrial buildings. Foundations could range from shallow, soil-bearing footings to footings on piles or caissons extending to deep, firm bearing. The nearness and characteristics of these improvements would affect to a large degree the adaptability of a site for open cut excavation, particularly in the greater depths considered. This consideration and the often desirable utilization of the entire



DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS

COMPARISON OF METHODS FOR STABILIZING AND DEWATERING SOILS

DE LEUW, CATHIER & COMPANY - CONSULTING ENGINEERS - CHICAGO

COURTESY OF AMERICAN CYANAMID COMPANY

site could make necessary shoring, or otherwise supporting, the sides of the excavation. This would result in a cost premium, in many cases, compared to the alternative method of sloping those sides. These total costs could be further increased by this requirement because of a decrease in accessibility to the working area, imposing a less productive technique in the excavation of the site.

As previously stated, it is expected that few sites would be located in business districts where major underground utilities would be located within street rights of way. However, in the streets of urban peripheries, there also exist many sub-trunk and lateral utilities of considerable size and importance where interruption or failure of service would not be permitted. While the existence of these utilities may not adversely affect an otherwise acceptable site, their location must be accurately determined, if not known, in order that maintenance of service and adequate protection be provided.

Efficient use of the limited areas available for urban sites will be a prime requirement. Consequently, for those available areas, the shape and size of open cut excavation must be adapted to the configuration and area of the site. These factors will constitute major effects on excavation methods and costs, particularly to the extent that they affect the relationship of depth to surface dimensions. To elaborate on this statement, the site configuration and resulting design of the excavation will have direct influence on:

- (a) The type and use of excavating and hauling equipment;
- (b) The method for moving excavating equipment into and out of the hole; and
- (c) The manner in which the spoil is loaded and hauled to temporary stockpiling or permanent disposal area.

As an example, and with respect to (a) above, a site of regular shape, approximating a city block in area could be excavated by an applicable bulk-pit method utilizing power shovels to fill trucks or dump wagons. Assuming the feasibility of ramps of 10 percent grade or less, this efficient method could be carried to the design depth

without modification. On the other hand, a small site, perhaps irregular in shape, might require excavation by a limited area-vertical excavation method. It would be appropriate in this instance to consider skip-boxes filled in the excavation by front-end loaders and lifted to the surface by stiff-legs mounted near the rim of the excavation.

These last mentioned factors will be demonstrated in later sections of the report dealing with specific open cut excavation studies.

TUNNELS IN EARTH

Geological Conditions

Subsurface conditions almost solely dictate the construction methods applicable to tunnels in earth, govern to a considerable extent the design features of those tunnels and, in many instances, are characterized by a lack of uniformity. The early tunnel builders met their greatest challenges when the initial conditions on which they based their methods unexpectedly changed to drastically different conditions, resulting in many instances in delays, abandonment of the work, injuries and fatalities. As their experiences led to the development of safer, more adaptable techniques, the possibility of failure, delay or accident has been reduced to a reasonable risk level. However, the inherent difficulties involved in driving a tunnel through earth and non-uniformity of the subsurface conditions have tended to maintain the active use of the conservative methods derived through experience. Some of these are flexible enough to meet without major modification the changing conditions encountered during construction.

Detailed investigation of the subsurface strata and water conditions along a tunnel route is just as vital to the design and construction of the project as is the subsurface survey for excavation and construction in open cut. However, some general classification in the character of the materials encountered can be made due to the relatively broad capabilities of each tunneling method appropriate to each classification. One authority on tunnel driving defines these classifications or types to be:

1. Running ground, which would include dry sand or gravel, water bearing sand or gravel, silts or muds. These materials must be supported immediately.
2. Soft ground, including soft or squeezing clay, damp sand, soft earth and certain types of gravel. The tunnel roof in these materials must be supported immediately, with the side walls capable of standing vertically for a very short period of time.
3. Firm ground, which includes firm clays, dry earth, cemented sands and gravels and dry loess. The roof

might be left unsupported for a very short period of time and the side walls and face could stand vertically for about an hour.

4. Self-supporting ground, including shales, hard clays and earth, sandstone and certain cemented sands and gravels. These materials will stand unsupported for a distance of four to sixteen feet prior to placing timbering or ribs.

A few of the major equipment and technique applications, suited to these general ground conditions, are described in the following sections.

Methods of Driving

Although the driving of tunnels antedates recorded history, the most rapid growth in the art occurred from the late 19th century to date due to the demand for vehicular tunnels and large water and sewerage conduits. During this period, the development of shields, improved lining materials, mechanized mucking and conveying equipment and water control methods have revolutionized this field of construction. Current developments in new and improved equipment and techniques promise even more progress in this field.

The primary or main earth tunnel excavation in relatively level terrain could be initiated by the sinking of a shaft, as is customary, or by the driving of an inclined ramp tunnel. The method of initiation would be selected by functional requirements, site conditions and economics, but would generally not affect the method of driving the main tunnel.

Tunnels Utilizing Shields. The shield technique is primarily utilized in running and soft to firm grounds, which, because of their instability, require substantial temporary support and shoring of the mined surfaces prior to placement of the primary lining. This method has been used with or without the use of compressed air as required by the characteristics of the conditions encountered. There are several basic types of shields, with variations on these basic designs as might be required by general conditions. In practice, a shield would be designed and constructed for a particular project for the specific conditions established or anticipated.

The most frequently utilized shield is the full circular type and in its simplest form is nothing more than a hollow steel cylinder of slightly greater diameter than the intended tunnel, equipped with a cutting edge at the forward end, skin plates which extend to the rear to form a tail, and hydraulic jacks around the periphery at the rear. Movement and direction of the shield is controlled by pressure of the jacks on the completed primary lining. The tail of the shield is of sufficient length to permit the installation of a section of the primary lining within the protected area, and is designed to withstand the expected soil loading. Normally, the skin plates remain in the tunnel after the end of the drive. The forward edge may be extended at the top and sides to form a hood which is of considerable value when the cross section encompasses loose ground strata above rock. It also offers protection to the miner in material requiring support before each movement of the shield.

Bulkhead type shields include a diaphragm extended over the full area of the section with sliding doors opening into the face area. In running ground, where settlement at the surface would not introduce serious problems, the shield can be moved ahead with the doors open or partly open permitting the material to enter the working area in controlled quantities. The diaphragm also furnishes stiffness to resist soil loading in the larger bores.

The open type shield does not have a bulkhead and can be used for small tunnels in the firmer grounds or where breasting is required.

Tunnels have been constructed in suitable ground conditions utilizing roof shields of partial circular cross section, and a few other tunnels have been constructed utilizing elliptical, horseshoe, and rectangular shields. The application of shields in tunnel construction, however, has normally utilized a circular cross section because of the tendency for shields to rotate during driving, as well as because of the inherent strength characteristic possible in the circular tunnel section in poor to fair ground conditions and interchangeability of the liner sections.

With a shield operation in running and soft non-consistent ground, the mucking operation involves primarily hand labor with the miners loading the muck on to short conveyors, discharging into narrow gauge cars of one to two yard capacity for

haulage to the shaft. There has been relatively little application of continuous conveyor haulage from the face to the shaft, probably because of the lack of flexibility in this technique as the tunnel is extended. Also, in most instances, the low rate of material excavation does not warrant the high conveyor capacities.

Tunnel Lining. For many years, the time honored primary lining for tunnels consisted of various systems of timbering. The advent of cast iron and steel liner plates has materially reduced the application of timber as a primary lining. Cast iron liner plates have been preferred for many large shield-driven tunnels since their monolithic form allows relatively easy erection, with adequate strength to withstand jacking pressure. They also present a suitable degree of watertight condition after completion of caulking procedures. Tunnel projects presently underway on this continent in connection with subway construction will have cast iron liner plates serving as the primary lining without secondary lining away from the station sections.

Steel ribs and liner plates, or in smaller tunnels, liner plates alone, have been widely used in this country as primary lining of tunnels in earth. In general, the steel lining has been accompanied by a secondary lining of reinforced concrete with the concrete lining serving as the main structural element. In one instance of note, this concept has been reversed, that being the Callahan Tunnel in Boston. This bore, having an outside diameter of 30 feet, eight inches, is lined with steel ribs and plates which are strengthened to serve as the primary structure with a thin concrete lining placed to protect the steel and to present a suitable finish. When used in conjunction with a shield, the steel lining is designed to resist the shove of the jacks.

Some use has been made of special precast concrete units as a primary lining. This method has been utilized on several projects in Great Britain and the United States as a less expensive alternate to either steel or cast iron. Where used with a shield, the units must be designed to withstand jacking forces. A secondary lining is necessary for the prevention of seepage in wet soils.

An erector arm is often included in the operating equipment on the shield or on the construction jumbo immediately to the rear of the shield to enable easy installation of the primary lining.

Tunneling Without Shield. In firm to self-supporting ground where the shield protection is not required, steel ribs, with or without steel liner plates or timber lagging is generally accepted as a primary lining for tunnels over 10 feet in diameter. In non-uniform soils, the tunnel is advanced by hand mining, excavating segments of the full section beginning at the top of the heading and as the excavation progresses around the section, installing the ribs and plates in the roof, sides and invert. Various other techniques to advance the heading can be utilized with equally satisfactory results.

Mechanical Excavators. Much progress is currently being made in the development and use of mechanical excavators. These devices are not new in concept, but the desire to reduce costs of tunneling has renewed interest in their application. They are primarily of value in firm and self-supporting ground of uniformly good characteristics.

As with shields, mechanical excavators are normally designed for the specific project requirements. One of these excavators employs the principle of a wheel with the rim being the diameter of the bore. The spokes are lined with cutting teeth spaced for full or partial coverage of the face, as dictated by the material. Rotation of the cutting wheel is powered by electric motors, geared down to provide the required torque.

Forward movement and direction of these devices is brought about and regulated by hydraulic power, either through jacks, when operating in conjunction with a shield, or by rams braced on the sides of the completed bore.

These devices are currently producing relatively spectacular progress with reduction in manpower requirements on projects where the material characteristics and uniformity allow this method. However, at the present time, the use of

the mechanical excavator, even considering the appurtenant provision of a shield, might be considered inappropriate in running to soft grounds. Support of the tunnel face in these materials could cause interference with the mechanized method which would eliminate the economies offered thereby. One technique, which combines the use of compressed air, the shield and the mechanized excavator, is under development at this time for use in these poorer soils and is discussed later in the report.

Plenum Method. Certain ground conditions occur where the use of compressed air without shield to retain running or soft grounds is appropriate, using hand mining methods. This is called the plenum method. It reduces to a minimum the potential displacement in ground at the surface which might occur by use of the shield.

Water Control

The primary method of control of water in wet tunnels in earth involves the use of compressed air in the heading. This requirement exerts a major influence on the cost of construction because of its adverse effect on the rate of excavation, movement into and out of the heading and the supplemental equipment and work force requirements and associated items. On the other hand, no more feasible method has been presented for dealing with the complex problems introduced when adverse water conditions occur and cannot be avoided.

Its limitations involve the maximum pressure practically possible considering the danger of "blow" in situations of minimum cover, the requirements by laws and the unions relating to the maximum work periods under various air pressures and the maximum differential pressures which can be accommodated in the tunnel section under consideration. These last two limitations are discussed more in detail in the following section on "Design Features".

The use of compressed air requires that an airtight bulkhead be placed in the heading, usually constructed of concrete of sufficient thickness to resist the design air pressure. Under rather heavy air requirements, this thickness could be required to be up to one-half of

the tunnel diameter. Installed in this bulkhead are the material, man and emergency air locks and the piping required for the operation within the heading.

Air compressor equipment is usually installed at the surface attended by a supplementary crew on a continuous basis until completion of that portion of the tunnel requiring compressed air.

In sand and gravel materials, adequate dewatering can sometimes be achieved by use of wellpoints driven into the strata ahead and to the sides of the heading, without recourse to compressed air throughout the tunnel. In deep tunnels, these would be driven from the tunnel and, in instances of shallow cover, could be driven from the surface.

Design Features

The size and shape of a tunnel in earth are considered the two principal design features having effect on the method and cost of excavating the bore. Conversely, these features are governed to a considerable extent by the materials through which these tunnels will pass.

For the purposes of this report, and as a practical approach to this discussion of design factors, a minimum size of tunnel bore can be established at this time to be in the 10 to 12 feet diameter range for the following reasons:

1. For mental comfort of personnel using a tunnel under conditions of stress, the headroom should not be less than eight feet. This dimension is also used in the design of conventional footway tunnels.
2. This headroom dimension may be considered optimum for service, connecting and access tunnels, thus providing some degree of standardization.
3. A tunnel of this size is considered to be the maximum (under average soft ground conditions) that may be constructed using liner plates without steel ribs. Thus, this standard size is an economic factor.

4. A leading tunnel and mine equipment manufacturer has stated that a minimum tunnel shield diameter should be in the range of 10 feet in order to provide adequate space requirements for the shield structure and accessory equipment and work area for the miners.

The maximum size of tunnel excavation in soft and running ground is limited to a substantial degree by the difference in pressure head exerted on the tunnel face between invert and crown. If sufficient air pressure is imposed in the tunnel to maintain dry, stable conditions at the invert, there will be a maximum of excess air pressure at the top. Under minimum cover conditions, or in "open" ground, the use of heavy air pressures could invite the danger of a "blow". A decrease in air pressure to lessen this possibility would introduce wet and running conditions in the invert to the detriment of the excavation and construction operation.

It is considered that at an air pressure of about 25 psi, a miner can work safely for a period of three hours, followed by a one to three-hour normal air rest period, then another three-hour work period. In other words, he may work safely only six hours in every 24 hours. This is considered the break point for reasonably efficient work periods and points up the limitations imposed by man's inability to work for practical and efficient time periods at pressures exceeding 25 psi. Work periods of less than three hours are considered neither practical nor economical. Work rules under compressed air are, in most instances, also fixed by union agreements or governmental law.

It is also found that the required investment in a shield increases at a greater rate than the increase in diameter. For example, the doubling of a tunnel diameter in the 16-foot range would more than quadruple the shield cost, imposing economic considerations in the matter of maximum tunnel size. This matter disregards the obvious problem of practical design and construction requirements for the shield of extremely large diameter.

It is principally for these reasons that few soft ground bores exceed diameters of from 30 to 35 feet. Where the larger cross sectional areas are required, it is more practical and economical to drive multiple parallel tunnel bores of smaller diameters and enlarge them to the required section by excavating between them.

Under apparently dry conditions in firm and self-supporting ground, the principal limitations on size and shape would be the capabilities of bridge action of the earth above the roof and the location of the tunnel with respect to the water table. When such near-ideal conditions are encountered, it is expected that conventional earth tunneling, heading and bench, top heading or side drift methods could be used to attain whatever design size and shape are required within limitation of economics. However, an important design feature with respect to shape of the section relates to the excavation problems and additional cost resulting from the load imposed on the straight rib legs of large arch tunnels by squeezing or swelling ground. In the materials exhibiting these qualities, it is generally more economical to provide a circular section, placing the ribs and supplemental supports in the most appropriate position to resist the uniformly imposed pressures.

In summary, earth tunnel design features which may affect economy and method of excavation are principally concerned with shape and dimensions in the following respects:

1. A minimum standard headroom dimension should be eight feet, thus establishing a minimum bore of 10 to 12 feet.
2. This minimum dimension is a practical and economical standard for most utility and access tunnels.
3. In soft or running earth, bores by shields of circular cross section and with compressed air are generally restricted to the range of 10 to 35 foot diameters.
4. In self-supporting dry ground, larger arch type tunnels are feasible, dependent on the favorable conditions relating to the bridge action period of the earth over the tunnel, the location of the ground water table and the magnitude of pressures from swelling ground.

Some of these factors will be applied or demonstrated in appropriate sections relating to specific tunnel excavations studies.

TUNNELS IN ROCK

Geological Conditions

The geological aspects of tunneling in rock can be best described by translating materials into terms relating to the geological structural formations, e.g. intact rock, stratified rock, etc. Ground conditions along possible tunnel alignments are ascertained by geologists and engineers in the field either by surficial examination or by subsurface exploratory drilling or by a combination of both. These conditions may vary widely along a tunnel alignment and the degree of detail to which geologic exploration is carried will greatly influence the method of tunneling and the reliability of a tunnel cost estimate. Therefore, cost estimates of tunnel construction should be based upon conservative assumptions with regard to subsurface conditions where information on these conditions has not been determined in detail.

In order to provide a standard for compiling basic field geologic data along possible tunnel routes, an attempt has been made by others to classify the various types of ground with regard to the relative ease or difficulty of tunneling operations therein. Following are presented descriptions of the various classifications of ground conditions:

Intact Rock, Intact rock contains neither joints nor hairline cracks. Consequently, when breaking, it breaks across sound rock and breakage is not influenced by joint and fracture patterns. Rock types that might fall into this group are quartz, diorite, gabbro, granite and quartzite.

Stratified or Schistose Rock, Stratified or schistose rock consists of individual strata with little or no resistance to parting along boundaries between strata. Strata may or may not be weakened by transverse joints. However, if transverse joints and fractures are spaced so closely as to destroy bridging action of the strata, rock is classified as very blocky and seamy, or moderately blocky and seamy. Distance between stratifications is generally less than five feet. Where distance between bedding planes is greater than five feet, the rock is better classified as moderately jointed, moderately blocky and seamy, or very blocky and seamy, depending on spacing of joints and fractures. Rock types that might fall into this group are shale, schist and sandstone.

Massive, Moderately Jointed Rock. Massive, moderately jointed rock contains joints and hairline cracks, but the blocks between the joints are locally grown together or so intimately interlocked that vertical walls do not require lateral support. Rock types that might fall into this group are quartz, diorite, granite and limestone.

Moderately Blocky and Seamy. Moderately blocky and seamy rock consists of chemically intact or almost intact rock fragments that are entirely separated from one another and imperfectly interlocked. In such rock, vertical walls may require support. In moderately blocky and seamy rock, the joints and fractures are so spaced that individual blocks are larger than two feet in diameter. This classification applies to both sedimentary and crystalline rocks. Rock types that might fall into this group are quartz, diorite, sandstone and granite.

Very Blocky and Seamy Rock. Very blocky and seamy rock consists of chemically intact or almost intact rock fragments which are entirely separated from each other and are imperfectly interlocked. In such rock, vertical walls may require some support. Very blocky and seamy rock differs from moderately blocky and seamy rock in that the joints and fractures are so spaced that the intervening blocks are less than two feet in diameter. Rock types that might fall into this group are shale, quartz, diorite, and granite.

Completely Crushed or Unconsolidated Rock. Crushed or unconsolidated rock consists of sand to pebble-sized particles that are chemically intact and are very loosely consolidated or unconsolidated. Fault gouge is sometimes present. Typical examples are terrace deposits, quartz diorite in fault zone, arkose.

Wet Competent Rock. Wet competent rock includes those rock types ranging from intact through very blocky and seamy under a saturated condition. Water inflows into the tunnel come from joints and fractures separating the individual blocks. Estimated inflows of 100 gpm or more from the heading must be anticipated before the ground is classified as wet competent. Typical examples might include sandstone, limestone, fault zones.

Wet Crushed or Unconsolidated Rock. The term "wet" is applied to this classification when the material is saturated. Inflows into the tunnel come from interstices between the individual particles. Estimated inflows of 100 gpm or more must be anticipated before the ground is classified as wet crushed or unconsolidated.

Method of Attack

The history of tunnel driving in rock, as with earth tunneling, predates recorded history. It was early in the 19th Century, however, that major rock tunnels were initiated to comply with the demand for rail connections between countries up until then divided by mountain barriers. The later invention of the compressed air drill and dynamite provided the basic tools necessary to refine the art of rock tunneling.

Most of the contemporary hard rock tunnels are driven using basically the same methods, the only changes having been improvements in the technique, material and equipment, such as the development of the drill jumbo, tungsten carbide drill bits and mechanical mucking devices.

The continuous boring method is a relatively modern technique applying to rock tunneling, although it was attempted as early as 1880 in an effort to drive a tunnel under the English Channel. Rock augers have been employed for some time in the mining industry to excavate soft rock such as coal, gypsum and salt. An example of this type of equipment is the Continuous Borer manufactured by the Goodman Manufacturing Company. More recently, James S. Robbins and Associates, Inc. has developed the mechanical "Mole", a practical device for boring tunnels in rock of lesser hardnesses.

As in the case of earth tunnels, the excavation can be initiated by means of either a shaft, inclined drift or a horizontal drift in a side hill location. The method selected depends on the functional requirements, site conditions and economics. While the method of driving the main tunnel would generally not be affected, the type of access will normally have considerable influence on the method of mucking.

Drilling and Blasting. All tunnels advanced by drilling and blasting follow a standard excavating cycle, drilling, blasting,

ventilating and mucking. The digging and loading operation can be accomplished by up-and-over loaders, conveyor loaders or power shovels. Rail cars, trucks, motorized buggies, or bulk conveyors may be utilized to remove the muck. When the bridge-action of the rock is short, it may be necessary to install the roof supports before proceeding with the mucking.

The full face method of advance, removing the entire cross sectional area of the tunnel in one cycle, is currently the most popular, being used in favorable rock conditions where the tunnel size is within the practical size of a drilling jumbo.

When the rock characteristics will not permit a full face operation, or in the case of very large tunnels, the method of top heading and benching is frequently employed. The top heading can be either driven full face or by means of a pilot tunnel that is subsequently widened to full width. After the top heading is completed or has advanced sufficiently, the bench is removed in one or more lifts, usually by drilling vertical holes with air-track drills. The bench can also be removed by drilling horizontal holes in the face parallel to the axis of the tunnel.

At times, a bottom heading is employed in lieu of a top heading and widened either by ring or long hole drilling. A variation of the bottom heading method employs the use of raises from a pilot tunnel to reach subdrifts along the top of the tunnel. This procedure increases the number of working faces with the pilot tunnel being used for mucking, permitting the rotation of cycle operations and crews.

Occasionally, under poor rock conditions, it may be necessary to employ the side drift method whereby bottom drifts are driven along both sides of the tunnel enabling the initial erection of the side supports. The top heading is then driven and the arch supports erected before the center portion of the tunnel is removed.

The pioneer tunnel method has been employed in certain instances to increase the number of working faces. This technique involves the driving of a small tunnel parallel to the axis of the main tunnel, from which side drifts are extended to the main

tunnel excavation area. The main tunnel can then be driven in both directions from each connection and the material mucked out through the pioneer tunnel. This technique provides the advantage of revealing any major geological changes or problem areas prior to driving the main heading. A pilot tunnel may also be required as a conduit to remove water. Problem areas can be grouted from the small heading, sealing them in advance of the main tunnel.

In actual practice, a variation of one of these methods or a combination of methods may be used, and the method of attack can vary as work progresses. Most often the method employed and the equipment used is predicated on the personal experience and ingenuity of the individual contractor as well as on the type of material encountered.

Mechanical Excavators. This method of tunneling utilizes the principle that rock will fracture when subjected to pressure and torque. The force is applied by rolling discs mounted in a rotating cutting head hydraulically jacked against the tunnel face. In the softer rocks, fixed drag-type cutters are often added to the cutter head to kerf the face between the rolling discs. These circumferential cuts expedite the fracturing of the rock, but the drag tools have been unable to withstand the impact encountered in cutting the harder stone. As the rock spalls from the face, it is scooped up by buckets mounted around the periphery of the rotating head and deposited on a conveyor for loading the haulage units.

Some of the advantages attributed to the continuous boring method as compared to the more conventional method of drilling and blasting are: safety due to the round, smooth, unshattered bore; no overbreak; a reduction in the size of crew required; the ability to advance at a faster rate; a more uniform broken rock size, facilitating the handling and disposal of the muck; and the elimination of blast damage to the tunnel or to property in congested areas.

There are two major limitations associated with this method of tunneling, the first being the inability to drill through the harder rocks, and the second being the high capital cost of the machine.

The first Robbins' "Mole" was used to bore the 25.75 foot diameter diversion tunnels for Oahe Dam in South Dakota through soft shale. Since then a total of eleven "Moles" have been built or are presently under construction.

The first attempt to bore harder rock was for a 9 foot tunnel in Chicago through limestone having a compressive strength of 18,000 to 24,000 psi, without success. Subsequently, a modified machine successfully cut a sewer tunnel in Toronto of 11 feet diameter through hard crystalline limestone, sandstone and shale having a compressive strength of from 8000 to 27,000 psi. In the fall of 1962, another "Mole" approximately 16 feet in diameter completed the final 8000 feet of tunnel at an average rate of 415 feet per week through a hard mudstone in Tasmania having a compressive strength of about 16,000 psi. The largest "Mole" to date is almost 37 feet in diameter, and is presently being readied for shipment to Pakistan for use in tunneling through soft sandstone and clay, with hard sandstone.

The "Moles" have all been reported to have had an average cutting rate of about 8 to 12 feet per hour and an average advance rate of approximately 25 to 35 feet per shift.

Through experience and mechanical improvement, it is possible that machines of increased ability will be developed to remove rocks of greater hardness. The cost might also be reduced if the equipment could be adapted to more than one job, thus increasing spread of amortization.

Forepoling. When a zone of thoroughly crushed or decomposed rock, or soft sediments, is encountered, it is necessary to revert to soft ground tunneling methods. If the arch stand-up time is so short as to normally require a shield, but a shield cannot be economically justified for the length of tunnel involved, the method of forepoling and breasting must be employed to support the roof and the working face. Forepoling involves driving timber or steel spikes ahead of the face. These act as cantilevers and bear the weight of the ground until their forward ends can be supported. Forepoling is a slow, expensive process, and consequently any area requiring this method should be avoided, if possible.

Water Control

Perhaps the one fact that has caused tunnelers the most difficulty has been the inflow of water, being probably the greatest single cause for delay and additional expense. The water can range in temperature from cold to scalding hot with quantities ranging from a continuous dripping to a devastating torrent. Two examples of non-subaqueous tunnels that experienced heavy inflows were the Mont Blanc in the Alps (19,000 gpm, which stabilized at 11,000 gpm after a few days) and the Tanna in Japan (55,000 gpm). The problem is compounded when the inflow includes mud, sand, or crushed rock.

Tunnel sites should be avoided, if possible, where preliminary investigations indicate the presence of aquifers or perched water. Experience has shown, however, that serious water problems are likely to be unpredictable.

Subsurface water is likely to be found in sedimentary rocks, such as sandstone and limestone, and in broken or fissured igneous rocks, such as lava. Occasionally, intrusive igneous rocks, such as granite and crystalline metamorphic rocks are aquifers. Faults, water-bearing sand or gravel seams, or mud lenses may contain considerable amounts of water.

The simplest procedure for handling a reasonable amount of water, usable in a side hill drift sloping toward the entrance, involves the provision of ditches along the invert, discharging away from the entrance. If the tunnel is initiated from a shaft or a downward sloping drift, all of the water must be pumped, materially increasing the cost and adding to the danger of flooding. Where a sizable quantity of water must be removed, a separate tunnel is often driven to handle the flow.

When an inflow of considerable magnitude is encountered, some method of alleviating the flow will be required before tunneling can proceed. The most common remedial action is to seal the problem area in advance of the heading. This is ordinarily accomplished by injecting a portland cement grout under pressure. Sawdust, sand, or other admixtures may be added to the grout in order to plug large cracks. This method is satisfactory if the grout is not overly diluted or carried away by flowing water before it has time to set up. When the latter conditions are encountered, recent innovations in chemical grouting might be applicable. These grouts have the ability to set up

rapidly in a predetermined time, and the manufacturer claims that they are capable of solidifying even in running water.

The use of compressed air to control water is as applicable in rock as in soft ground. The problem areas are usually more localized, however, and so the expense of furnishing the bulkhead, air locks, and compressor plant would not be warranted except as a last resort.

Other less common methods of controlling subsurface water can be applied under special circumstances, such as dewatering by means of wellpoints or deep wells and freezing.

Design Features

The size and shape of a tunnel are dependent upon geological conditions as well as on the space requirements. A circular section may be required in swelling or squeezing rock or any rock exerting considerable side pressure and is also used when the bottom is unable to support the roof loads on foot blocks. A horseshoe shape is the most commonly used section in competent rock, providing a close approximation to the circular section with the advantage of having a plane floor. In flat bedded rocks, where the spacing of the transverse joints is greater than the width of the tunnel, it is possible to provide a rectangular cross section if the bending stresses in the slab bridging the tunnel are smaller than the tensile strength of the rock. Spans greater than approximately 50 feet require exceptionally sound and intact rock.

The amount of additional support required is dependent upon the magnitude of the static and dynamic loads. Static loading is a function of depth, up to the limit of the natural bridging action of the rock, and of the residual stresses within the rock itself. These stresses, normally in a state of equilibrium, are changed by the tunneling operation; consequently, the rock has a tendency to pop into the excavation area, often with explosive force.

A quantitative estimate of the amount of support required in a proposed tunnel, even with the best geological information and advice available, involves a certain amount of approximation. The uncertainties are reflected in the large safety factors required in all phases of design. In tunneling it is preferable to be on the conservative side rather than to be inadequately prepared if an emergency arises. The unexpected is more the rule than the exception.

In some exceedingly sound rocks under favorable conditions, no added support may be required to maintain the tunnel, even though a lining may be necessary or desirable from the standpoint of functional use. Frequently in sound rock a stable condition can be attained by means of rock bolts, or rock bolts and wire mesh. Where the rock is susceptible to weathering and deterioration if left exposed, gunite may provide the required protection.

Generally, however, most rock tunnels will require the installation of a liner of some type for part, if not all, of their length. Probably the most frequently used liner utilizes steel ribs, with or without lagging, for the primary support with concrete added to form the permanent lining. The design and extent of the steel ribs varies depending upon conditions and the method used to drive the tunnel. The simplest case occurs when only the roof requires some additional support. Arched ribs are placed under the roof and supported on wall plates set into recesses in the wall at the spring line. If the roof load cannot be carried by the rock walls themselves, it may be necessary to transfer the load to the tunnel floor by either continuing the ribs or supporting them on posts. Where mild side pressure is encountered, invert struts are added, and the side posts may be curved outward instead of being vertical. Invert struts are also used to prevent the bottom from heaving. When a circular tunnel section is necessary due to conditions previously discussed, a full circle type rib is used.

It is not unusual for the primary liner to vary in design along the length of a tunnel due to the non-uniformity of conditions. The rib spacing will vary also to meet local conditions. The ribs are usually designed to carry the anticipated load when spaced three to four feet apart, but may be placed as close as nine to twelve inches to withstand the actual loads encountered.

The design strength of the primary liner is also predicated on if and when a permanent concrete liner is added. Frequently the maximum loading on the lining is attained gradually due to the bridge-action period of the rock. Thus, if a permanent concrete liner is installed initially during construction, the primary liner need only support the interim loading. If stand-up time is sufficiently long to allow the concrete liner to be placed and cured before being subjected to loading, it might be possible to dispense with the primary liner entirely.

Every tunnel project is likely to be unique because of the infinitely varied geological conditions. It is not possible to develop a prototype design that would not subject to revision to meet the actual conditions as revealed by the investigation and construction processes.

STUDIES OF EXCAVATION TECHNIQUES

GENERAL SUBSURFACE PHYSICAL CONDITIONS

The scope of this report covers conditions that might be encountered within the continental limits of the United States. In order to illustrate any of the various excavation techniques, it is first necessary to establish specific subsurface conditions and describe in brief the physical properties of these materials that would have a bearing upon their excavation. In selecting these subsurface conditions, there must be recognized the fact that specific physical properties of the subsoils vary rapidly and radically over short distances. Furthermore, these properties are usually complicated by ground water, the one most important varying factor affecting subsoil excavation and its cost.

To portray even in a general way the various possible types of subsurface conditions, some organizing and generalizing is necessary. One approach to the subject is by Nevin M. Fenneman, who has prepared an elaborate treatise on the physiography of the United States, that being the study of the origin or process of development of the various land forms. Fenneman has divided the United States into 8 major divisions, 25 provinces, 78 sections and 86 different characteristics.

One example of this type of organization is shown in what Fenneman refers to as the Osage Plains section of the Central Lowland Province. The area within the boundaries of this section includes parts of Texas, Missouri, Kansas, and nearly all of Oklahoma. This is an unglaciated plain of low relief, represented by alternate layers of soft and hard rock that dip gently to the west or northwest. The softer formations are of either shale or sandstone. There are many local escarpments and high hills formed by the harder rocks. The accompanying low areas are filled with loose sand, silt and sometimes soft clay.

The Seaboard Section of the New England Province is a small fraction of the size of the Osage section. The local area of Boston, which is in the Seaboard Section, is characterized by its soft bay mud to depths of 200 feet, while a short distance away there are massive rocks protruding above sea level to over 200 feet in elevation.

These two examples are presented to illustrate the wide variations in the physical character of the subsurface materials that can be encountered in a single physiographical section.

A map showing Fenneman's physical divisions of the United States is included in a folder on the back cover of this report.

It is concluded, therefore, that the organization of the United States into physiographic regions is not the key to a selection of a particular excavation technique. However, the information is a very useful tool in analyzing the subsurface conditions. It is very important, therefore, to recognize that the excavation techniques described in this report are for hypothetical boring logs and soil or rock conditions accompanying each technique. There should be no assumption that the techniques could be classified for use in general types of subsurface conditions for various regions of the United States.

In order to portray physical properties of the subsurface conditions, the subsurface data presented herein were selected from published sources or composed to represent varying subsurface conditions. The primary basis for selection was to illustrate a specific subsurface condition which would permit techniques to be described and analyzed. The log of subsurface conditions which describes the soils in a vertical sequence from top to bottom permits a convenient method of portraying the varying conditions related to open cut excavation. However, for tunneling techniques where the excavation advances horizontally across several soil or rock types, it is considered practical to organize the materials into types representative of the various geological structural characteristics.

STUDIES OF OPEN CUT EXCAVATION

General Considerations

The provision of deep shelters in many of our urban areas can be feasibly provided from the standpoint of excavation through the method of cut-and-cover construction - that is, removing earth, constructing the shelter in the excavation, and then backfilling over the completed structure. The structure could be located at less than required depth below existing grade with backfill mounded to provide the desired protection or placed at a depth necessary for protection, providing a finished backfill at existing grade.

The scope of this report does not include the functional planning of the shelter. Some discussion regarding access to the shelter area is appropriate, however, in setting forth additional bases for these studies. This access could be provided by several methods. Ramps from street level of either minimum length considering depth of the shelter and limiting ramp grade, or of extended length providing added protection from the center of populace served would be an appropriate method of access to the shelter facility. These ramps could be constructed by cut-and-cover or tunnel methods or a combination of these techniques. It is quite possible that the methods of excavation in instances could be economized or simplified by the provision of ramps of adequate size and appropriate gradient for excavating and construction equipment and materials, later serving as functional access to the shelter area. Another type of access could be provided by construction of vertical shafts from street level to the shelter enclosing ramps of the required gradient. This would not necessitate excavation outside the site for functional access construction, allowing this provision at minimum cost, but would require the greatest amount of unprotected travel to the shelter. Still another mode of access might apply in areas of concentrated populace, wherein subsurface passageways could lead from basements and sub-basements of large apartments, hotels, or office buildings to the nearby shelter on acceptable gradients.

As will be pointed out in the specific open cut excavation studies, subsurface conditions can be encountered where deep excavations are not feasible, within even generous economic limits, considering merely the excavation phase of the work. These

conditions relate as well to the construction, maintenance and operation costs of the shelter facility. Functional planning of the entire project in these instances must be subject to alternative methods of provision of shelter areas. A few possible alternatives are discussed later in the report. It is obvious that the costs of shelter areas under these more stringent conditions will be amplified many times over the costs to be derived under the median conditions studied herein as being generally appropriate for open cut excavation.

Selection of Specific Profiles

The study profiles presented herein were composed to illustrate conditions that might be encountered in urban areas. Consideration was also given to profiles that would require dewatering systems of varying types and at various depths. Varying rock conditions are also shown to illustrate the method of treating strong competent rocks that may be cut vertically as well as the weaker formations that would require slopes and benching to prevent excessive spalling. Under some conditions, sheeting and bracing are required to retain the slopes. With the various combinations of conditions, it will be possible to study the change in cost as a function of either a change in material or a change in water condition.

Establishing Open Cut Excavation Dimensions

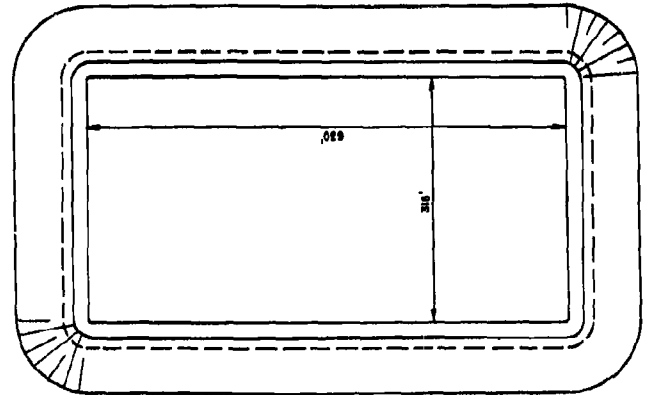
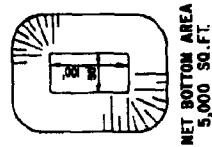
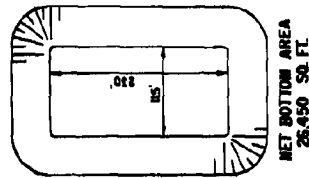
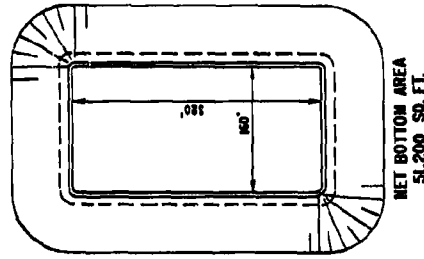
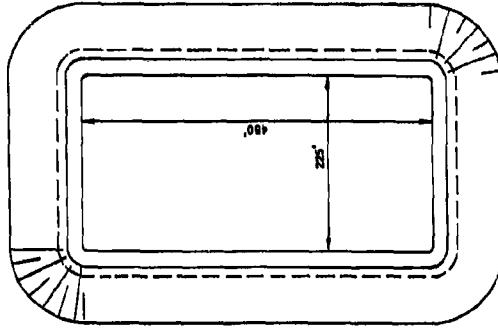
The bottom areas of excavation studied herein range from 5000 square feet to 200,000 square feet. This limit was subdivided into these minimum and maximum with three additional areas of 26,450 square feet, 51,200 square feet, and 101,250 square feet to provide a workable basis for the determination of variation in excavation cost. Exhibit 3 illustrates these study areas. The range of depths of excavation studied extends from minimum practical cover to 150 feet in depth; likewise, this range was subdivided into depths of 25, 50, 100 and 150 feet for cost analysis purpose.

The studies of excavation at the specific locations which follow are based on these graduated sizes of bottom areas and depths.

The shape of the excavation, as discussed before under "Considerations Affecting Excavation Methods", relate to the costs and

DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS
OPEN CUT EXCAVATION
STUDY RANGE AREAS
PLANS

DE LEW, CATHER & COMPANY - CONSULTING ENGINEERS - CHICAGO



NOTE: SLOPE CONDITIONS SHOWN
ARE FOR LOCATION A

function of the project. For the purpose of this report, it was determined that a rectangular excavation would be the basis of these studies, with length equal to twice the width. In some of our cities, this shape, in the larger sizes, would correspond to a city block.

It will be noted that the excavated areas in these studies are developed utilizing side slopes in appropriate materials. Retainage devices, such as sheeting or cofferdams, are provided only where necessary to retain the soils of unstable characteristics. In specific instances, these devices might necessarily be included in the project because of property limitations or shoring requirements.

General Cost Considerations

The specific study examples which follow detail the conditions relating to materials encountered, conditions imposed, techniques used and costs resulting therefrom. Certain general cost considerations are set forth here as governing all of the studies of open cut excavation.

1. Excavated material would be loaded in trucks for disposal. Haulage or disposal costs away from the departing point at the top of excavation or haulage ramp are not included. This stipulation should be modified in actual instances where temporary disposal of much of the excavated material would be made at the site for backfill over the shelter. This consideration would bring into focus other possible methods of moving the excavated earth on the larger sites.
2. Benefits of the sale of excess earth or rock are not included.
3. Ramp excavation costs are included; backfill of the ramps is not included.
4. Cost of backfill is not included.
5. Costs of underpinning or shoring buildings are not included.

6. Pumping Provisions and costs as required for water control are included only for the excavation phase of the work. These costs would probably continue through the construction phase.
7. Bottom trimming of excavated areas and side trimming (in rock) which would be required prior to concrete placement is not included.
8. Sheeting or similar retainage device, where included in these studies, is not subjected to surcharge.

These studies result in the presentation of graphs which, for the particular conditions developed for each study, relate the cost factor per square foot of effective (net bottom) area to the depth of excavation. These graphs are shown on Exhibits 6, 9, 12, 15, 18, 21, 24, 27 and 30. The cost factors so indicated equal dollars, U.S. average for that item, based on the Engineering News-Record 20-City Construction Cost Index of September, 1962. Conversion of these factors to dollars related to the construction area may be made with reference to the ENR indices of that date, with due caution exercised regarding the limitations imposed by these studies and basic assumptions, discussed herein.

Discussion relating to the specific examples studied herein, together with accompanying logs, data and cost factor curves, follows.

STUDY OF OPEN CUT EXCAVATION

LOCATION A

The conditions assumed for this location would be typical for the Chicago area, excepting for a variation in the location, depth and thickness of the strata indicated. Exhibit 4 illustrates the subsurface conditions and Exhibit 5 shows slope conditions for the various depths studied.

Techniques and Equipment Utilized - Excavation at this location down to the top of the rock presents no particular problems and could be readily performed with a two cubic yard shovel loading directly into trucks. Excavation in the hardpan overlying the rock would proceed at a slower rate than in the sand and clay above. A ripper would be used to loosen the hardpan and thus expedite the excavation.

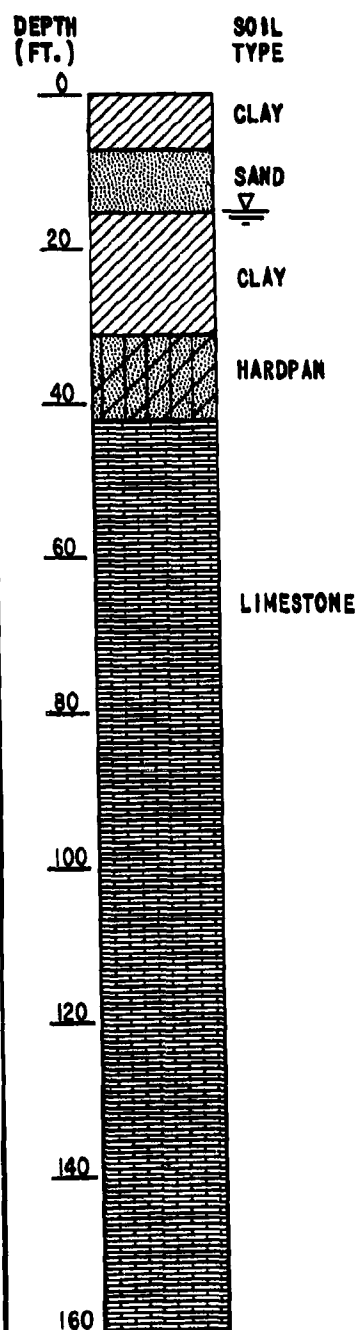
It is assumed that the rock excavation would require one pound of dynamite per cubic yard of excavated material. After the rock is drilled and shot, the material would be loaded into trucks for disposal.

Where excavation is required to a depth of 50 feet only, the rock excavation would be 8 feet. In these cases, the excavated rock would be loaded directly into trucks with a two cubic yard shovel.

Where it is necessary to carry the excavation to the 100 feet or 150 feet depths, the excavated rock would be loaded by means of a tractor shovel or similar equipment into "Dumptors" or other relatively small shuttle-type dump trucks for transportation to the hoisting equipment. Rock would be hoisted in skips and loaded into trucks at elevation -42 (top of rock).

Control of water in the excavation area would be by open pumping utilizing sumps and ditches.

Cost factors per square foot of effective (net bottom) area at Location A are shown on Exhibit 6.



GENERAL SUBSURFACE CONDITIONS

Log A has been composed from test boring data available in the vicinity of Chicago, Illinois. Physiographically, Chicago is located in the Eastern Lake section of the Central Lowland province. The surface of the bedrock, which is limestone, varies from local outcroppings to depths of about 160 feet. The materials above bedrock are the result of glacial action.

The material to a depth of 7 feet is a stiff to hard desiccated clay. Between 7 and 15 feet is a very fine uniform sand. From 15 to 31 feet is a medium stiff compressible clay; from 31 to 42 feet is hardpan. Hardpan in this area is a very dense clayey silt with a trace of fine sand.

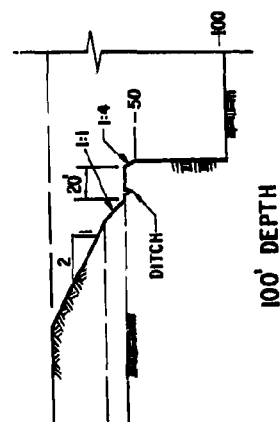
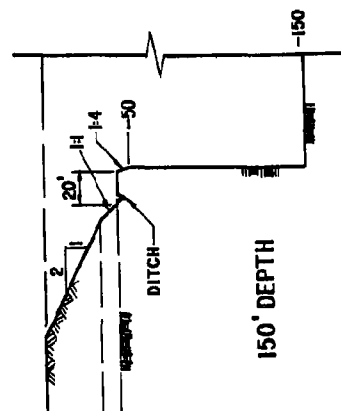
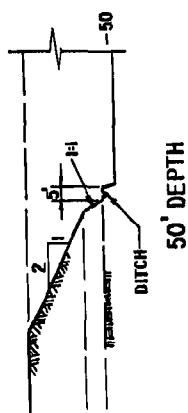
The water table is located within the sand strata and will vary slightly with the seasons of the year. The water table is assumed to be at a depth of 15 feet. In addition to the surface water, flowing water is also encountered in the upper 8 $\frac{1}{2}$ feet of the limestone, which is weathered and fractured.

DATA PERTAINING TO EXCAVATION

Excavation for all materials above rock can be accomplished with conventional earth moving equipment. For the materials down to the hardpan a slope of 2:1 is suggested. However, the hardpan may be excavated on a 1:1 slope. The upper 8 feet of the limestone which is weathered and fractured, should be excavated on a 1/4 to 1 slope with at least a 5 foot bench. The bench in this case is to allow some spalling at the upper surface of the limestone without impairing the stability of the overburden.

DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS
OPEN CUT EXCAVATION
SUBSURFACE CONDITIONS-LOCATION A

DE LEUW, CATHY & COMPANY · CONSULTING ENGINEERS · CHICAGO

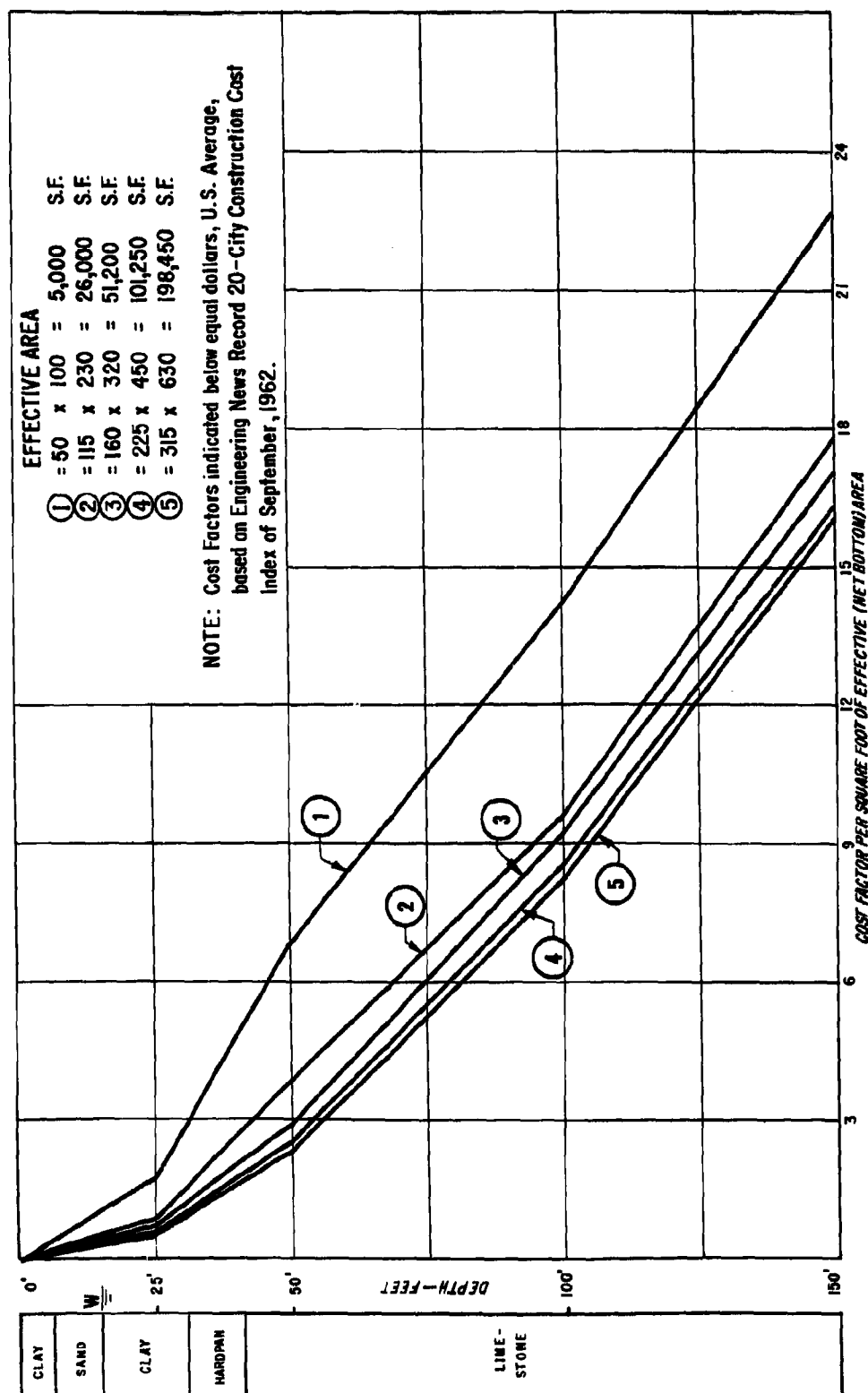


NOTE:

THE RANGE OF NET BOTTOM AREAS
FOR THIS LOCATION ARE AS
SHOWN ON EXHIBIT 3.

DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS
OPEN CUT EXCAVATION
SLOPE CONDITIONS - LOCATION A
PART SECTIONS

DE LEUW, CATHEN & COMPANY - CONSULTING ENGINEERS - CHICAGO



DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS

OPEN CUT EXCAVATION COST FACTOR CURVES - LOCATION A

DE LEUW, CATHIER & COMPANY - CONSULTING ENGINEERS - CHICAGO

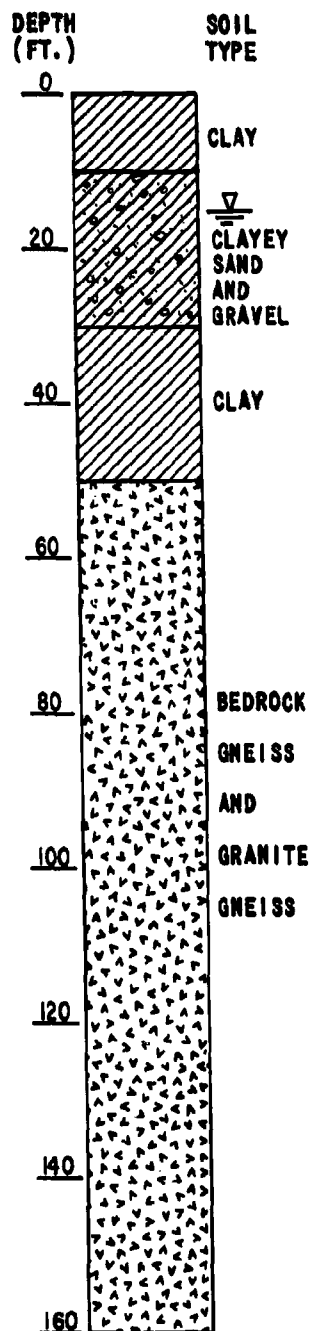
STUDY OF
OPEN CUT EXCAVATION

LOCATION B

The conditions assumed for this location represent the stratification encountered in Washington, D.C. Although the upper 50 feet of overburden could be expected to vary considerably, the existence of bed rock is present throughout the area. Exhibit 7 illustrates the subsurface conditions and Exhibit 8 shows slope conditions for the various depths studied.

Techniques and Equipment Utilized - Excavation at this location would be very similar to Location A, except that hardpan is not encountered. It has been assumed that the rock excavation would require two pounds of dynamite per cubic yard of excavated material.

Cost factors per square foot of effective (net bottom) area at Location B are shown on Exhibit 9.



GENERAL SUBSURFACE CONDITIONS

Log B has been composed from test boring data available in the vicinity of 22nd Street and New York Avenue in Washington, D.C. Physiographically, Washington, D.C., is divided between the Embayed section of the Coastal Plains province and Piedmont Upland section of the Piedmont province. This presents a very complex section geologically and the subsurface conditions can be expected to vary considerably over a short distance.

The top 10 feet consist of a very stiff desiccated clay. Between 10 feet and 30 feet is a clayey sand and gravel. Between 30 feet and 50 feet is a very stiff overconsolidated clay. Bedrock is encountered at a depth of 50 feet, and is a gneiss or granite gneiss. This is a metamorphic rock which has many plate shaped particles and seams of mica. The bending planes and the lines of cleavage are very irregular and can be found to go in practically any direction.

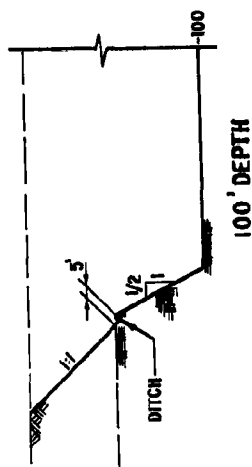
The water table will be found in the clayey sand and gravel and is assumed at a depth of 15 feet.

DATA PERTINENT TO EXCAVATION

The material overlying the rock can be readily excavated with conventional earthmoving machinery. Although the water table is high, it will be found only in the clayey sand and gravel strata and can be easily handled with sump pumps. For temporary excavations, a slope of 1:1 may be used; if the excavation is to be permanent then 2:1 slopes should be used. The underlying rock is decomposed near the surface and a 5 ft. bench should be located at the top of the rock to prevent sloughing of the earth slope in the event of any spalling in the underlying rock. Slopes in the rock may be 1/2:1; however, 8 ft. wide benches should be spaced at vertical intervals not to exceed 50 feet.

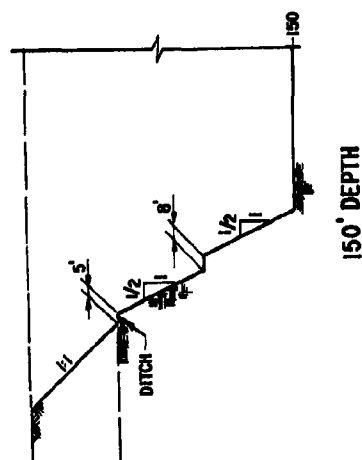
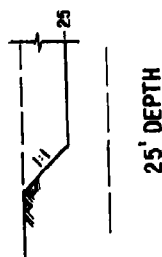
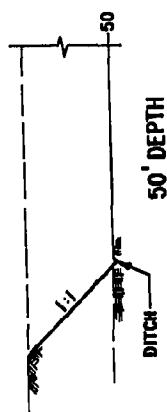
DEEP CIVIL DEFENSE SHELTERS IN URBAN AREAS OPEN CUT EXCAVATION SUBSURFACE CONDITIONS—LOCATION B

DE LEUW, CATHAR & COMPANY · CONSULTING ENGINEERS · CHICAGO

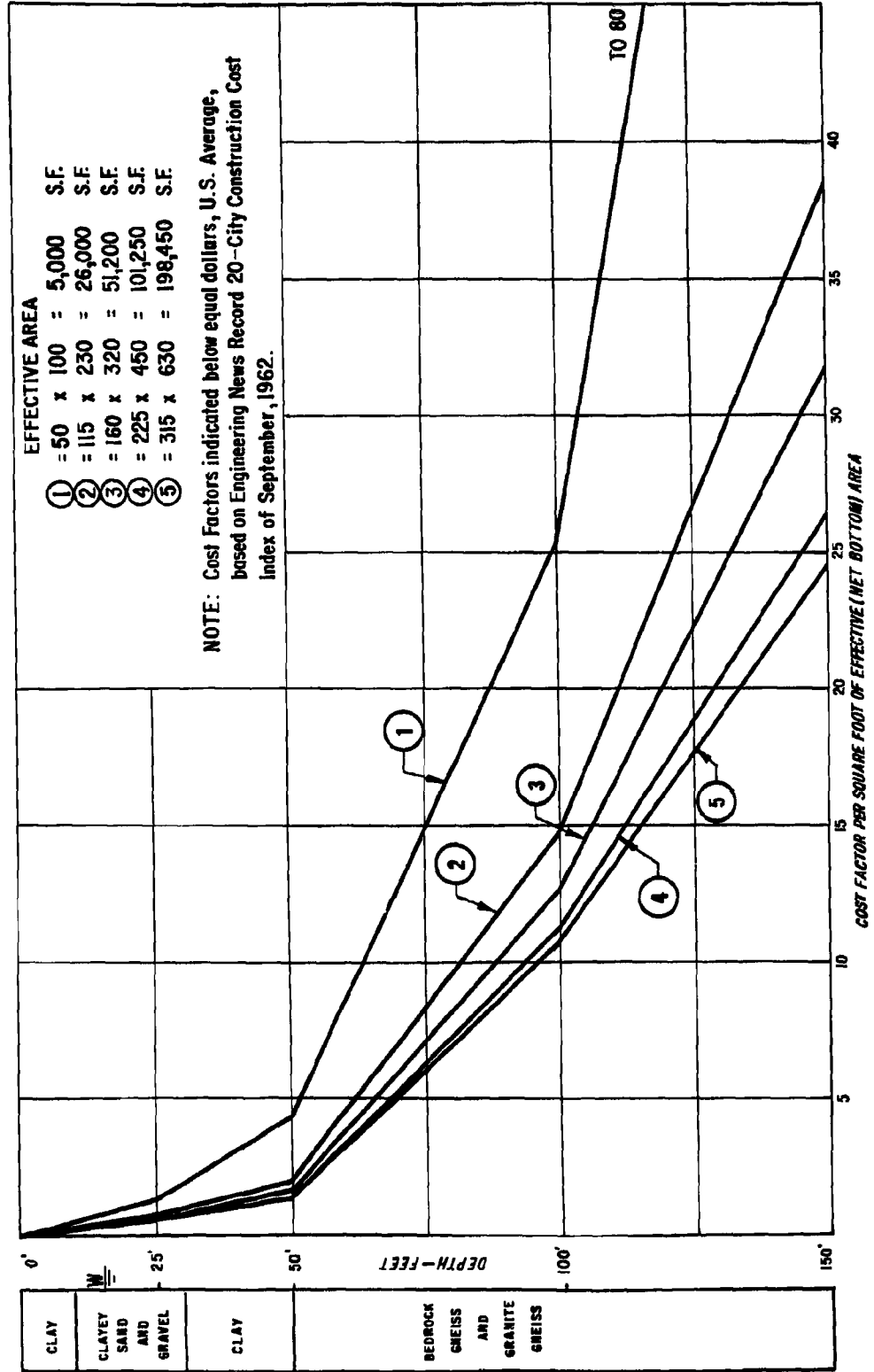


NOTE:

THE RANGE OF NET BOTTOM AREAS
FOR THIS LOCATION ARE AS
SHOWN ON EXHIBIT 3.



DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS
**OPEN CUT EXCAVATION
SLOPE CONDITIONS - LOCATION B**
PART SECTIONS
DE LEUW, CATHER & COMPANY - CONSULTING ENGINEERS - CHICAGO



DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS
OPEN CUT EXCAVATION
COST FACTOR CURVES - LOCATION B
DE LEUW, CATHER & COMPANY - CONSULTING ENGINEERS - CHICAGO

STUDY OF
OPEN CUT EXCAVATION

LOCATION C

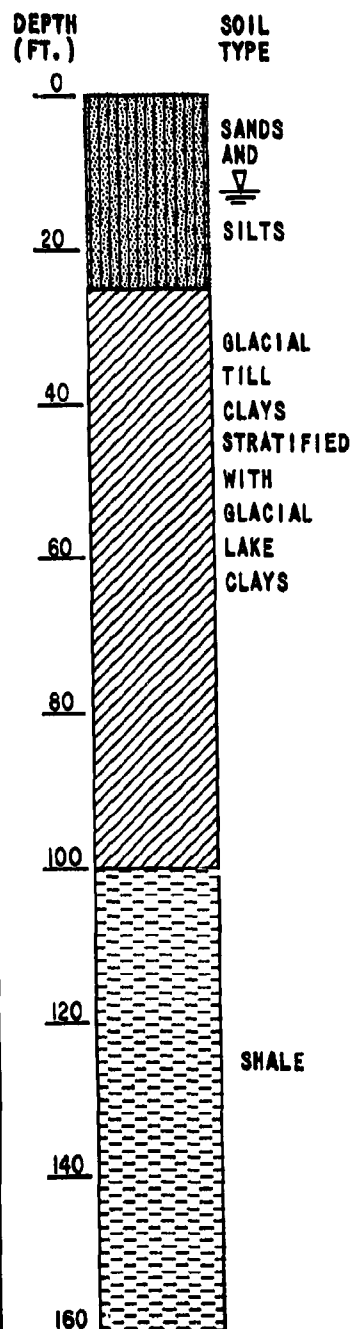
The conditions assumed for this location portray the stratified glacial clays found in the northern part of the United States and together with the presence of shale as the bed rock formation. Limited special dewatering measures in this instance are required to control the flow from aquifers encountered intermittently throughout the excavation area. Exhibit 10 shows the subsurface conditions and Exhibit 11 indicates the slope conditions required.

Techniques and Equipment Utilized - The top 25 feet of material above the clay would be readily excavated with a two cubic yard shovel. Wellpoints would be required for 20 percent of the excavation perimeter. The wellpoint pumping would be supplemented with open pumping, utilizing sumps and ditches.

The clay between elevation -25 and -100 would also be loaded into trucks with a two cubic yard shovel, but the rate of progress would be slower than in the silty sand.

The shale below elevation -100 will require one half pound of dynamite per cubic yard. The excavation method will be the same as described for Location A.

Cost factors per square foot of effective (net bottom) area at Location C are shown on Exhibit 12.



GENERAL SUBSURFACE CONDITIONS

Log C has been composed from test boring data available in the vicinity of Cleveland, Ohio. Physiographically, Cleveland is located in Eastern Lake section of the Central Lowland province. The bedrock in the immediate vicinity is shale. The overlying materials are all of glacial origin.

The materials to a depth of 25 feet consist of stratified sands and silts with moderate cohesion. Almost all the material below the silty sand and above the shale consists of glacial clays and with few exceptions are at least moderately silty. Very soft clays are not encountered. These glacial clays can be further subdivided into glacial till and glacial lake clays.

The bedrock below a depth of 100 feet is the Black Ohio Shale. The material is very heavily compressed and will support considerable loads. However, upon exposure to air, the surface has a tendency to crack and spall with some swelling being evidenced.

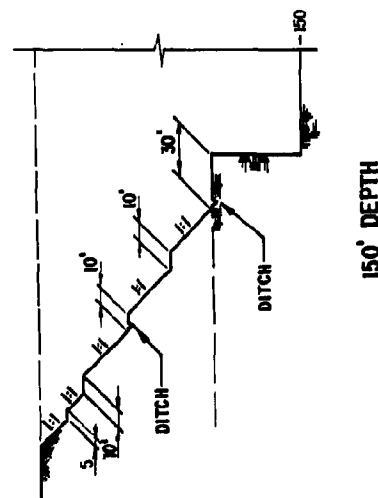
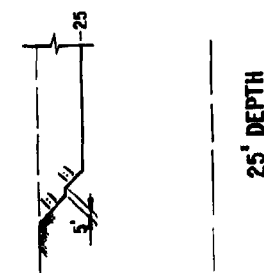
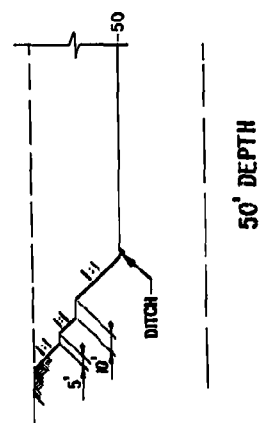
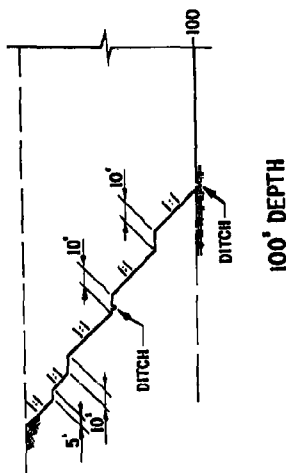
DATA PERTINENT TO EXCAVATION

The sands and silts can be readily excavated with conventional earth moving equipment. Although the water table will vary it is considered to be at a depth of 15 feet for this log. The permeability will be reduced by the silt and large excavations can be handled by sumps. It may be assumed that 20 percent of the strata perimeter is to be equipped with well points and a slope. Although the clays can be readily excavated the glacial lake clays may contain silt lenses and a 1:1 slope with 10 foot berms at 25 foot vertical intervals is suggested to provide stability.

The underlying shale will have to be blasted to facilitate removal. The exposed surfaces will exhibit some spalling and the upper layer may be weak. The slope in the shale may be made vertical. There will be no water in the shale.

DEEP CIVIL DEFENSE SHELTERS IN URBAN AREAS OPEN CUT EXCAVATION SUBSURFACE CONDITIONS-LOCATION C

DE LEUW, CATHEN & COMPANY · CONSULTING ENGINEERS · CHICAGO



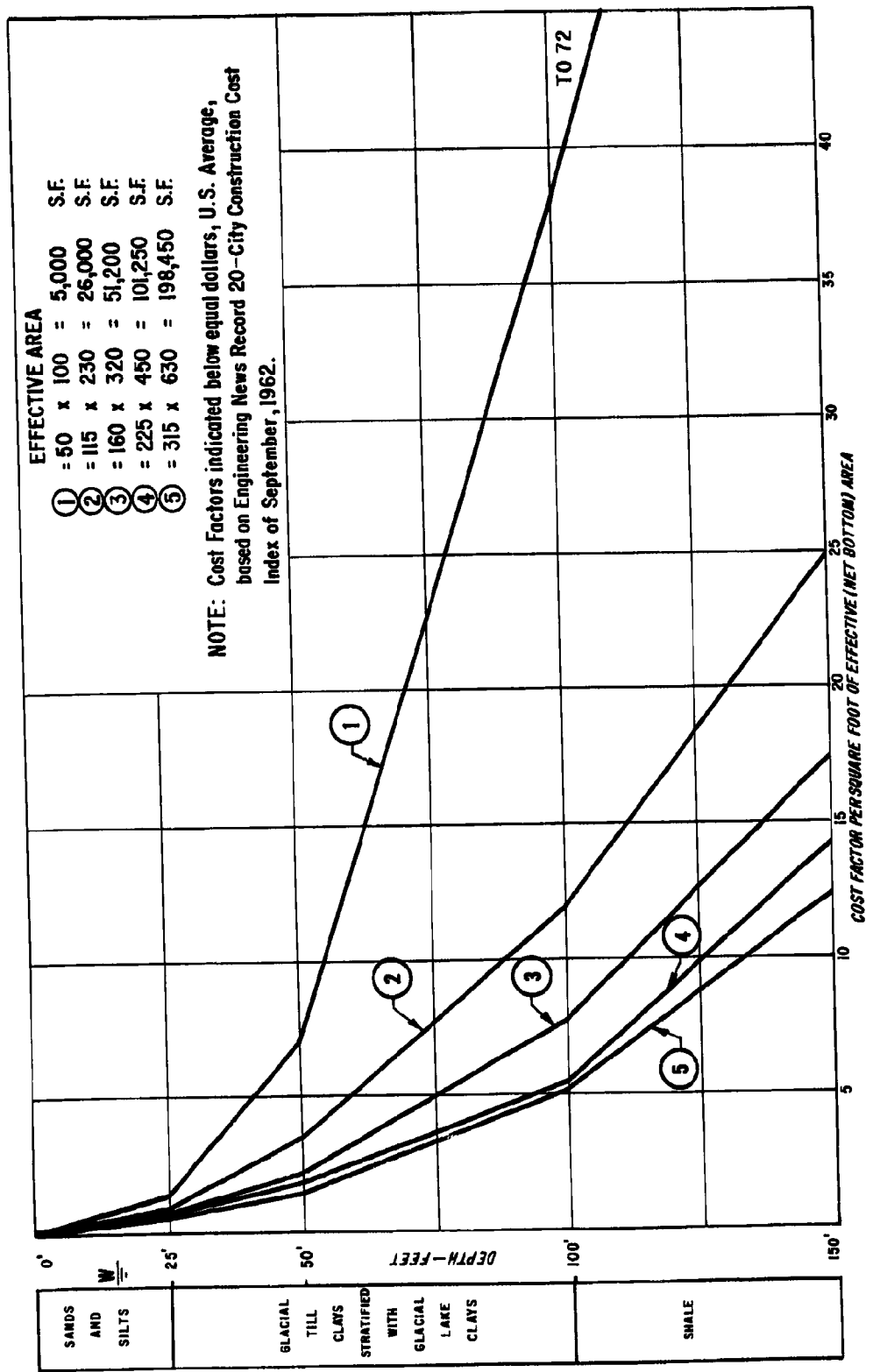
NOTE:

THE RANGE OF NET BOTTOM AREAS
FOR THIS LOCATION ARE AS
SHOWN ON EXHIBIT 3.



DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS
OPEN CUT EXCAVATION
SLOPE CONDITIONS - LOCATION C
PART SECTIONS

DE LEUW, CATHER & COMPANY - CONSULTING ENGINEERS - CHICAGO



DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS
OPEN CUT EXCAVATION
COST FACTOR CURVES - LOCATION C
DE LEUW, CATHIER & COMPANY - CONSULTING ENGINEERS - CHICAGO

STUDY OF OPEN CUT EXCAVATION

LOCATION D

The conditions illustrated in this study present stringent characteristics with respect to porous strata and high ground water conditions. Even with the employment of dewatering devices on a large scale and control measures, such as grouting, excavation in these strata would be limited to minimum depths. Exhibits 13 and 14 show the subsurface and slope conditions assumed for this location.

Techniques and Equipment Utilized - The water condition at this location would be quite serious. It would not be considered feasible to excavate to a depth greater than 25 feet. Steel sheeting, with a wellpoint system outside the steel sheeting, and a second stage wellpoint system within the sheeted area, together with chemical grouting for 30 percent of the perimeter, would be required to control the water.

The material down to the water line would be removed prior to the installation of the first stage wellpoint system so that the wellpoint header line will be placed close to the water level. Because of the shallow depth and the type of material involved, this excavation would be performed with a tractor shovel loading directly into trucks. This method of excavation would easily keep ahead of the driving of the steel sheeting and the installation of the first stage wellpoint system.

After the steel sheeting is driven, the first stage wellpoint system would be installed.

Bracing of the steel sheet piling would be installed as the excavation proceeds. In the case of the 5000 square foot area, the steel bracing would span the 50 feet width. In all other cases, the width of the excavation would be too great to permit bracing to be economically placed across the excavation. The sheet piling in these instances would be supported with diagonal bracing.

Where the steel sheeting is braced across the area to be excavated, a clamshell bucket would be used to perform the excavation because of

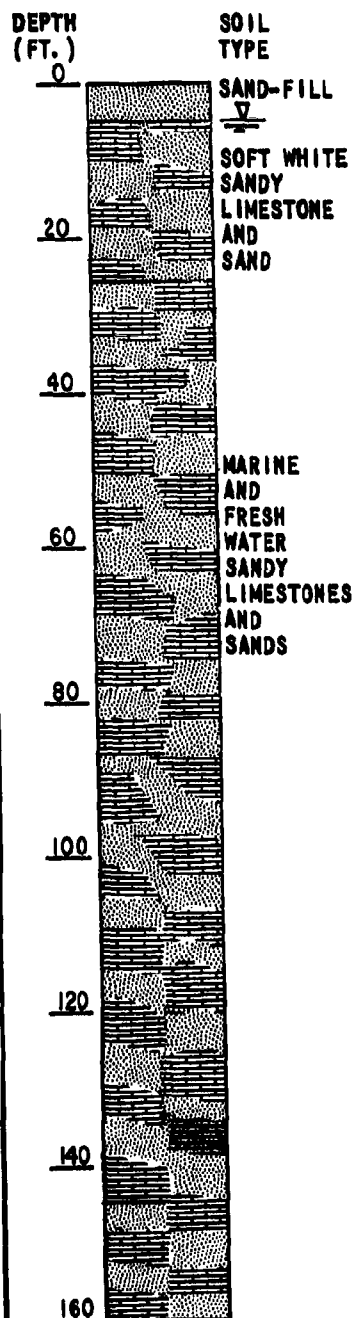
the interference from the bracing. In the larger areas where the steel sheeting is supported with diagonal bracing, the excavation below elevation -5 would be performed with a two cubic yard shovel, except that a tractor shovel would be required in the areas of interference with the diagonal bracing. In all cases where sandy limestone is encountered, the material would be broken up with a ripper prior to excavation.

When the excavation proceeded to a point where the steel sheeting and the first stage wellpoint system would be no longer completely effective in preventing water from entering the excavated area, a second stage wellpoint system would be installed with the header line adjacent to the inside face of the steel sheeting and surrounding the excavated area.

The water pressure and porosity of the limestone would be such that chemical grouting would be required to seal off the water. Some open pumping with sumps and ditches would supplement the sheeting and grouting techniques.

A bulldozer would be utilized to keep the loading area clear and thus expedite the excavation operation and to maintain the ramp roadways. Laborers would be needed to assist in the clamshell loading where the bucket cannot reach close to the steel sheeting and also to assist in excavating the material immediately adjacent to the diagonal braces.

Cost factors per square foot of effective (net bottom) areas at Location D are shown on Exhibit 15.



GENERAL SUBSURFACE CONDITIONS

Log D has been composed from published test boring data available in the vicinity of Miami, Florida. Physiographically, Miami is located in the Floridian section of the Coastal Plain province. The general characteristics of this section are young marine plains, with sand hills, swamps, sinks, and lakes.

In general, the material consists of a 5 foot sand and rubble fill. From 5 to 25 feet is a soft, white, sandy limestone and sand. Below this to the bottom of the log is a marine and fresh-water sandy limestones and sands. The water table has been assumed at a depth of 5 feet. The material between 5 and 25 feet has a rather low permeability although it will permit well points for dewatering. Below a depth of 25 feet, the material has a high permeability and the majority of the water flow is through the porous limestones and sands.

DATA PERTAINING TO EXCAVATION

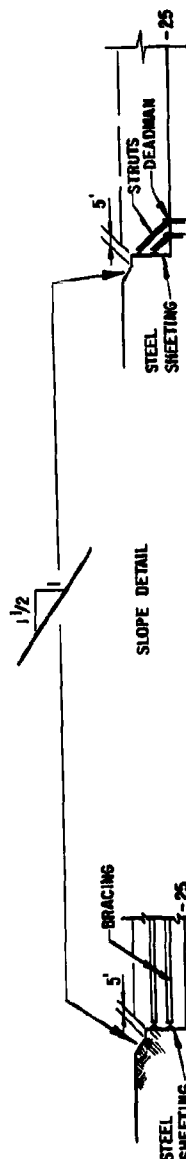
The material throughout the depth of this log can be readily excavated with conventional earth-moving equipment; however, rippers may be required through some of the limestone layers. Well points will be needed throughout the excavation or other means to control the water; also sheeting will be necessary for depths in excess of 15 feet.

As an aid in reducing the permeability of the porous limestone grouting has been found to be satisfactory.

In order to provide a basis for an estimate, it can be assumed that 30 percent of the material below elevation 25 will consist of the porous limestone which can be grouted.

DEEP CIVIL DEFENSE SHELTERS IN URBAN AREAS OPEN CUT EXCAVATION SUBSURFACE CONDITIONS-LOCATION D

DE LEUW, CATHAR & COMPANY · CONSULTING ENGINEERS · CHICAGO



25' DEPTH
APPLIES TO REMAINDER OF AREAS

25' DEPTH
APPLIES TO 5000 S.F. AREA

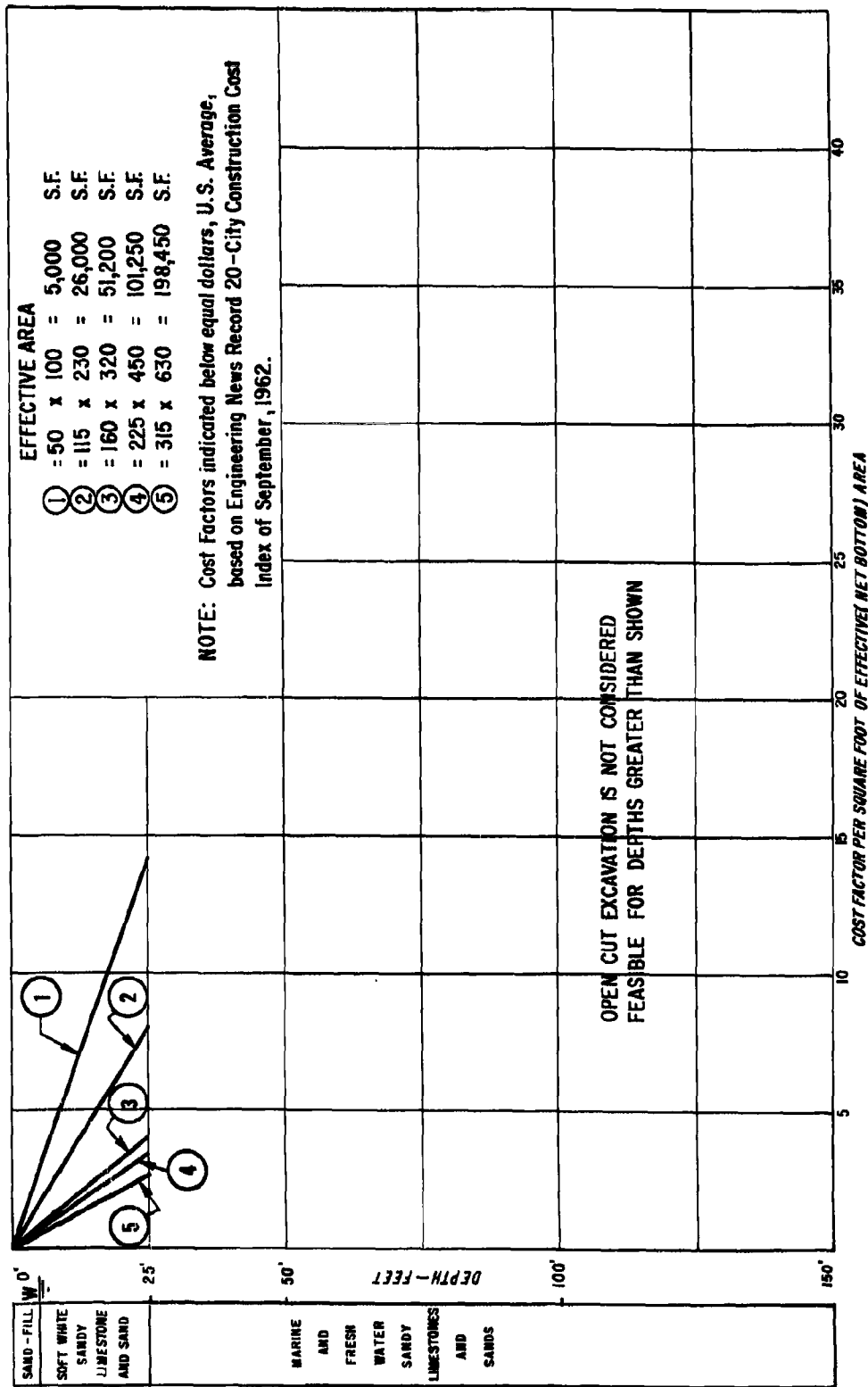
NOTE:

THE RANGE OF NET BOTTOM AREAS
FOR THIS LOCATION ARE AS
SHOWN ON EXHIBIT 3.

OPEN CUT EXCAVATION IS NOT
CONSIDERED FEASIBLE FOR DEPTHS
GREATER THAN 25 FT.



DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS
OPEN CUT EXCAVATION
SLOPE CONDITIONS - LOCATION D
PART SECTIONS
DE LEUW, CATHIER & COMPANY - CONSULTING ENGINEERS - CHICAGO



DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS

**OPEN CUT EXCAVATION
COST FACTOR CURVES - LOCATION D**

OF T.F.W. CATHART & COMPANY - CONSULTING ENGINEERS - CHICAGO

STUDY OF OPEN CUT EXCAVATION

LOCATION E

The conditions assumed for this location could be encountered in many of the central states and is typical of the wind blown deposits found in these regions. Exhibit 16 portrays the subsurface conditions and Exhibit 17 presents slopes required for the study excavation.

CASE I

Techniques and Equipment Utilized - Excavation at this location down to the water table would present no particular problems. From the water table at elevation -50 down to elevation -100, steel sheeting would be required to prevent the material from caving in.

The available lengths of steel sheet piling, together with the cost of bracing sheet piling walls of great depths, limits the practical depth of excavation to about 50 feet below the water table. It is, therefore, not considered feasible to excavate at this location to a depth greater than -100 feet -- 50 feet above the water table and 50 feet below.

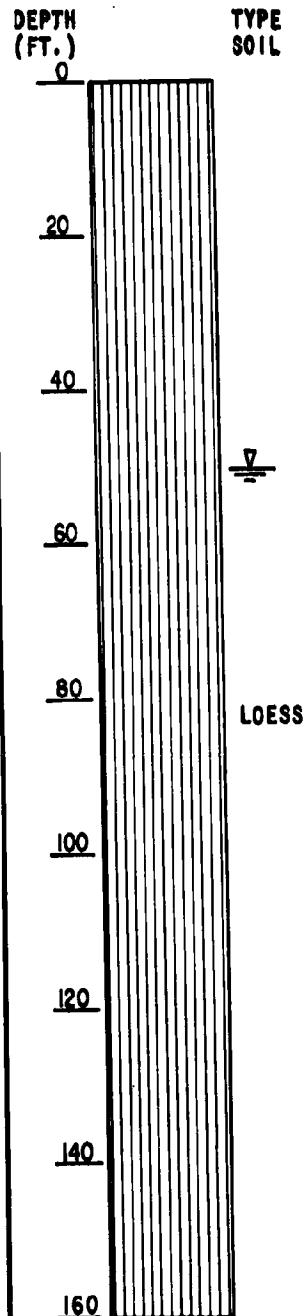
The dry material in the top 50 feet would be excavated with a two cubic yard shovel and loaded into trucks. After this material is removed, steel sheet piling would be driven on the perimeter of the excavation and bracing of the steel sheet piling installed as the excavation progresses. In the case of the 5000 square foot area, the steel bracing would span the 50 foot width. In all other cases, it is considered that the width of the excavation would be too great to permit bracing to be economically placed across the excavation and the sheet piling in these instances would be supported with diagonal bracing.

Excavation would proceed in similar manner as described for Location D.

Cost factors per square foot of effective (net bottom) area at Location E are shown on Exhibit 18.

CASE II

Case II covers a situation where swelling clays are present in the loess material. Steel sheet piling would be required from near the surface of the ground to the bottom of the excavation. Separate cost factors have not been calculated for this case as they would be very similar to those shown for Location H, Exhibit 27.



GENERAL SUBSURFACE CONDITIONS

Although Log E is not from a specific location, it is representative of a loess material that can be encountered in many locations in the Great Plains province. This material is a cohesive wind-laid soil with an effective grain size between approximately 0.02 and 0.006 mm. The material is chiefly angular quartz grains that are slightly cemented. The soils contain an intricate network of more or less vertical root holes. In a dry state the material will stand on a vertical slope. However, when wetted, the structure breaks down and it has practically no strength.

In addition, some soils in the western parts of the interior plains contain the clay mineral bentonite as a cementing agent. This, of course, complicates the structural properties by introducing swelling characteristics. The material can be encountered in a dry form to a depth as great as 80 feet. Also, the permeability in the vertical direction is approximately seven times that in the horizontal direction. The water table has been assumed at a depth of 50 feet.

DATA PERTAINING TO EXCAVATION

The material throughout the entire depth can be readily excavated by convention earth moving equipment.

Case 1: The water table is located at a depth of 50 feet with non-swelling clay as binder. The material may be cut vertically or to a 1/2:1 slope with 10-foot horizontal benches provided at 25 foot vertical intervals. Below the water table the excavation should include sufficient sheeting to retain the slopes.

Case 2: When swelling clays are present the excavation above the water table should also be protected with sheeting and bracing.

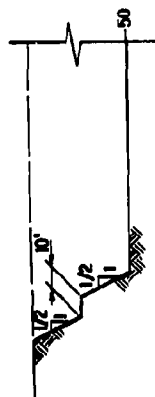
DEEP CIVIL DEFENSE SHELTERS IN URBAN AREAS OPEN CUT EXCAVATION SUBSURFACE CONDITIONS—LOCATION E

DE LEUW, CATHER & COMPANY · CONSULTING ENGINEERS · CHICAGO



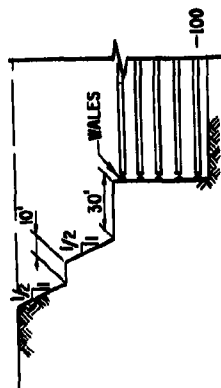
25' DEPTH

APPLIES TO ALL AREAS



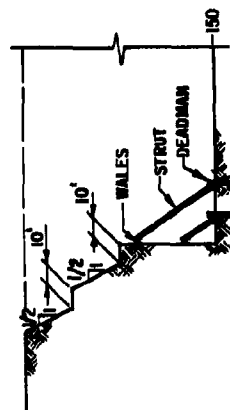
50' DEPTH

APPLIES TO ALL AREAS



100' DEPTH

APPLIES TO 5000 S.F. AREA



150' DEPTH

APPLIES TO AREAS FROM 26,450 TO 198,450 S.F.



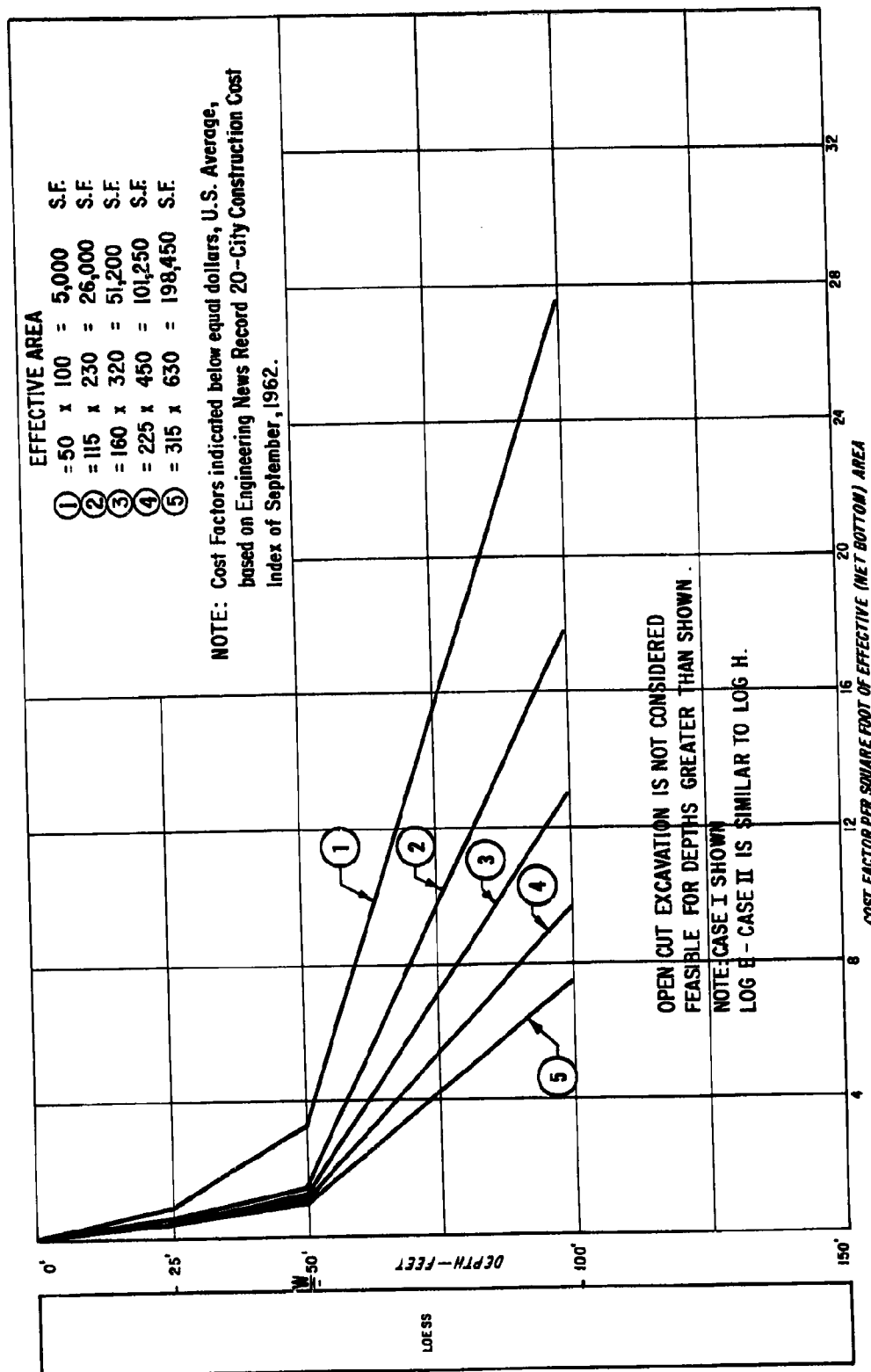
NOTE:

THE RANGE OF NET BOTTOM AREAS FOR THIS LOCATION ARE AS SHOWN ON EXHIBIT 3.

OPEN CUT EXCAVATION IS NOT CONSIDERED FEASIBLE FOR DEPTHS GREATER THAN 100 FT.

DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS
OPEN CUT EXCAVATION
SLOPE CONDITIONS-LOCATION E
PART SECTIONS

DE LEUW, CATHER & COMPANY - CONSULTING ENGINEERS - CHICAGO



STUDY OF OPEN CUT EXCAVATION

LOCATION F

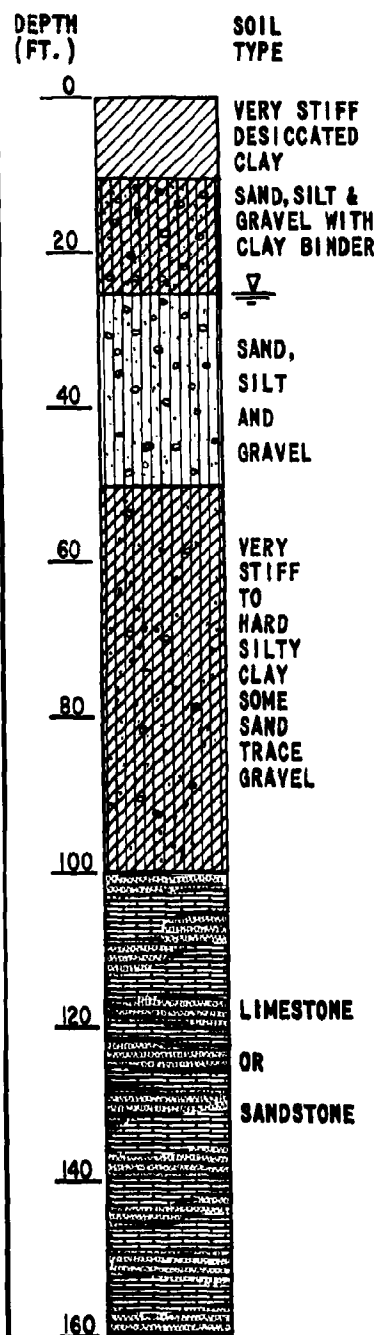
The conditions shown for this site were designed to show the change in rates of excavation as a result of encountering stratified deposits of stiff, cohesive materials alternating with granular materials. Exhibits 19 and 20 show the subsurface and slope conditions assumed at this location.

Techniques and Equipment Utilized - The top 25 feet of material above the water table would be readily excavated and loaded into trucks with a two cubic yard shovel. The rate of output would be less in the top 10 feet of stiff clay than in the next 15 feet of silty sand and gravel. The material from elevation -25 to -100 would be removed with the same equipment. The rate of progress in the very stiff clay from -50 to -100 will be less than in the sand and gravel from -25 to -50.

Due to the very low permeability of the sand and gravel and, of course, the stiff clay, the water would be controlled by open pumping with sumps and ditches.

The limestone or sandstone below elevation -100 will require one half pound of dynamite per cubic yard. The method of removal would be the same as described for the rock excavation at Location A.

Cost factors per square foot of effective (net bottom) area at Location F are shown on Exhibit 21.



GENERAL SUBSURFACE CONDITIONS

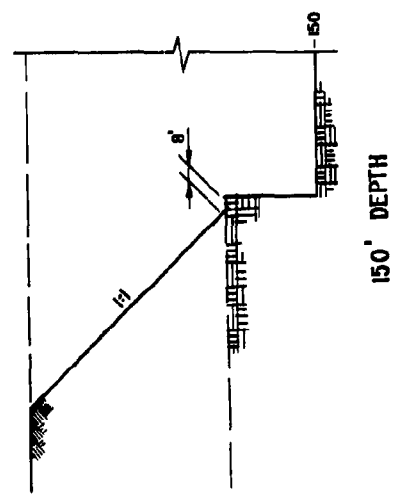
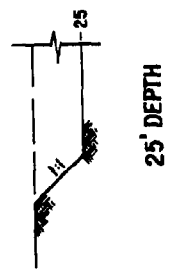
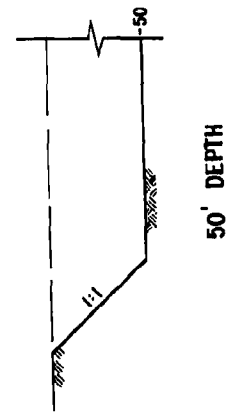
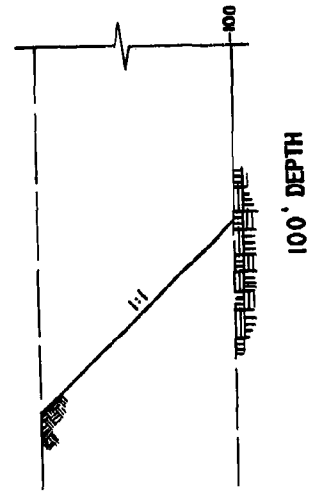
Log F has been composed from test boring data indigenous to the Till Plains section of the Central Lowland province. All the material above bedrock is the result of glacial action. The top 10 feet consists of a very stiff desiccated clay. From 10 to 25 feet the material is a mixture of sand, silt and small amounts of gravel with a cohesive clay binder. From 25 to 50 feet the material is a mixture of sand, gravel and silt with a very low permeability. Between 50 and 100 feet, the material is very stiff to hard silty clay with some sand and traces of small gravel. At a depth of 100 feet, rock is encountered which would be either limestone or in some cases, sandstone. The water table will be located at about 25 feet.

DATA PERTINENT TO EXCAVATION

The material to a depth of 100 feet can be readily excavated with conventional earthmoving equipment. The materials will all be relatively stable with a 1:1 back slope although some water can be expected in the layer between 25 and 50 feet. An 8-foot berm should be cut at the top of the rock after which the bedrock may be cut vertically to the desired depth of the excavation. This type of material is a dense limestone or sandstone and will require a moderate charge of blasting prior to excavation.

DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS
OPEN CUT EXCAVATION
SUBSURFACE CONDITIONS-LOCATION F

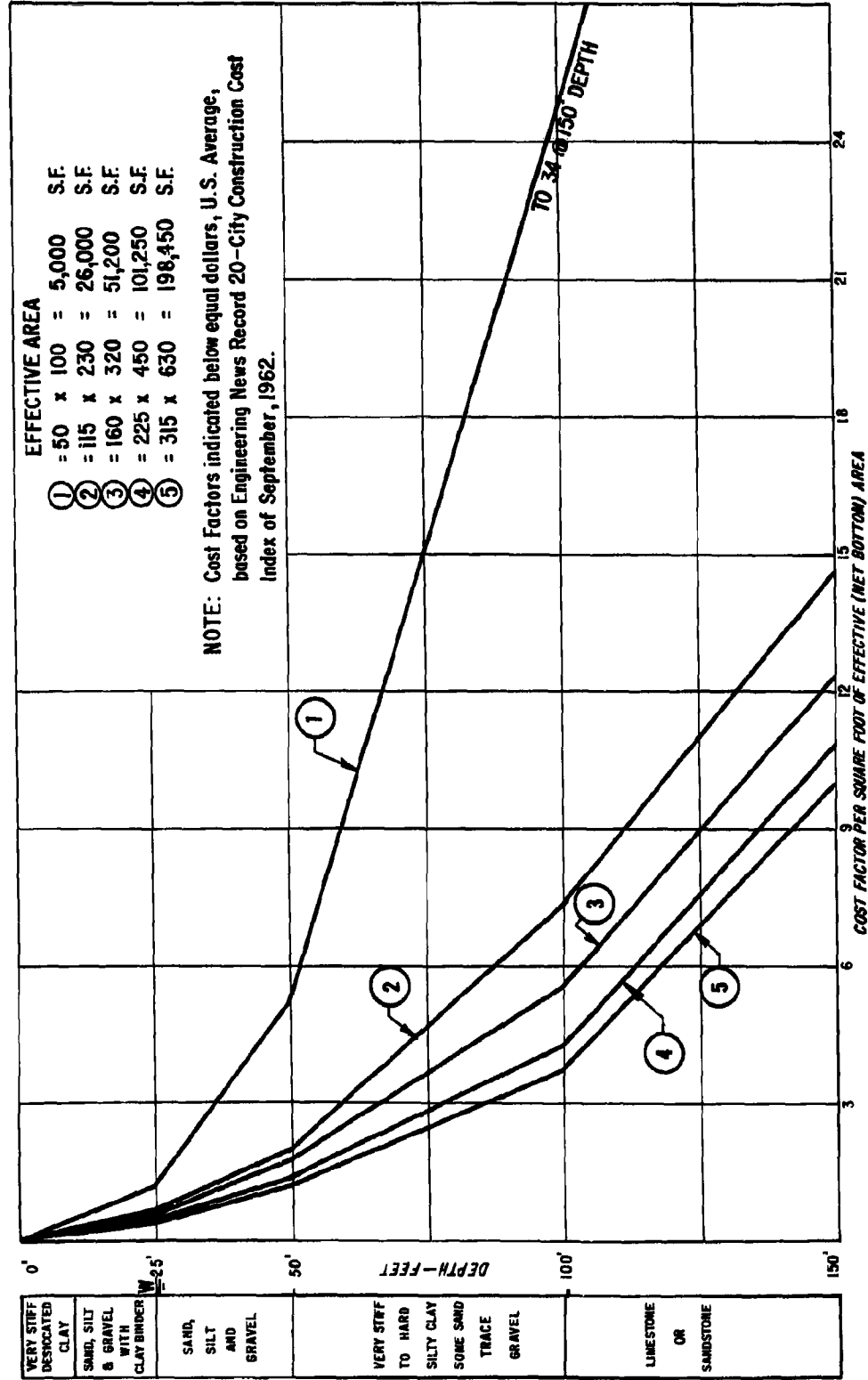
DE LEUW, CATHY & COMPANY · CONSULTING ENGINEERS · CHICAGO



NOTE:
THE RANGE OF NET BOTTOM AREAS
FOR THIS LOCATION ARE AS
SHOWN ON EXHIBIT 3.



DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS
**OPEN CUT EXCAVATION
SLOPE CONDITIONS - LOCATION F**
PART SECTIONS
DE LEUW, CATHIER & COMPANY - CONSULTING ENGINEERS - CHICAGO



DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS
OPEN CUT EXCAVATION
COST FACTOR CURVES—LOCATION F
J. H. HARRIS & COMPANY, CONSULTING ENGINEERS - CHICAGO

STUDY OF OPEN CUT EXCAVATION

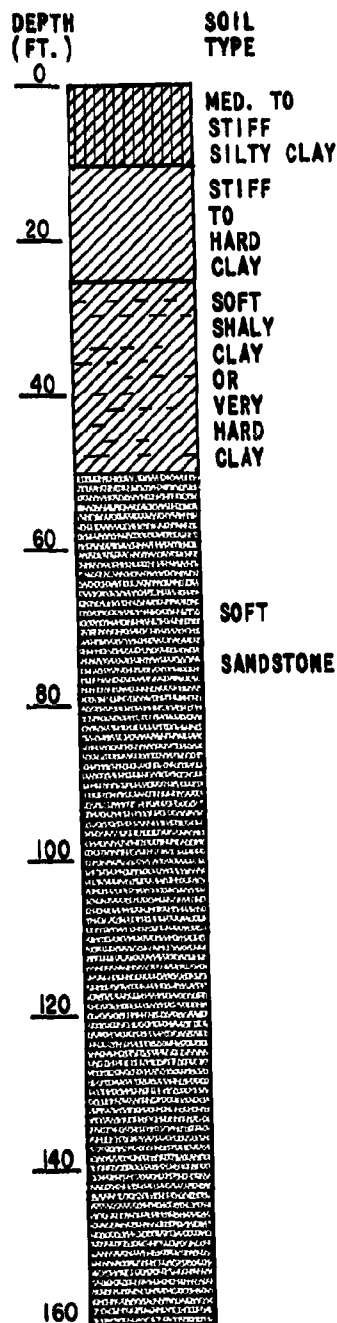
LOCATION G

The conditions illustrated for this location point up the situation wherein increased costs per unit of bottom area are realized in soft rock, as compared to costs of excavating in much harder strata. This increase is caused by the flatter slopes and consequent additional excavation required, even though no major dewatering or expensive appurtenances are required. Exhibits 22 and 23 show the subsurface and slope conditions assumed in this study.

Techniques and Equipment Utilized - Excavation at this location would be the same as for Location B down to the top of the rock, except that no serious water problem would occur. All excavation would be above the water table with the only pumping necessary as incidental to heavy rains. It is assumed that this water removal would be accomplished as required from time to time with small pumps.

It has been assumed that rock excavation would require 0.3 pounds of dynamite per cubic yard of excavated material.

Cost factors per square foot of effective (net bottom) area at Location G are shown on Exhibit 24.



GENERAL SUBSURFACE CONDITIONS

Log G has been composed from last boring data obtained in Oklahoma. Physiographically, the data is from the Osage Plains section of the Central Lowland province. This section is characterized by old scarped plains, beveling faintly inclined strata and the main streams are intrenched. This represents a complex geological stratification and the subsurface conditions can be expected to vary considerably over short distances. In fact, the bedrock varies from local outcroppings to depth in excess of the project study of 150 feet.

To provide a specific basis for further computations the following soil conditions are presented:

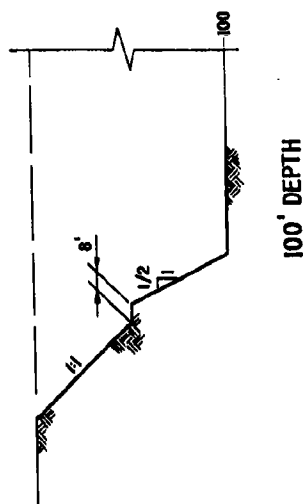
From 0 to 10 feet is a medium to stiff silty clay; from 10 to 25 feet is a stiff to hard clay; from 25 to 50 feet is a soft, shaly clay or a very hard clay. Below a depth of 50 feet is soft sandstone. The water table is considered below a depth of 160 feet.

DATA PERTINENT TO EXCAVATION

The entire depth of overburden can be excavated on a 1:1 slope. There should be an 8 foot berm at the top of the rock, and the rock may then be excavated with normal earth moving equipment after either ripping or blasting with light charges.

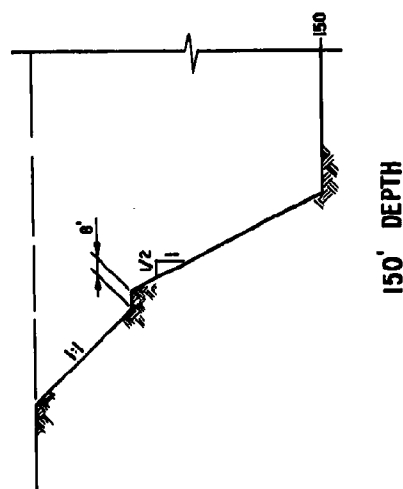
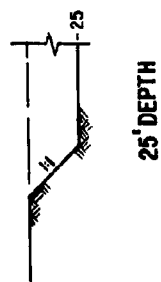
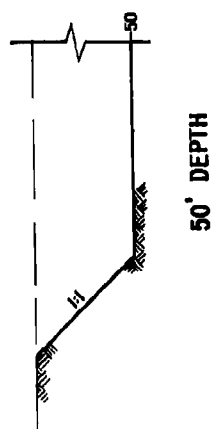
DEEP CIVIL DEFENSE SHELTERS IN URBAN AREAS OPEN CUT EXCAVATION SUBSURFACE CONDITIONS-LOCATION G

DE LEUW, CATHER & COMPANY • CONSULTING ENGINEERS • CHICAGO



NOTE:

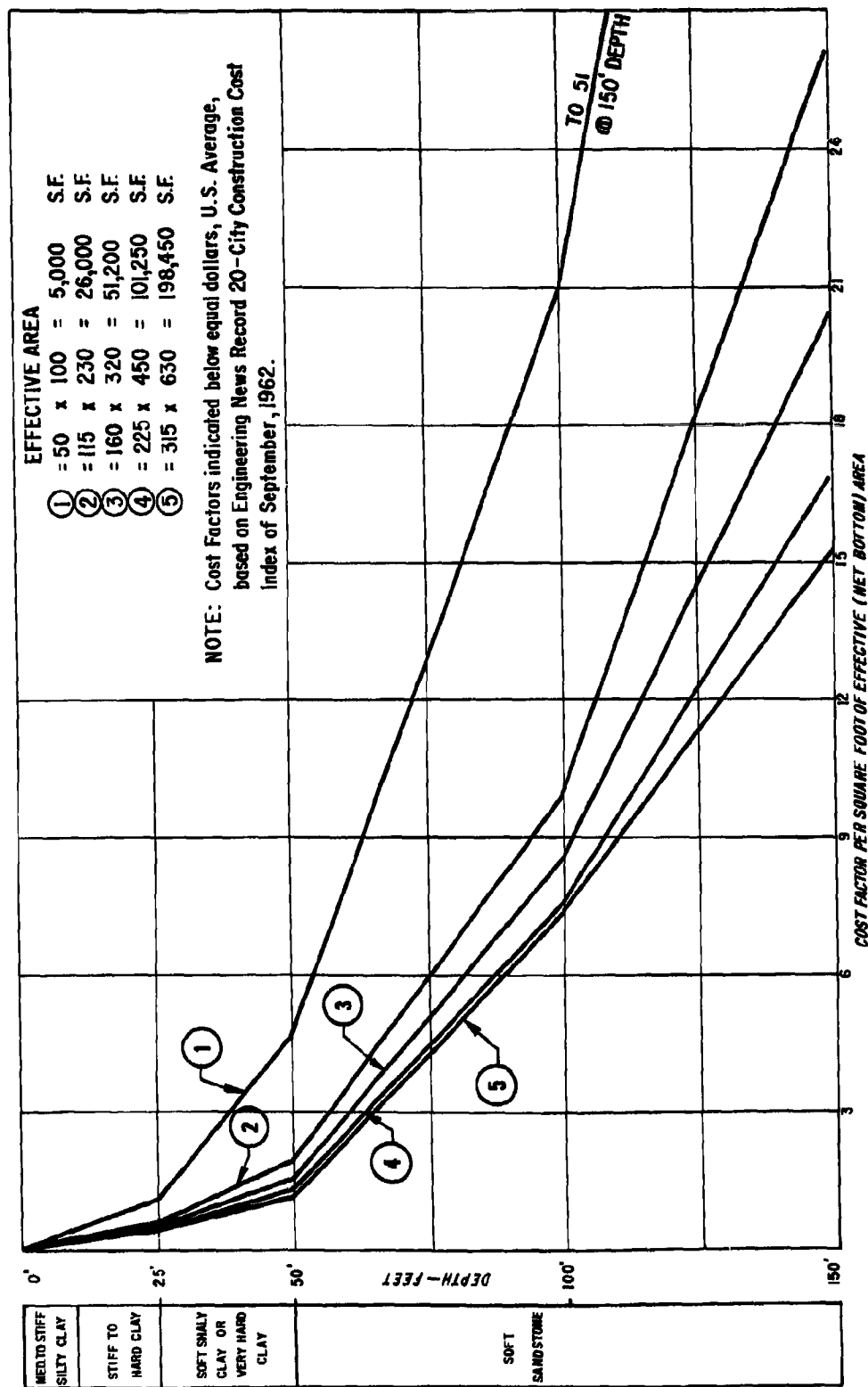
THE RANGE OF NET BOTTOM AREAS
FOR THIS LOCATION ARE AS
SHOWN ON EXHIBIT 3.



DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS

OPEN CUT EXCAVATION SLOPE CONDITIONS - LOCATION G PART SECTIONS

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DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS

OPEN CUT EXCAVATION COST FACTOR CURVES - LOCATION G

DE LEUW, CATHAR & COMPANY - CONSULTING ENGINEERS - CHICAGO

STUDY OF
OPEN CUT EXCAVATION

LOCATION H

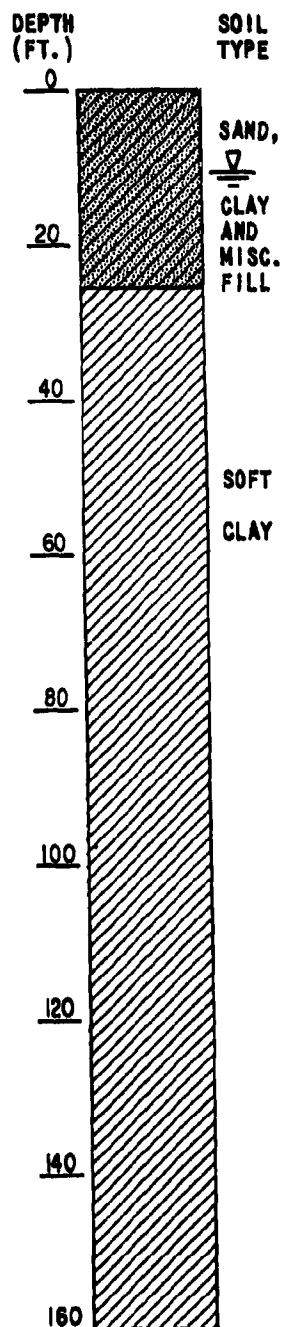
The conditions assumed for this location are typical for situations wherein soft soil conditions limit excavation into the earth. The requirement of costly appurtenances would be essential to reach even moderate depths in these strata. Exhibits 25 and 26 show the subsurface and slope conditions which would be encountered in excavating at this location.

Techniques and Equipment Utilized - The soft clay at this location would limit the depth to which the excavation may be made. The available lengths of steel sheet piling, together with the cost of bracing sheet piling walls of great depth, would limit the depth of excavation to approximately 50 feet in the soft clay. It is not considered feasible to excavate at this location to a depth greater than 75 feet, 25 feet of overburden above the soft clay and 50 feet into the soft clay.

The overburden could be readily excavated and loaded into trucks with a two cubic yard shovel.

Installation of steel sheet piling and bracing and excavation of the soft clay would proceed as described under Location D.

Cost factors covering excavation at Location H are shown on Exhibit 27.



GENERAL SUBSURFACE CONDITIONS

Log H has been composed from a general knowledge of subsurface conditions in the cities of Boston, Chicago and Detroit. Although these cities are located in different Physiographic provinces they have areas of similar soil conditions. The soil conditions shown here are normally loaded clays with either a recent alluvium or man-made surface strata. To provide a specific basis for further computations the following soil descriptions are provided. From the ground surface to a depth of 25 feet the material is a sand, clay and miscellaneous rubble fill. From 25 feet to a depth of 160 feet or more the material is a soft clay. The clays are characterized by high moisture contents and low strengths.

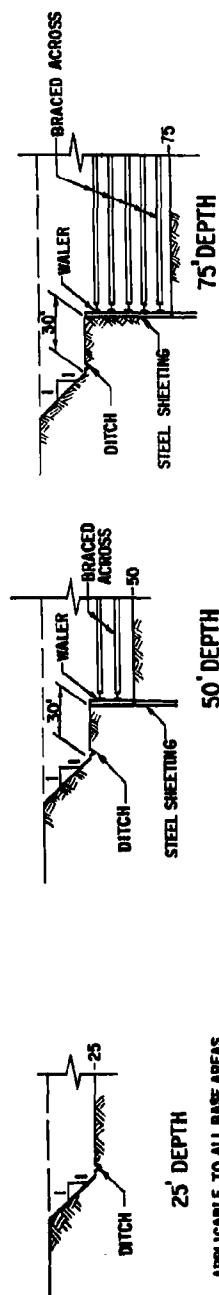
The water table is located at a depth of 10 feet.

DATA PERTINENT TO EXCAVATION

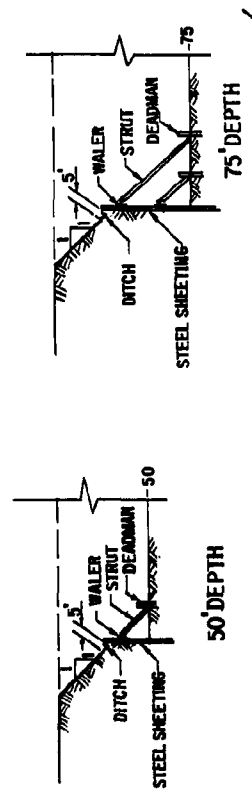
The material to a depth of 25 feet can be excavated on a 1:1 slope, and the water can be easily handled with sump pumps. Below a depth of 25 feet, it will be necessary to provide sheeting and bracing for the entire depth. It is readily recognized that this would not be one of the types of materials that would be selected for open cut operations, however, it is the type of material that is frequently encountered in soft ground tunneling, and would require pressure as well as shields for advancing the tunnel.

DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS
OPEN CUT EXCAVATION
SUBSURFACE CONDITIONS-LOCATION H

DE LEUW, CATHY & COMPANY · CONSULTING ENGINEERS · CHICAGO

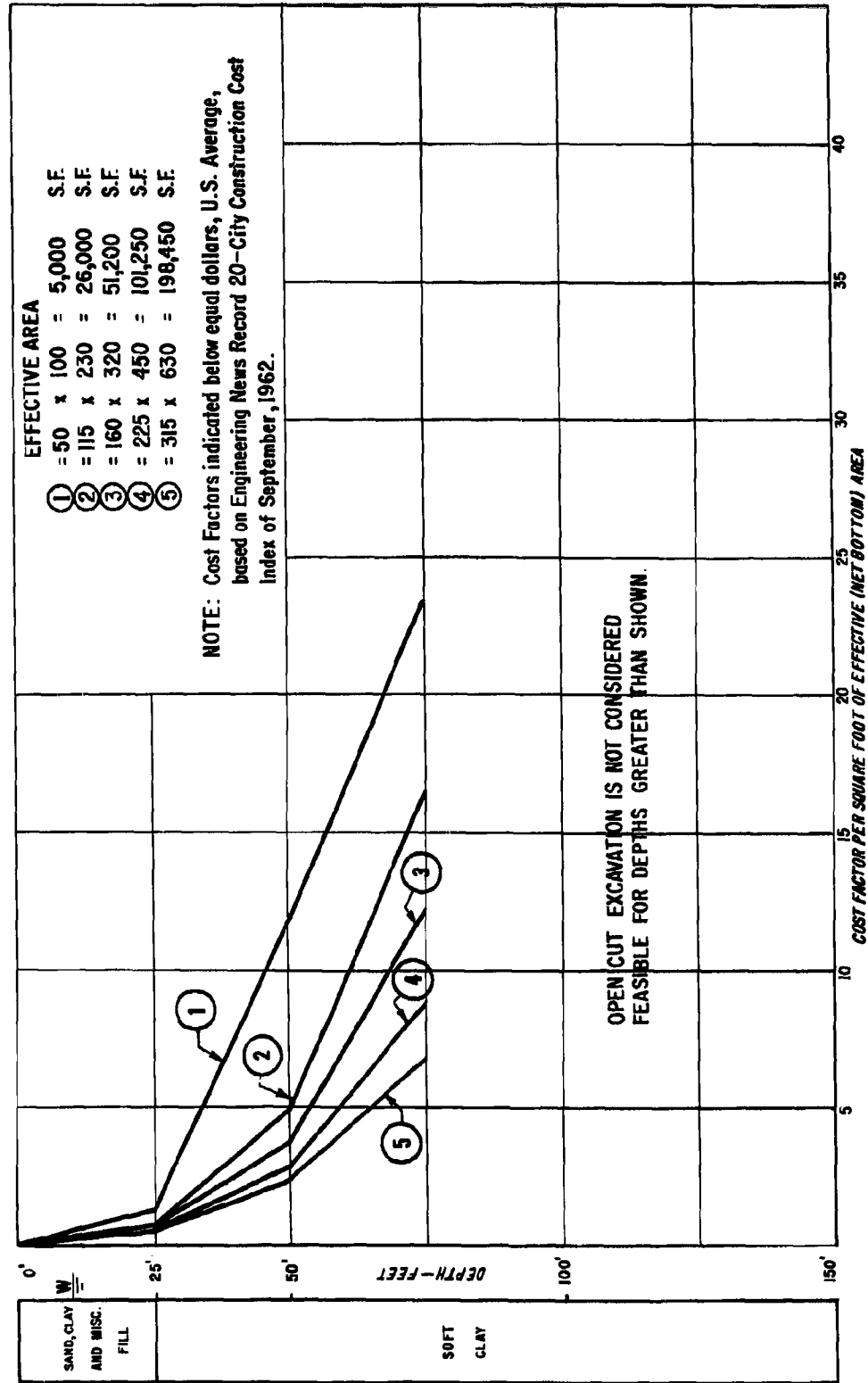


NOTE:
THE RANGE OF NET BOTTOM AREAS
FOR THIS LOCATION ARE AS
SHOWN ON EXHIBIT 3.
OPEN CUT EXCAVATION IS NOT
CONSIDERED FEASIBLE FOR DEPTHS
GREATER THAN 75 FT.



DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS
OPEN CUT EXCAVATION
SLOPE CONDITIONS—LOCATION H
PART SECTIONS

DE LEUW, CATHY & COMPANY - CONSULTING ENGINEERS - CHICAGO



DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS

OPEN CUT EXCAVATION COST FACTOR CURVES—LOCATION H

DE LEUW, CATHIER & COMPANY · CONSULTING ENGINEERS · CHICAGO

STUDY OF OPEN CUT EXCAVATION

LOCATION I

The conditions assumed at this site illustrate the characteristics encountered in strata of highly porous nature below the ground water table. Such conditions would impose the requirement of costly dewatering devices and would limit the depth of excavation. Exhibits 28 and 29 present the subsurface and slope conditions which would be encountered at this site.

Techniques and Equipment Utilized - The water condition at this location would present a serious problem. Wellpoints would be required to lower the water table to the full depth of excavation.

Theoretically, it would be possible to install wellpoints and pumps with accompanying header lines at successive vertical depths of, say, 15 feet and continue to lower the water table by a like amount. However, each time a line of wellpoints is installed, it is necessary to provide a 5 foot berm along the entire perimeter of the excavation. The cost of this excess excavation (which would be considerable with berms at 15 foot vertical intervals for a depth of almost 150 feet) together with the cost of renting, installing and operating the additional wellpoint lines, would make the overall excavation cost prohibitive. Therefore, it is not considered economically feasible to install more than a three-stage wellpoint system which would permit the excavation at this location to proceed to a depth of 50 feet.

If excavation is required only to a depth of 25 feet, a single stage wellpoint system would be installed with the header pipe on a 5 foot berm at the water table elevation -10.

If excavation is required to a depth of 50 feet, a three stage wellpoint system would be installed with the header lines resting on 5 foot berms, the top header being at the water line and the other two approximately at elevations -25 and -38, respectively.

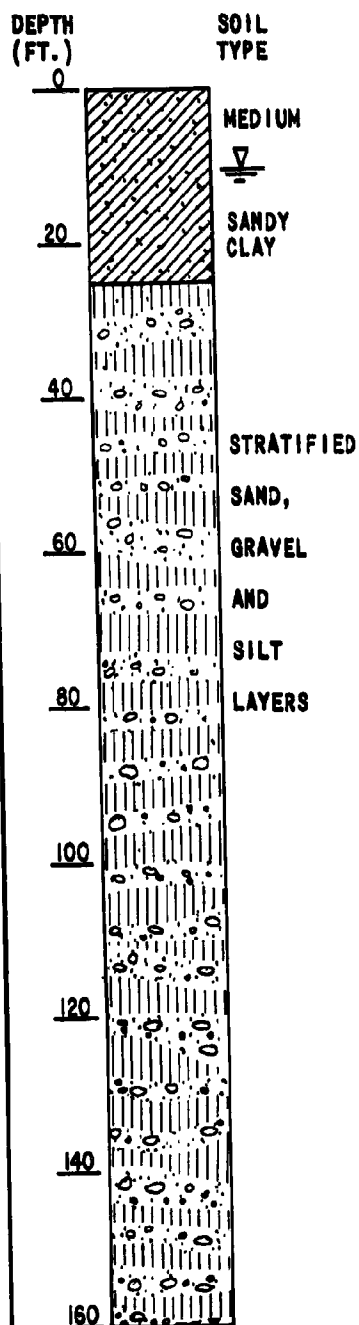
In order to economize on the rental of the wellpoint system, the first stage wellpoint system would be removed and reinstalled in the third stage position after the second stage wellpoint system has been operating long enough to permit this to be done.

In all cases, supplementary wellpoints and header lines would be required across the bottom of the excavation.

Excavation would be performed with a two cubic yard shovel loading directly into trucks. A bulldozer and operator would be provided to keep the loading area clear and maintain the ramps. Laborers are needed for incidental work in conjunction with the excavation. Steel sheeting would be provided for 10 percent of the perimeter of the excavation to retain silt pockets.

Cost factors determined for excavation at Location I are shown in Exhibit 30.

In order to illustrate the fact that it is not economical to excavate to a depth greater than about 50 feet in this type of material, cost factors for these areas were determined, carrying the excavation to a depth of 100 feet. These cost factors, also shown in Exhibit 30, indicate the rapid rise in excavation costs for depths over 50 feet in material of the type encountered at this location.



GENERAL SUBSURFACE CONDITIONS

Log I has been composed from published test boring data available from the vicinity of Memphis, Tennessee. Physiographically, this area is in the Mississippi Alluvial Plain section of the Coastal Plain province. Geologically, the materials are stratified sands, gravels and silt layers prominent throughout the lower Mississippi River Valley. To provide a basis for further computations the specific subsurface profile is provided. From the surface to a depth of 25 feet is a medium sandy clay; from 25 feet to a depth of at least 160 feet are stratified deposits of sands and gravels with layers of varying thickness of silt. The water table will be encountered at a depth of 10 feet.

DATA PERTINENT TO EXCAVATION

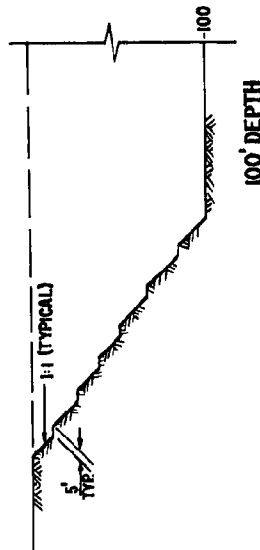
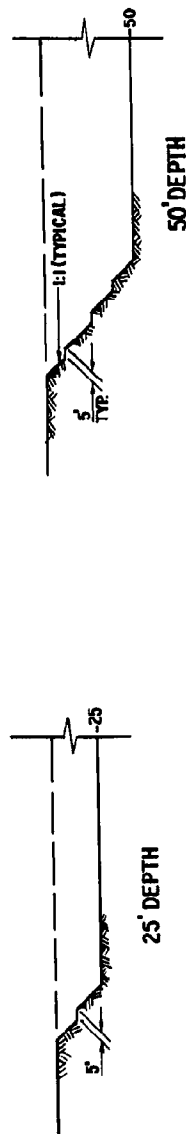
The material to a depth of 160 feet may be excavated on a 1:1 slope; however, it would be necessary to use well points or other dewatering devices for the entire depth of excavation.

It is also possible that the silt pockets will be of such magnitude that they would require local sheeting or bracing. For this particular log, it can be estimated that the bracing would constitute 10 percent of the perimeter of the total excavation.

These soils have a high permeability and dewatering devices will be required for the slopes and also the entire base of the excavation. This will be necessary to maintain the water level below the bottom of the excavation and prevent a "quick" condition.

DEEP CIVIL DEFENSE SHELTERS IN URBAN AREAS OPEN CUT EXCAVATION SUBSURFACE CONDITIONS-LOCATION I

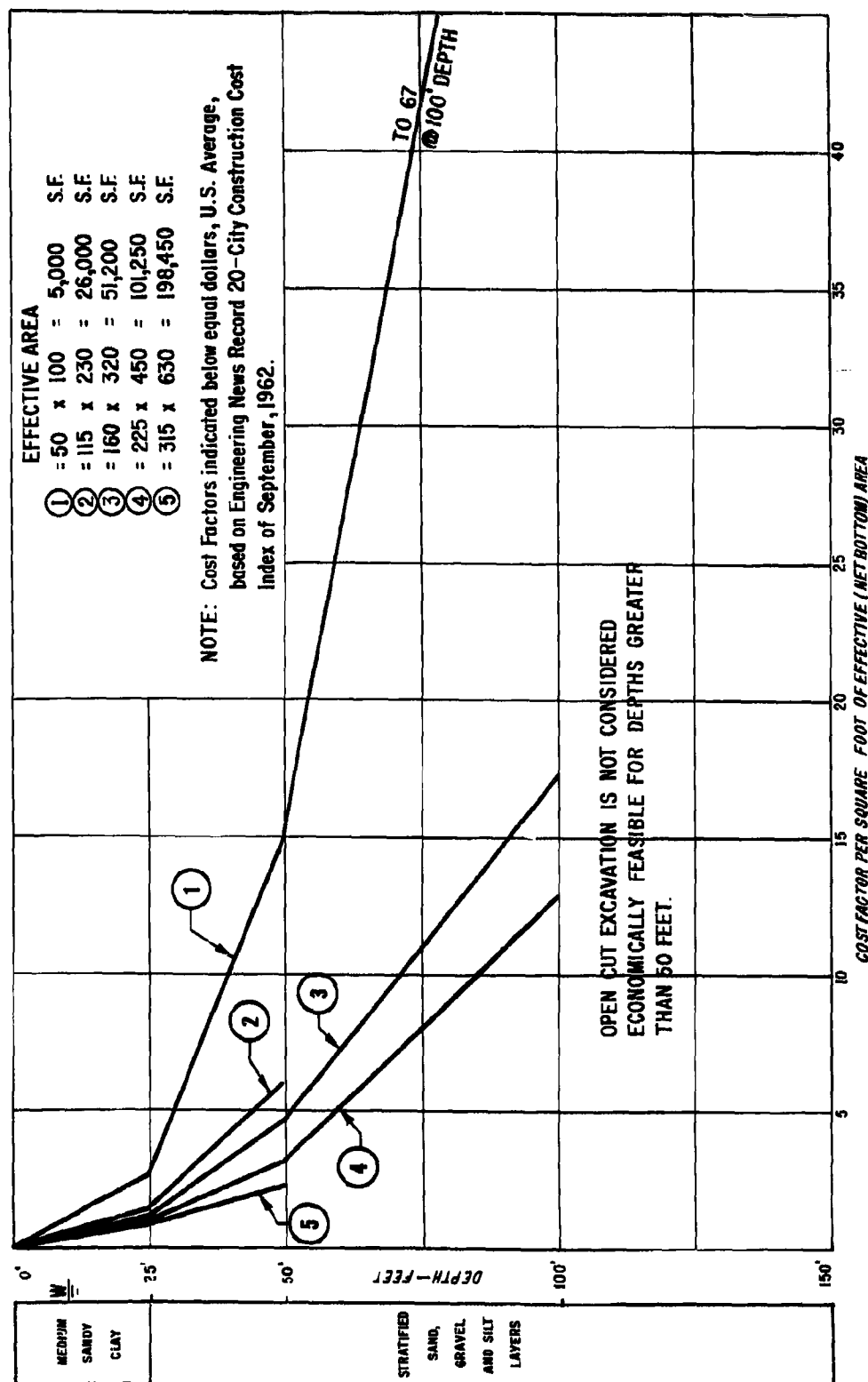
DE LEUW, CATHAR & COMPANY - CONSULTING ENGINEERS - CHICAGO



NOTE:
THE RANGE OF NET BOTTOM AREAS
FOR THIS LOCATION ARE AS
SHOWN ON EXHIBIT 3.
OPEN CUT EXCAVATION IS NOT
CONSIDERED ECONOMICALLY FEASI-
BLE FOR DEPTHS GREATER THAN
50 FT.

DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS
OPEN CUT EXCAVATION
SLOPE CONDITIONS—LOCATION I
PART SECTIONS

DE LEUW, CATHNER & COMPANY - CONSULTING ENGINEERS - CHICAGO



DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS

OPEN CUT EXCAVATION COST FACTOR CURVES - LOCATION I

DE LEUW, CATHER & COMPANY · CONSULTING ENGINEERS · CHICAGO

OPEN CUT EXCAVATION

ALTERNATIVE AND NEW TECHNIQUES AND EQUIPMENT

The preceding studies are, as stated, based on the condition of the basic usage of the shovel-truck combination. It would be in order, however, to cover the capabilities of alternative excavation equipment, since it is possible that these alternative techniques could be best suited to the functional requirements and the materials and conditions encountered. These would be of particular interest in cut-and-cover construction.

Bulldozers and Bullgraders

The bulldozer is of great value in opening up cuts, removing boulders and constructing access roads. Where material needs only to be moved 200 to 300 feet, for instance, in stockpiling materials at the periphery of an excavation, the bulldozer, either singly or in tandem combination, would compete favorably with the shovel or dragline under certain conditions in the excavation operation. It is also ideally suited for backfilling operations, especially when unsuitable ground conditions are present.

The bullgrader, with its angle blade, provides the added feature of side casting. Somewhat greater production results are achieved in loose material with the larger blade of the bullgrader. In the sloping and finishing operations, both pieces of equipment perform well, with the added advantage of the bullgrader in its slicing characteristic, thus providing a closer control of grade.

Scrapers

The crawler tractor drawn scraper can be used for digging, hauling and spreading materials, as well as cutting slopes and finishing. It can self-load more effectively than rubber-tired scrapers due to the crawler's greater tractive effort. For this same reason, this equipment can negotiate steeper grades. The crawler needs less haul road maintenance and can work efficiently under more adverse conditions. The economical haul distance is very seldom over 1000 feet in any one direction.

The two-axle rubber-tired scraper can load, haul and spread most materials where the soil is firm enough to support the machine and is soft or fragmented enough for loading. This machine is highly maneuverable in all types of material. This machine also has a good tractive effort with over 50 percent of the load weight on the drive tires. Pusher tractors are required to provide the most economical loading condition. Economical haul distance is usually over 1000 feet.

Bottom Dump Wagons

This equipment is capable of hauling and spreading more material than any scraper. They can haul over long distances at high speeds, if haul road conditions permit. They are applicable where material can be top loaded by shovel, belt loader or dragline. Material spreading is lower and more controlled than when dumped by off-the-road end dump haulers and therefore requires less leveling and grading.

End Dump Trucks

This equipment is used for hauling blasted rock or slag too heavy or abrasive to convey by other means - usually any material which must be shovel-loaded. With much of the total weight on the drive axle, this vehicle can negotiate steeper grades than scrapers, wagons or special highway trucks. They are best suited where steep grades must be negotiated, utilizing the superior horsepower-to-weight ratio of this equipment. This machine can be used to dump on level grades, over fills or into hoppers or grizzlies.

Tracked Equipment versus Rubber-Tired Equipment

Crawler tractors provide superior flotation and traction for moving heavy loads in all types of unstable material. With their compact design, spreading their weight on the drive track area, together with their superior pivoting capabilities, these machines can provide traction and maneuverability unequalled by the rubber-tired equipment. Their slow speed and destructive action on finished surfaces forms their biggest disadvantage, requiring haulage from site to site.

Self-propelled rubber-tired units provide desirable speed and mobility characteristics for long-haul operation, but lack in tractive effort, with auxiliary pusher equipment generally required. The development of large diameter low pressure tires and introduction of the wide base tubeless tire have helped to improve tractive effort.

Clam Shells and Draglines

The use of these pieces of equipment should be limited to excavating in materials which are highly plastic and difficult to handle and will not support other excavating equipment. Clam shells are also suitable for use where interferences or available space impose limitations on other techniques.

Bucket Wheel Excavators

Requirements of the coal mining industry in this country and overseas have brought about the development of bucket wheel excavators for the stripping of overburden, with the most spectacular progress made in increased capacities of these units within the last two decades.

Requirements in this field are extreme; for example, some open cut operations are designed around a daily production rate of one-half million tons of overburden and coal with the depth of these open cut excavations extending to 1000 feet. Obviously, units of the size required to move this volume of material have no application in this study. However, the efficiency of the bucket wheel technique has prompted the development of units of this type to the requirements of relatively lower volume operations.

One company specializing in this equipment, Mechanical Excavators, Inc., offers a series of unit capacities of interest with reference to the volumes of excavation contemplated in these studies. These range in working capacity from 300 to 3000 cubic yards per hour. One application of a bucket wheel excavator at a dam project in New Mexico has resulted in the excavation of approximately seven million bank cubic yards of impervious and pervious material over the past 16 months, averaging 1862 bank cubic yards per hour, with

an average of 85.6 percent availability. The pervious materials encountered consisted of cemented gravel, extremely hard, with the cut banks remaining vertical. In another application, a unit excavated approximately 100,000 cubic yards of sandstone at the rate of 1600 cubic yards per hour, where a shovel operation technique would have required drilling and blasting the material for removal.

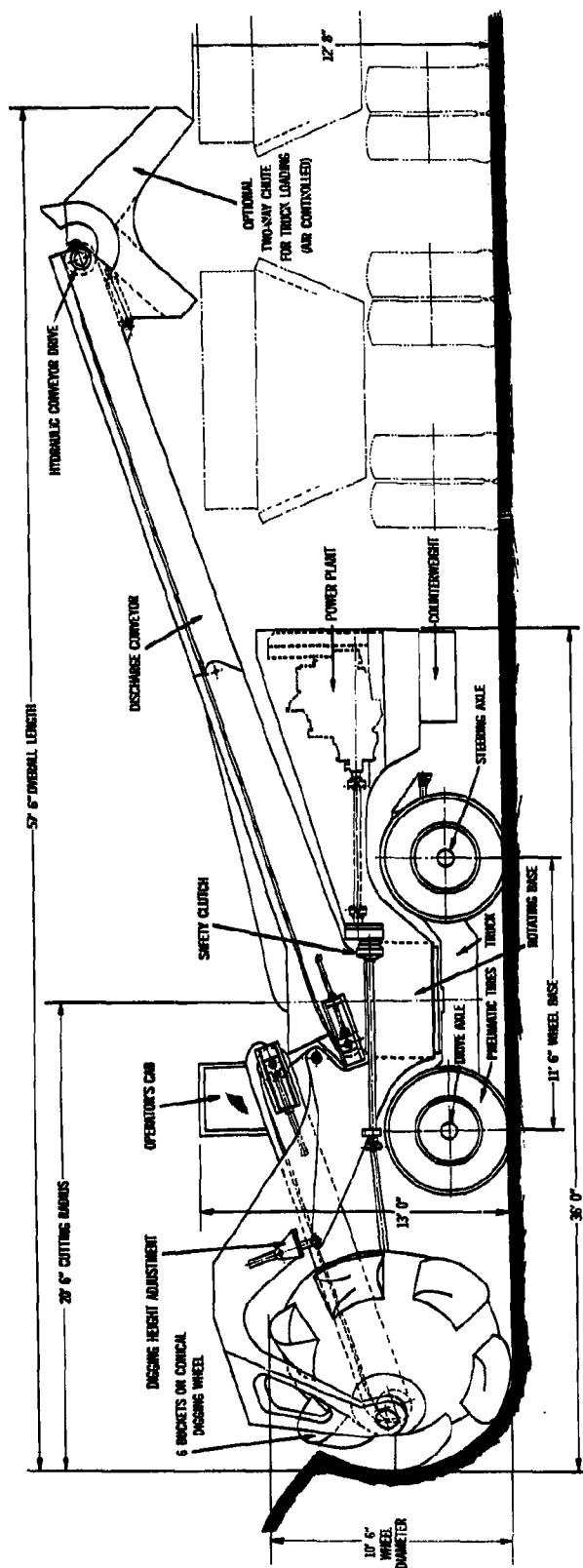
This company offers six models, with the 300 and 500 cubic yard per hour units mounted on rubber tires, and the 700, 1000, 2000 and 3000 cubic yard per hour units crawler-mounted. The tire-mounted units would be particularly of interest in this study because of their overall size, permitting travel over a road system, considering bridge clearances and similar restrictions. An elevation and specifications for Models 300 and 500 of this series is shown on Exhibit 31. The 2000 and 3000 cubic yards per hour units may also be provided with optional equipment allowing for increased cutting heights. Automatic controls may also be installed, allowing the excavating cycle to be pre-set and automatically and continuously operated within the pre-set limits, freeing the operator for observation and control over the entire excavating process.

These units range in cost from about \$120,000 for the 300 cubic yard per hour model to about \$750,000 for the 3000 cubic yard per hour model.

These units are said to be capable of excavating materials extending in characteristic from sticky clays to shale and sandstone. The manufacturer states that the units will - by the milling action of the bucket wheel - eliminate large lumps of materials, and that it will also separate over-size boulders to one side where they may be picked up with appurtenant equipment. Material could be moved from the excavator by conveyors or by dumping into trucks or wagons.

As a matter of interest, one study was prepared utilizing the bucket wheel technique for the 101,250 square foot area at Location A, excavated to 25 feet of depth. This example indicated that the cost factor per square foot of effective (net bottom) area utilizing this technique might be in the range of 0.35 compared to 0.60 utilizing the standard technique illustrated in our studies.

It is believed that the bucket wheel excavator offers the most advantages, with respect to economy and high production rate in the



SPECIFICATIONS

	MODEL 500	MODEL 300	MODEL 500	MODEL 300
Overall Length	57' 6"	57' 6"	57' 6"	57' 6"
Overall Width	11' 6"	11' 6"	11' 6"	11' 6"
Overall Height	13' 6"	13' 6"	13' 6"	13' 6"
Wheel Diameter	17' 6"	17' 6"	17' 6"	17' 6"
Wheel Speed	12 R.P.M.	12 R.P.M.	12 R.P.M.	12 R.P.M.
Number of Buckets	6	6	6	6
Bucket Capacity (Theoretical)	1/6 Cu. Yd.	1/9 Cu. Yd.	1/9 Cu. Yd.	1/9 Cu. Yd.
Swing Speed at Cutting Radius	0 to 90 F.P.M.	0 to 90 F.P.M.	0 to 90 F.P.M.	0 to 90 F.P.M.
Propel Speed —				
Forward and Reverse (10% Gradeability @ 18 M.P.H.)	0 to 18 M.P.H.	0 to 18 M.P.H.	0 to 18 M.P.H.	0 to 18 M.P.H.
*Maximum Theoretical Capacity	720 Cu. Yd. P.H.	480 Cu. Yd. P.H.	480 Cu. Yd. P.H.	480 Cu. Yd. P.H.
Ladder Belt — Width	30"	30"	30"	30"
— Length	16' 0"	16' 0"	16' 0"	16' 0"
— Speed	450 F.P.M.	350 F.P.M.	350 F.P.M.	350 F.P.M.
Discharge Belt — Width	30"	30"	30"	30"
— Length	11' 5"	11' 5"	11' 5"	11' 5"
— Speed	500 F.P.M.	500 F.P.M.	500 F.P.M.	500 F.P.M.
Engine with Torque Converter	25 H.P.	25 H.P.	25 H.P.	25 H.P.
Hydraulic Motors — Ladder Belt	25 H.P.	25 H.P.	25 H.P.	25 H.P.
— Discharge Belt	40 H.P.	40 H.P.	40 H.P.	40 H.P.
— Swing	40 H.P.	40 H.P.	40 H.P.	40 H.P.
— Propel	6-18.00 x 25-24 Ply	6-18.00 x 25-24 Ply	6-18.00 x 25-24 Ply	6-18.00 x 25-24 Ply
Tires	11' 5"	11' 5"	11' 5"	11' 5"
Wheelbase	11' 0"	11' 0"	11' 0"	11' 0"
Recommended Cutting Height	20' 0"	20' 0"	20' 0"	20' 0"
Cutting Radius	41 Tons	41 Tons	41 Tons	41 Tons
Total Weight of Machine (Approx.)	41 Tons	41 Tons	41 Tons	41 Tons

**Optional Equipment—35' long conveyor with 2-way chute for continuous truck loading.

*Digging capacity will vary with the type of material.

larger ranges of excavations contemplated herein, of any open cut technique available at present, stipulated on the capabilities of this equipment in the materials encountered.

Attention would necessarily be given to the matter of maintaining an adequate number of haulage units or sufficient conveyor capacity to support the high rate of continuous output of this equipment.

Water Control

One process of interest in the control of ground water in sand prior to excavation is under development by Cementation Company Limited, London, England. This technique is called the "jet grouting process", and is intended to produce virtually impermeable cutoff walls to a depth of at least 40 feet, using cement in combination with the natural materials.

This technique employs a rotating bit, drilled to the full depth required for the cutoff wall, with the bit fitted at the bottom with a radial jet. As the bit is slowly withdrawn, the rotating jet ejects cement grout. The grout combines with the natural materials with the resultant mixture settling into the cavity created below the bit to form a concrete column. The size of the column is controllable by the size of the jet and the pressure applied, and the strength of the column varied by modifying the grout concentration. As the injection and withdrawal of the bit proceeds along the intended cutoff, the grout columns so formed result in a monolithic concrete wall, said to be almost impermeable to ground water.

It would appear that since the development of this process has not yet included the installation of any type of reinforcing, that the utilization of this cutoff wall where the total area inside the wall would be excavated would limit this technique to a circular shape. However, it would seem that this technique could be utilized on larger excavations in sand, where the cutoff wall so formed would be an adequate distance from the excavation to allow slope stability.

The average rate of progress is quoted to be 120 square feet of completed cutoff wall per rig shift.

Shelters in Water Bearing Ground

Some of our urban areas in the coastal regions are located in terrain only slightly above sea level with ground water conditions presenting a nearly impossible situation, as related to even relatively shallow excavations. If it is deemed mandatory that shelters would be provided under the surface at these locations, two possible approaches might be considered to the problem.

Caissons. The first might employ a caisson technique, similar to that utilized to provide foundations for some of our larger bridge structures. These caissons, which would probably require the use of compressed air, would be sized to the shelter area requirements and equipped with a cutting edge. As the caisson is constructed above grade, it would be lowered by excavating within the base of the caisson to the intended elevation. These caissons could also be connected in series by short tunnels to form a larger shelter area.

It was noted in a recent issue of Engineering News-Record that a contractor in Japan is engaged in constructing a 140 feet by 110 feet, nine-story building, with an 80-foot deep basement, with the basement area excavated by the caisson technique. Progress in lowering the building is quoted to be approximately nine inches per day, utilizing mechanical excavating equipment in the interior area, with hand excavation utilized around the perimeter cutting edge. In this instance, a bentonite slurry trench was utilized outside the periphery of the building to decrease friction and to seal off ground water.

Sub-aqueous Tunnels. The second technique would involve the excavation of material in the wet. After the hole is dredged to the required size and depth, a prefabricated tunnel section, designed to be watertight and to resist buoyancy, would be installed and backfilled - this technique being the same as that used in the construction of some of our sub-aqueous vehicular and rail tunnels.

The inherent complexities of these last two techniques, with respect to evaluation of costs and detailed feasibility, are not within the scope of this report. However, it can be stated that these and other possible approaches to the matter of installation of subsurface shelter areas in the low terrains characterized by extremely serious ground water conditions, would be manifested by extremely high construction, maintenance and operating costs.

STUDIES OF TUNNELS IN EARTH

General Considerations

There will be locations in urban areas where lack of open area, high property values, and density of development will not allow the consideration of the open cut excavation method in the construction of shelter space. As illustrated in the open cut excavation studies, soil conditions will be encountered in some of our urban areas where the open cut methods would be uneconomical and impractical, considering depth requirements, function, and associated features. The alternative of considering tunnel shelter areas under these conditions would, in some instances, be feasible. It could be quite possible that a comparison of total costs for a specific shelter area in a specific subsurface material would reveal the economy of a tunnel shelter compared to one constructed in conjunction with open cut excavation methods. As discussed in the General Considerations of Studies of Open Cut Excavations, the matter of access to the tunnel shelters could affect, to a considerable extent, the methods used to excavate and construct the shelter area.

Establishing Soil Conditions, Tunnel Geometrics and Dimensions

Two general parameters of ground conditions were assumed for the purpose of studying tunnels in earth with respect to the costs of excavation and primary and secondary lining. The first general condition assumes that these tunnels would be located in soft or running ground that could be stabilized by the use of compressed air or in firm ground having the characteristics of firm clay, with this condition requiring the use of a shield in the driving of the tunnel. The second general parameter assumes that the tunnels would be located in stiff clay or in soft clay that could be stabilized by compressed air, allowing the advance to proceed without the use of a shield.

Criteria governing these studies has established that headroom of nine feet per floor would be provided over the usable floor area of the shelter. This clearance was utilized in designing the optimum size tunnel sections, single and multi-floor types, for inclusion in these studies.

The wall and floor thicknesses indicated on the designs are based on the general requirements imposed by the materials surrounding the tunnel, by the method of driving, and by the assumption that the tunnel would have 75 feet of cover over the crown of the cross section. Variations in these thicknesses would result from a change in these assumptions.

Shield Driven Tunnels. Three designs of tunnel--Sections A, B and C--were developed for tunnels in earth using the shield, each of which utilizes the circular cross section in accordance with normal shield practice. Reference is made to Exhibit 32 showing these sections.

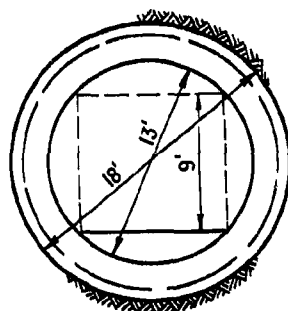
Section A provides an inside diameter of 13 feet with an outside (driving) diameter of 18 feet. This section would provide approximately 9 square feet of usable floor area per lineal foot of tunnel on a single floor.

Section B provides a dual tunnel section, interconnected with a secondary excavation, with the primary bores having an inside diameter of 13 feet and outside (driving) diameter of 18 feet. This section would provide 30 square feet of usable floor area per lineal foot of tunnel on a single floor.

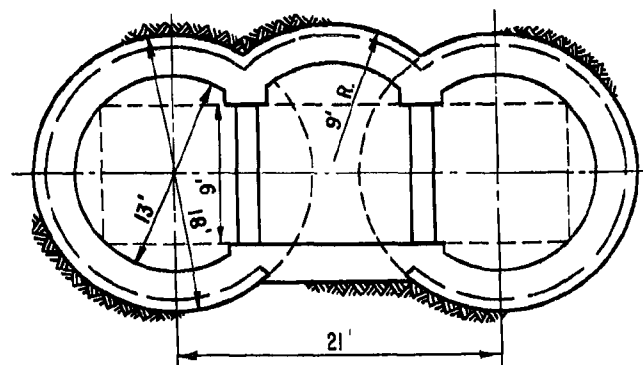
Section C has an outside (driving) diameter of 34 feet and an inside diameter of 27 feet. This section allows the installation of two floors, providing a usable floor area of 38 square feet per lineal foot of tunnel.

It is noted that no greater outside diameter has been studied, due to the economic factors involved in the shield and rib design and the normal expectation under the conditions assumed that the differential ground pressures between crown and invert of the tunnel could not be dealt with in construction.

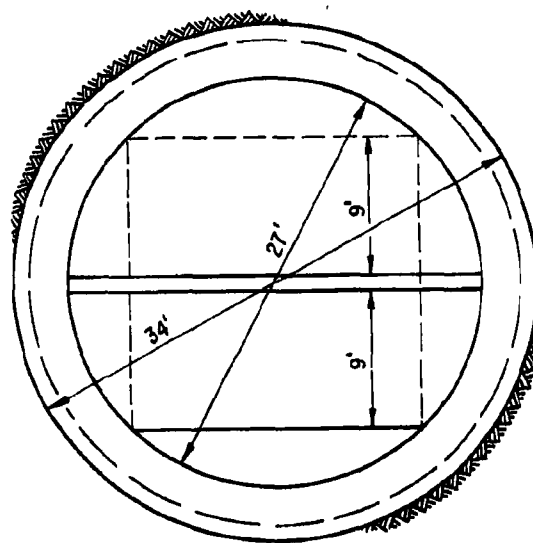
Tunnels Driven Without Shield. Two designs, Sections D and E, were developed for earth tunneling under conditions not requiring a shield. The removal of the shield requirement and assumed improvement in ground conditions allows the design of the cross section to be in the form of a semi-circular arch with vertical sides and a flat bottom. See Exhibit 33 for these sections.



SECTION A



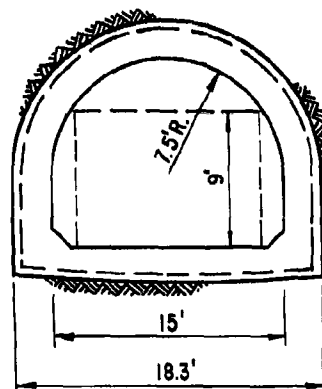
SECTION B



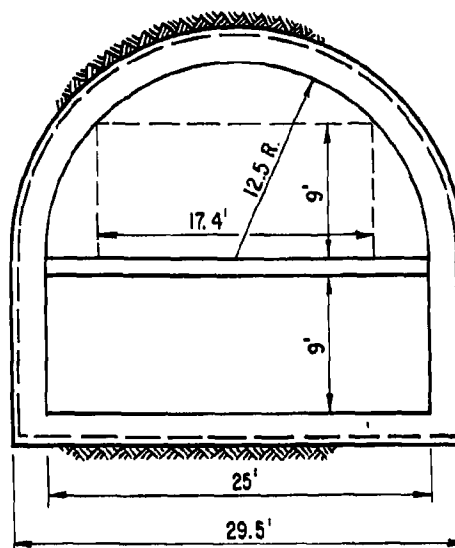
SECTION C

DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS
TUNNELS IN EARTH REQUIRING SHIELDS

DE LEUW, CATHY & COMPANY · CONSULTING ENGINEERS · CHICAGO



SECTION D



SECTION E

DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS
TUNNELS IN EARTH WITHOUT SHIELDS

DE LEUW, CATHER & COMPANY · CONSULTING ENGINEERS · CHICAGO

Section D provides an inside base width of 15 feet and a maximum inside height of 12 feet, with approximately 12 square feet of usable floor area per lineal foot on a single floor.

Section E provides an inside base width of 25 feet and a maximum inside vertical height of 22.5 feet, with a usable area of 42 square feet per lineal foot of tunnel on two floors.

Studies of Specific Conditions - Tunnels in Earth

Under the section on Tunnel Geometrics and Dimensions, the materials through which the assumed tunnels would pass were established. Sections A, B and C are developed for conditions imposed by plastic clay, and Sections D and E provide for excavation through stiff clay, with 75 feet of cover over the crown of the tunnel in each case.

The following assumptions are used in these studies to develop the cost factors of excavation and separate factors of lining as required:

1. Access to the tunnel is by shaft; the shaft cost is not included.
2. The cost of mucking the material into a hopper or trucks at the top of the shaft is included; the cost of disposal of the material is not included.
3. Hand mining is utilized throughout. When the work proceeds under minimum air pressure or under free air, labor requirements in the heading are based on three shifts per day, six day week for mining and installation of the primary lining and one shift per day, six-day week for installation of the concrete lining. When the work requires compressed air of 15 to 25 psi, labor requirements in the heading are based on four shifts per day (six hour shift), six day week for mining and installation of the primary lining and one shift per day (six hour shift), six day week for installation of the concrete lining.
4. A contract length for each section was established to be 5500 lineal feet, with tunneling within that length advancing from a single shaft.

5. Compressed air, where utilized, would amount to 15 to 26 psi at the heading in shield driven tunnels and 5 to 15 psi where the shield is not required.
6. Steel ribs and liner plates would be utilized for the primary lining with reinforced concrete for the secondary lining. When using a shield in free air, wood lagging would be substituted for steel liner plates.
7. The lining of the tunnels is predicated on static loading only.
8. The cost of floors required for multi-level usage is not included.

Utilizing these bases, estimated costs were developed per linear foot for each section considered and, thereafter, per square foot of usable area.

The consideration of variable cost factors for tunnels providing lesser or greater usable areas than the hypothetical contract was given much thought, with these conclusions:

1. It is apparent that there is a minimum length of tunnel of the sizes considered, below which it is not practical, economically, to set up the plant, excavate a shaft, provide air compressor equipment and install a shield, if required. This length is not ascertainable within the limitations of this study, but would probably be determined by a reasonable decision based on the total contract price measured against the equipment investment involved, considering the specialized contractor capabilities.
2. In a program of considerable magnitude, several contracts for tunnel construction could be awarded concurrently with added contracts awarded in a second phase, scheduled to provide for utilization of the contractors' special equipment and experienced labor. Contractors would have an opportunity on the second phase to reduce, possibly to a considerable extent, the bid price, because some of the more expensive equipment items might be fully depreciated after the first

phase, even though they would still be capable of usage in a second and following contracts.

It is concluded, therefore, that it would not be possible, within practical means, to determine a variation in unit costs of various lengths of tunnel from a minimum length to the maximum length, based on usable square footages yielded thereby.

The costs for shafts could cause little effect on the total cost for a tunnel in the case of the longer and larger tunnels at shallow depths or a more substantial effect in the case of shorter tunnels of less size at greater depths. Reference is made to "Preliminary Report of Data on Construction and Use of Deep Shafts", June 1962, as prepared by the Corps of Engineers. This report presents estimated cost curves at 1962 price levels covering a range of shaft diameters from 10 to 30 feet under "ideal" and "severe" conditions. These indicate, for example, that the costs per linear foot for a 15 foot diameter shaft could range from about \$800 to \$1,620 and for a 30 foot diameter shaft from about \$2,450 to \$5,100.

Referring to this and other relevant data, it is suggested that provision for the cost of shafts, where required in a general assessment of tunnel shelter costs, be made at a rate per linear foot of depth of one and one-half to two times the cost per linear foot for a comparable tunnel under comparable conditions.

The estimates of cost of the tunnel sections which follow were prepared using as a basis labor rates, equipment rental rates or purchase prices, and material prices in effect in the Chicago area as of November 1962. It is considered that these costs would not change materially if transposed to other urban areas of the country, because of the highly specialized labor and equipment required for tunneling operations. Any variation for this reason could easily be offset by other factors governing the bidding processes.

Table 1 provides a summation of usable floor areas, required lengths of tunnels producing from 5,000 to 300,000 square feet of usable area and estimated construction costs for Sections A, B and C, which require the use of a shield.

Table 2 provides the same data for Sections D and E which, under the assumed conditions, do not require a shield.

TABLE 1
TUNNELS IN EARTH REQUIRING SHIELD
USABLE AREAS AND LENGTHS

	<u>Section A</u>	<u>Section B</u>	<u>Section C</u>
	<u>In Square Feet</u>		
Usable Areas of Tunnel Per Linear Foot	9.4	30.4	38.4
Length of Tunnel Required to Produce Usable Areas of:	<u>In Linear Feet</u>		
5,000 Sq. Ft.	530	(Negligible)	
50,000 Sq. Ft.	5,320	1,640	1,300
100,000 Sq. Ft.	10,650	3,280	2,600
200,000 Sq. Ft.	21,300	6,560	5,200
300,000 Sq. Ft.	31,950	9,840	7,800

Estimated Construction Costs
From Study Examples

Using Compressed Air

Cost Per Linear Foot	\$1,160	\$2,160	\$2,580
Cost Per Square Foot of Usable Area	124	71	67

Without Compressed Air

Cost Per Linear Foot	\$ 860	\$1,620	\$1,940
Cost Per Square Foot of Usable Area	92	53	51

TABLE 2
TUNNELS IN EARTH WITHOUT SHIELD
USABLE AREAS AND LENGTHS

	<u>Section D</u>	<u>Section E</u>
Usable Areas of Tunnel Per Linear Foot	<u>In Square Feet</u>	
	12.0	42.4
Length of Tunnel Required to Produce Usable Areas of:	<u>In Linear Feet</u>	
5,000 Sq. Ft.	420	(Negligible)
50,000 Sq. Ft.	4,160	1,180
100,000 Sq. Ft.	8,320	2,360
200,000 Sq. Ft.	16,640	4,720
300,000 Sq. Ft.	24,960	7,080

Estimated Construction Costs
From Study Examples

Using Compressed Air

Cost Per Linear Foot	\$ 710	\$1,530
Cost Per Square Foot of Usable Area	59	36

Without Compressed Air

Cost Per Linear Foot	\$ 670	\$1,470
Cost Per Square Foot of Usable Area	56	35

TUNNELS IN EARTH

NEW TECHNIQUES AND EQUIPMENT

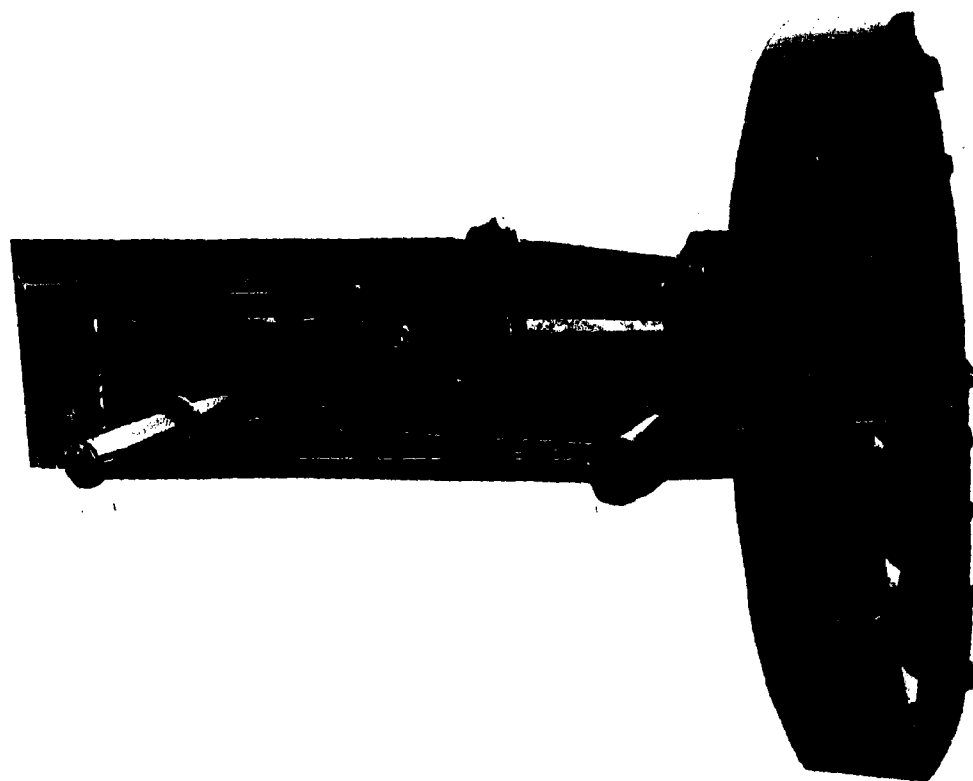
Boring Machines

Within the past decade, many tunnels have been driven successfully, expeditiously and economically utilizing boring machines, with or without shields. Their best application would be in the self-supporting soils, although these devices have been used with success in softer grounds in conjunction with compressed air. The important consideration in the selection of this technique relates to the uniformity of material. On one project in stiff gray clay, under free air, a tunneling machine showed a rate of progress of up to 40 feet per shift on a 14 foot outside diameter tunnel. This same basic design was being used on a 16 foot diameter sewer tunnel through moderately hard sandstone under free air. The machine may be adapted to different subsurface conditions by modifying the spacing and type of teeth on the cutting arms. For example, in the clay tunnel, mild steel teeth were spaced to provide a full face cut, whereas in the hard sandstone, they were of a harder alloy, spaced to cover only 50 percent of the face with the remainder of face material being removed by the chipping action of the teeth. Exhibit 34 shows one type of boring machine used in earth tunneling in firm clays.

Another project in Canada provided a comparison of hand mining versus machine excavation. The hand mined tunnel, 16 feet in diameter in free air, required nine men per shift at the face for three eight hour shifts, producing an advance of nine feet per day. The machine excavated tunnel required five men per shift at the face for three eight hour shifts, producing a rate of advance of 18 feet per day.

One study example, utilizing the boring machine technique, was prepared to assess the cost differential, if any, compared to the hand mining technique. This example, based on tunnel Section A, with shield and without compressed air, indicates that a potential saving in construction cost of approximately 25 percent would be realized, under the conditions assumed for the study.

EXHIBIT 34



SCOTT EXCAVATING MACHINE
COURTESY OF TERRAFORM FOOTING COMPANY

Placement of Tunnel Lining

An interesting technique, relating to the placement of concrete lining for an earth tunnel, has been used on a project in South America. This tunnel, 13.5 feet in diameter, was driven with a Mayo shield. Behind the shield was a bulkhead, conforming in size to the end cross sectional area of the concrete liner. This bulkhead was connected to and followed by the form for the lining.

As the shield was jacked forward, concrete was placed in the space vacated by the shield. As the shield was again advanced, the jacking action bore against the bulkhead, compressing the wet concrete and also forcing it into the tail void. This compression forced some water out of the concrete, causing it to react in like manner to concrete placed by the vacuum process. Incremental movements of about three feet were made at about four hour intervals. The design did not provide for reinforcing in the concrete lining.

This technique might be most appropriate in materials which exhibit somewhat more stable characteristics, although the use of the shield in this instance indicated the presence of soft ground conditions to some degree.

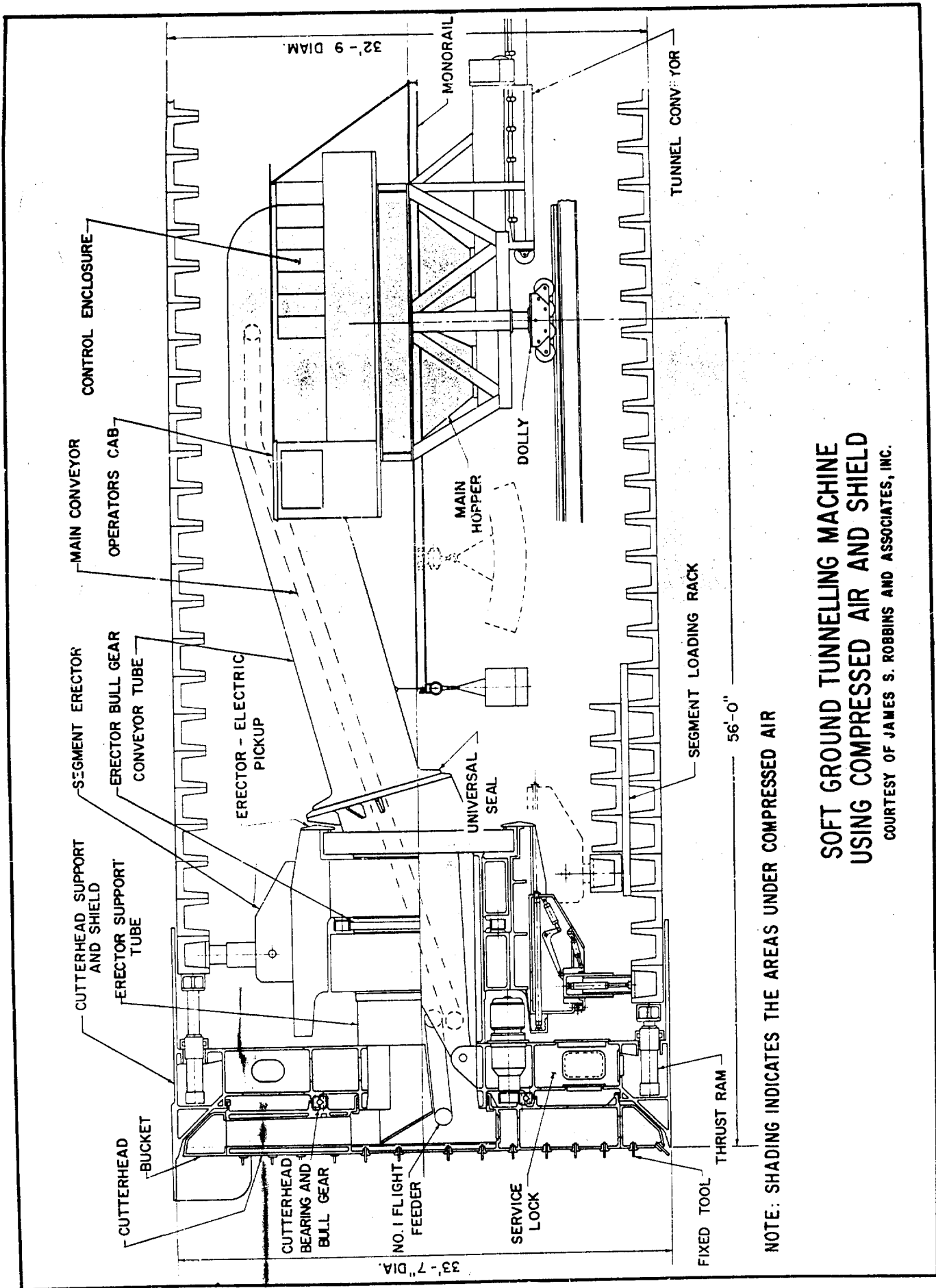
Boring Machine Utilizing Shield and Compressed Air in Poor Ground Conditions

A manufacturer of rock boring machines, James S. Robbins and Associates, Inc., is currently constructing a boring machine for a subway tunnel in Europe which will be located in subsurface materials with extremely soft and wet characteristics. The water table will be located about 85 feet above the tunnel invert, which will require that 35 psi air pressure be imposed on the tunnel face. The tunnel size which will be excavated is nearly 34 feet outside diameter. These conditions, under normal techniques, would impose extremely stringent working conditions in the tunnel.

This device utilizes a conventional shield within which is mounted a boring machine. The machine and shield are monolithically constructed with a bulkhead directly behind the cutting mechanism. The face of the tunnel will be stabilized by compressed air of 35 psi, with this pressurized area extending to the rear by enclosure to include the conveyor from the face and hoppers receiving the muck. All other portions of the tunnel

area will be under free air, allowing normal working conditions for the primary and secondary lining operations. Exhibit 35 indicates a cross section of this device, with the shaded areas showing those portions of the tunnel face, lining and equipment under the additional air pressure. It is expected that this machine will advance at a steady rate of progress of about two and one half feet per hour, with the erection of the lining segments keeping pace.

The economic and construction advantages of driving tunnels under free air have prompted the expenditure of much time and money to develop equipment which would allow this condition in soft ground tunneling requiring compressed air at the face. However, the Robbins machine is the first such device which holds real promise for the achievement of this aim. The test of this equipment, which will be initiated in March 1964, will be watched by soft ground tunnelers throughout the world, and if successful, will certainly represent a major step forward in the art of soft ground tunneling.



NOTE: SHADING INDICATES THE AREAS UNDER COMPRESSED AIR

SOFT GROUND TUNNELLING MACHINE USING COMPRESSED AIR AND SHIELD

COURTESY OF JAMES S. ROBBINS AND ASSOCIATES, INC.

STUDIES OF TUNNELS IN ROCK

General Considerations

The general considerations discussed for "Studies of Tunnels in Earth" are also applicable to the consideration of tunnels in rock. Even in relatively level terrain, depth and functional requirements could cause the proposed shelter to be located in rock. In hilly or mountainous terrain, it would be expected that these shelters might appropriately be tunnel driven into the hill slopes.

The matter of access, as previously discussed, also pertain to rock tunnels. Although side hill drifts would be the preferable means of functional and construction access and would certainly be used wherever the terrain would permit, it is realized that the locations do not predominate where this technique could be utilized. Equally as many, if not more, rock tunnels might have to be initiated from a shaft or inclined drift because of the terrain.

Establishing Soil Conditions, Tunnel Geometrics and Dimensions

The scope of this report limits the studies to median geological conditions, considering neither the best nor the worst circumstances. "Moderately Blocky and Seamy" rock was chosen as the representative classification from the competent rock category as a basis for studies of tunnels in rock. The rock is assumed to be relatively dry; that is, with less than 100 gallons per minute inflow. A horseshoe-shaped tunnel having vertical sides is considered an appropriate section in this material. The roof will normally have to be supported and the vertical walls may also require some lateral support.

The nine foot minimum headroom criteria used to establish the optimum cross section and usable floor area, and the 75 foot depth to the crown used to estimate the loading, as adapted for the earth tunnel studies, apply to the rock tunnel studies.

Four rock tunnel sections were developed and are illustrated on Exhibit 36.

Section F provides an inside base width of 15 feet and a maximum inside height of 12 feet with 12 square feet of usable floor area per lineal foot of tunnel on a single floor.

Section G provides an inside base width of 25 feet and a maximum inside height of 22.5 feet, with approximately 42 square feet of usable floor area per lineal foot of tunnel on two floors.

The inside dimensions for Sections F and G are identical to those for Sections D and E used for earth tunnels in self-supporting ground.

Section H provides for a three-level shelter. The inside tunnel dimensions are 36 feet across the base and 32.5 feet maximum height, and the usable floor area per lineal foot of tunnel is approximately 92 square feet.

Section J provides for approximately 158 square feet of usable floor area per lineal foot of tunnel on four floors. The inside dimensions for this section are 46 feet across the base and 42.5 feet maximum height.

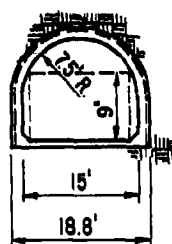
As previously noted, a practical limit of base width in the range of 50 feet has been established for a single bore in rock; therefore, Section J represents the largest section deemed feasible.

Studies of Specific Conditions - Tunnels in Rock

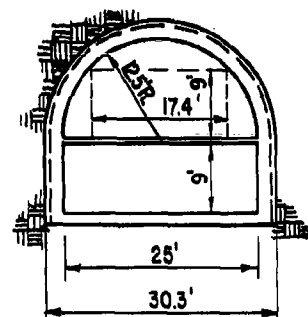
The assumed rock type and the tunnel geometrics were established in the preceding section.

The following assumptions are used in developing the cost estimates for excavating and lining each of the four rock tunnel sections:

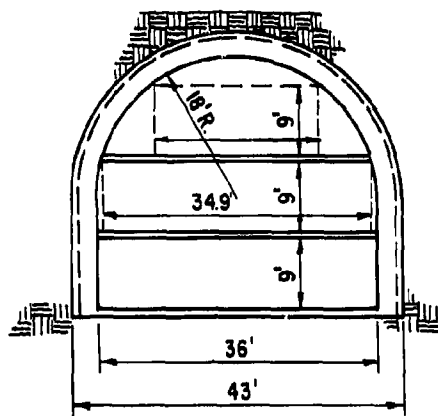
1. Access to the tunnel would be by means of a shaft; the cost of constructing the shaft is not included.
2. The method of mining is by drilling and blasting from a single heading. Sections F and G are advanced full-face, and Sections H and J by top heading and bench.



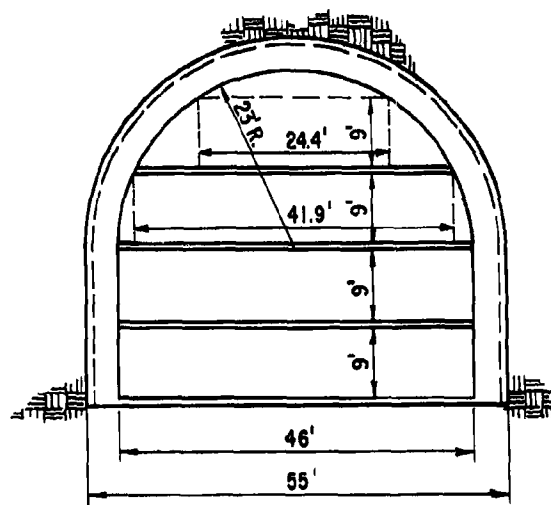
SECTION F



SECTION G



SECTION H



SECTION J

DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS

TUNNELS IN ROCK

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3. The method of mucking is by rail mine cars.
4. The cost of mucking from the heading to hoppers or trucks at the top of the shaft is included; the cost of the hoppers or trucks and the cost of disposing of the muck beyond this point is not included.
5. The contract length is 5500 feet.
6. Labor requirements are based on a three-shift day, six-day week for mining; and on a one-shift day, six-day week for installing the concrete lining.
7. The primary lining is steel ribs with timber lagging, and the permanent lining is reinforced concrete. The lining is predicated on static loading only.
8. The cost of floors required for multi-level usage is not included.
9. Labor, material and equipment costs are based on Chicago area rates and prices in effect as of November 1962.

Estimates were developed from these studies, covering labor, material and equipment. The results are summarized in Table 3 which shows estimated unit costs per lineal foot of tunnel and per square foot of usable floor area.

The discussion under Earth Tunnels pertaining to the possible variation in cost due to contract length and geographical location also applies to tunnels in rock.

TABLE 3

TUNNELS IN ROCK

USABLE AREAS AND LENGTHS

	<u>Section F</u>	<u>Section G</u>	<u>Section H</u>	<u>Section J</u>
	<u>In Square Feet</u>			
Usable Areas of Tunnel Per Linear Foot	12.0	42.4	92.2	158.3
	<u>In Linear Feet</u>			
Length of Tunnel Required to Produce Usable Areas of:				
5,000 Sq. Ft.	420	120	(Negligible)	
50,000 Sq. Ft.	4,160	1,180	460	320
100,000 Sq. Ft.	8,320	2,360	1,080	630
200,000 Sq. Ft.	16,640	4,720	2,160	1,260
300,000 Sq. Ft.	24,960	7,080	3,240	1,890

Estimated Construction Costs
In Moderately Blocky and Seamy Rock
From Study Examples

Per Linear Foot	\$ 680	\$1,500	\$2,160	\$2,860
Per Square Foot of Usable Area	57	35	24	18

TUNNELS IN ROCK

NEW TECHNIQUES

There is a continuing need for improved and new techniques for hard rock tunneling to increase productivity and decrease costs. At present, the use of mechanical methods is still relatively in its infancy, but the results already obtained by equipment such as the Robbins' "Mole" point to greater capabilities for the future, as discussed in previous sections covering these devices.

Exhibits 37 and 39 illustrate three types of these machines. Referring to Exhibit 39, Model 161 (16 feet, one inch in diameter) was used in the Tasmania tunnels, and Model 71 (seven feet, one inch in diameter) was utilized in the driving of an interceptor tunnel in Washington, D.C.

Other types of mechanical hard rock miners are being presently developed, for example, the "Rockmate", Mining and Tunneling Enterprises, Inc., and the "Bootstrap Miner", Alkirk Corporation.

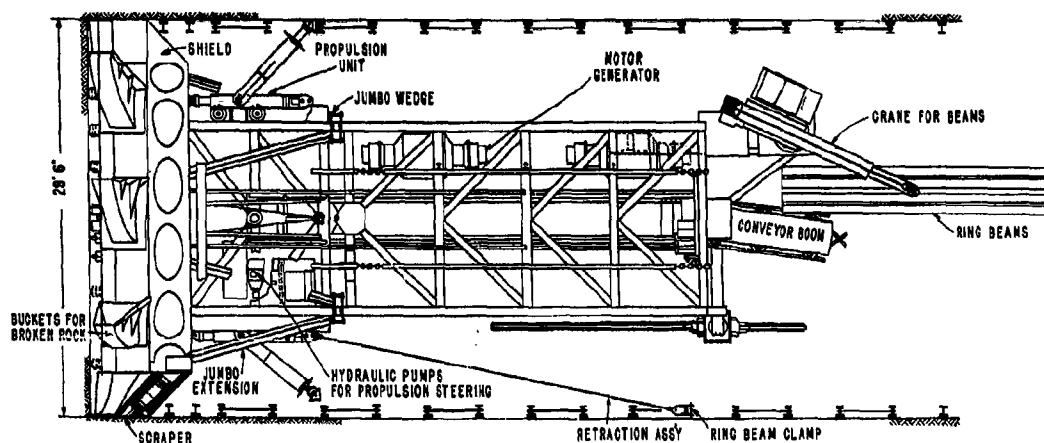
The "Rockmate" machine operates on an entirely different principle than the "Mole", using percussion to chip away the rock face. A battery of compressed air operated jack hammers are mounted in a rotating head. The air used to operate the hammers is also used to exhaust the cuttings from the tunnel. The rig is mounted on crawlers, and is capable of being remotely controlled.

The developer claims that this equipment is capable of mining any shape or size of tunnel in the hardest of rocks. To date, however, the performance data reported covers a prototype machine of two feet in diameter mounting nine cutting hammers, which is said to allow an advance at an average rate of four feet per hour through flint impregnated limestone. This machine is illustrated on Exhibit 38. A machine of three feet in diameter is presently under construction.

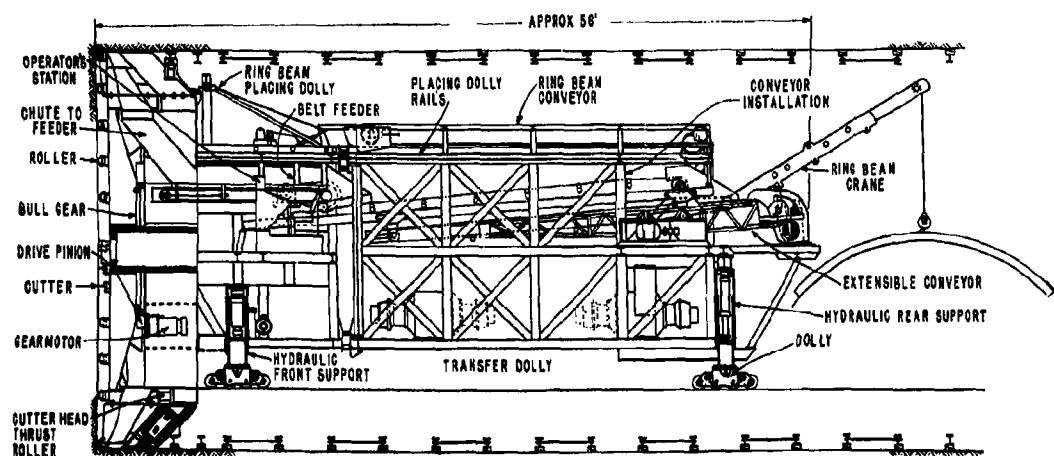
The Alkirk "Bootstrap Miner" acquires its name from its method of developing forward thrust. The principle of fracturing the rock from the face using rolling cutters, similar to that employed

by the "Mole", is employed. Instead of deriving its thrust by pushing or bracing from behind, the Alkirk machine pulls itself against the face. A "pilot" shaft drills its own hole or is inserted in a pre-drilled hole at the center of the face. An expansible anchor located at the end of the "pilot" provides the pull required.

It is claimed that this machine will overcome the drawback of inadequate thrust that has previously limited the ability of mechanical miners to cope with hard rock. Design studies have been made by the manufacturer for an anchor with a holding capacity in excess of one million pounds. The Alkirk miner is illustrated on Exhibit 40.



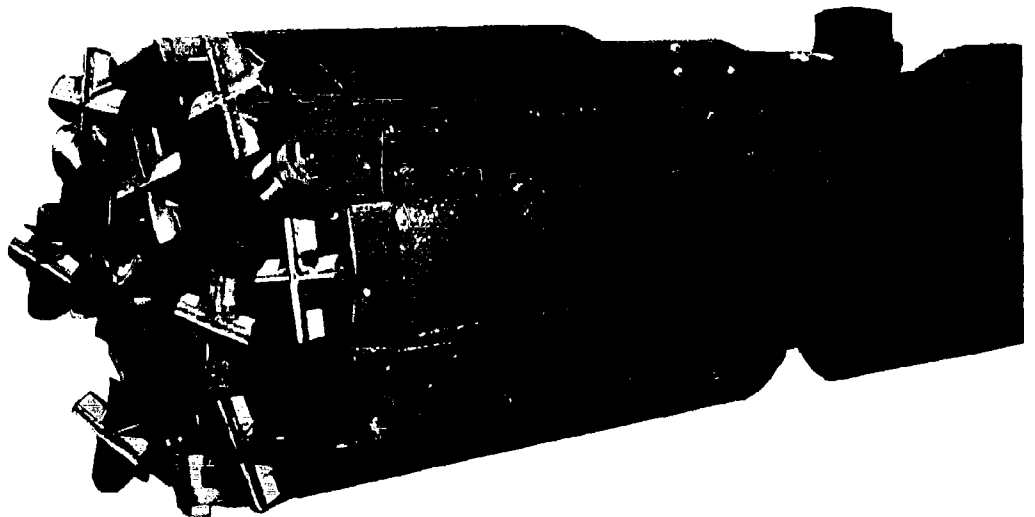
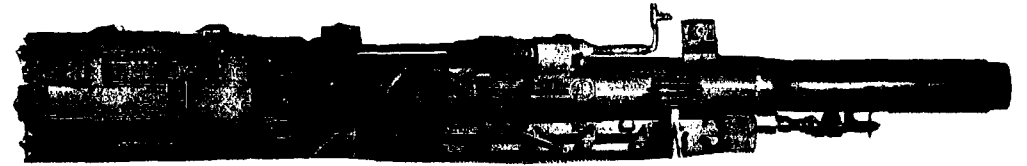
PLAN VIEW



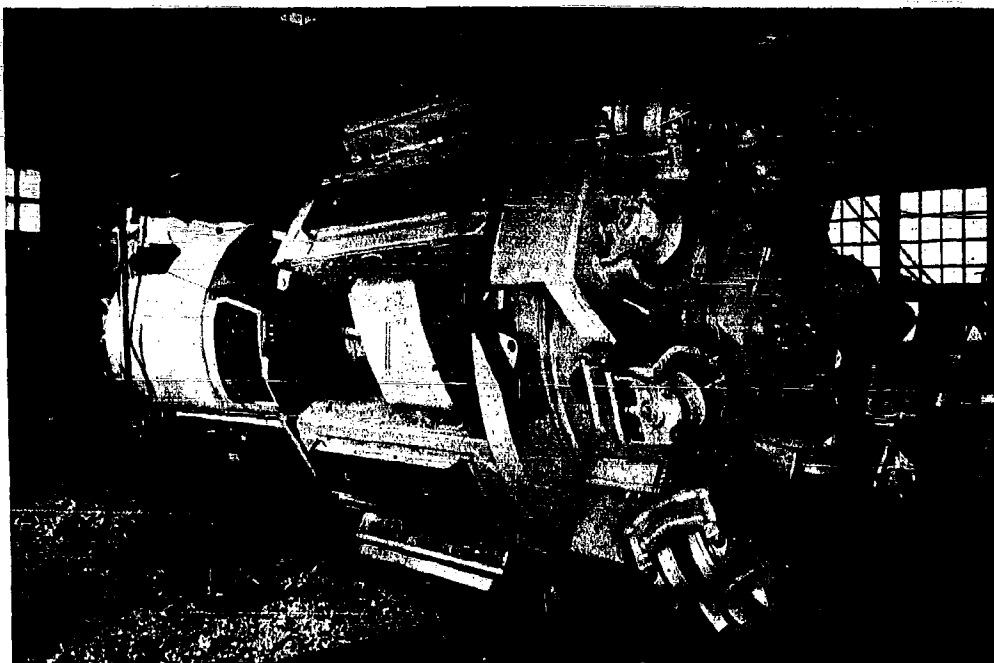
SIDE ELEVATION

ROBBINS ROCK BORER

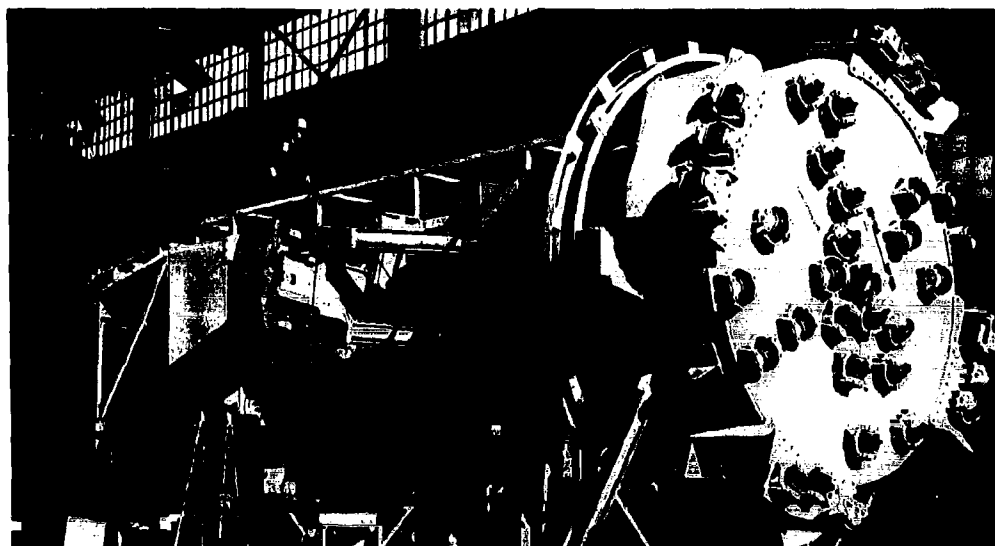
COURTESY OF ENGINEERING AND MINING JOURNAL



ROCKMATE BORING MACHINE
COURTESY OF MINING AND TUNNELLING ENTERPRISES, INC.



MODEL 71
7'-0" DIAMETER

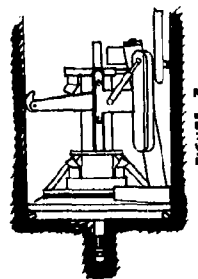
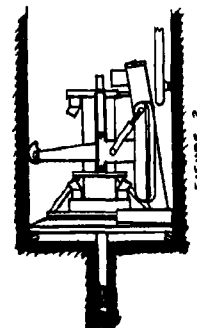
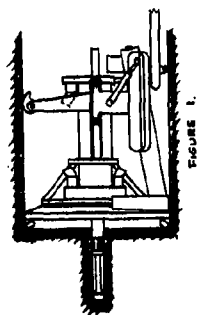
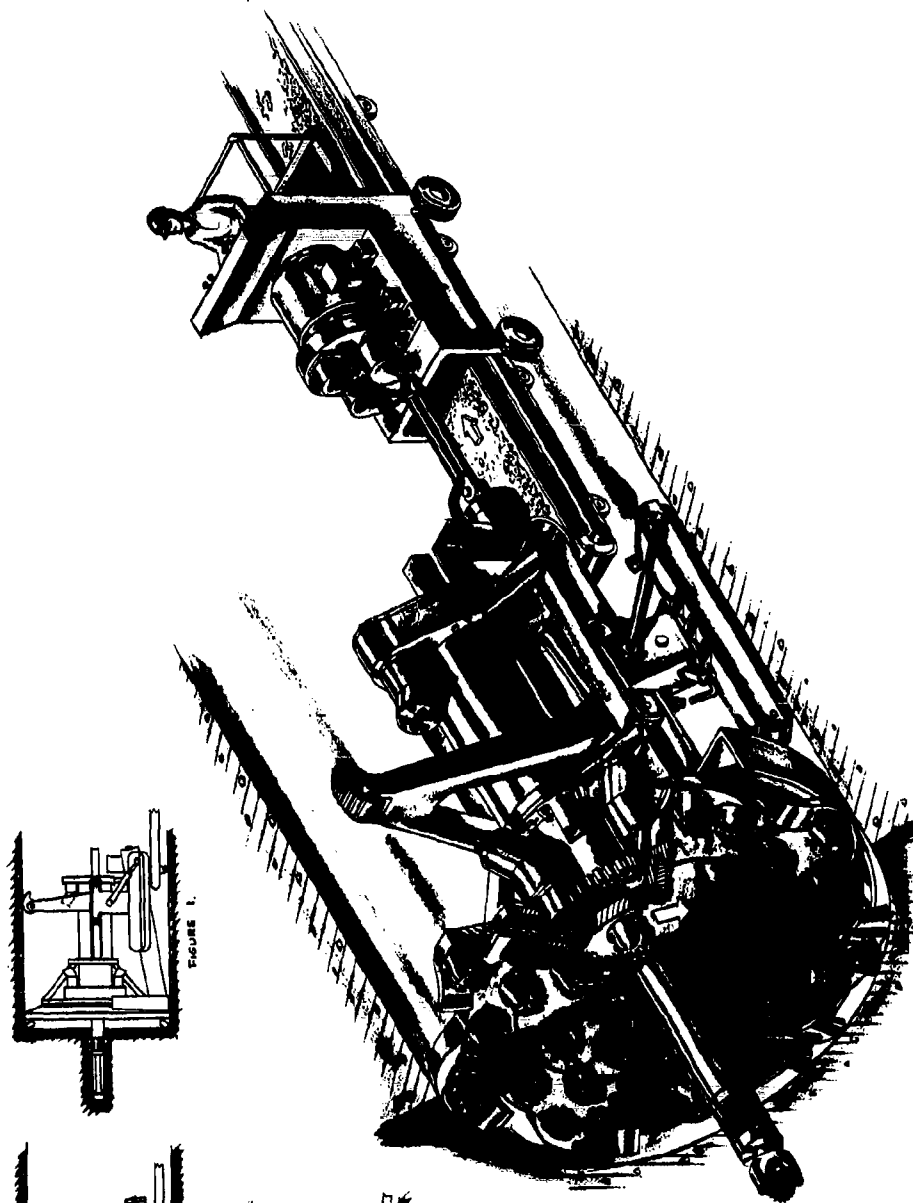


MODEL 161
16'-1" DIAMETER

ROBBINS HARD ROCK BORERS

COURTESY OF ENGINEERING AND MINING JOURNAL

ALKIRK HARD ROCK TUNNELER
COURTESY OF LAWRENCE MACHINE AND MANUFACTURING CO.



FLAME SPALLING OF ROCK

A relatively new technique has been developed, utilizing a gas flame torch process, in the sub-division and removal of intact granite from a New Hampshire quarry. This same process is being used in the surface treatment of granite for architectural finish.

Present Applications of Gas Flame

This process is one of many under operation and development by the Humphreys Corporation, Concord, New Hampshire, all based on the reaction of high temperature-pressure-velocity gas and arc-gas (plasma) flames on various materials and compounds.

The gas flame is produced by the combustion of a mixture of air or oxygen with fuel oil or natural or propane gas under pressure. Practical operations to date in connection with granite removal and treatment have indicated that considerable economies are realized, with satisfactory results, utilizing air and fuel oil, as compared to the alternative mixtures. A conventional air compressor forms the pressurizing unit, operating through a portable automatic control system to furnish the mixture to the combustion chamber at high pressure. Under these conditions, the ignition process is complete, thereby producing high temperature-velocity gases, enabling high heat transfer to the material under treatment.

The sub-division of intact granite in the quarry operation, which was observed, is carried out by the manual application of a torch or pipe burner to the rock surface, producing a trench or channel "cut" about four inches in width. Air is delivered to the combustion chamber at 50 pounds per square inch gauge pressure at a rate of 200 cubic feet per minute. Fuel oil is delivered at 15 pounds per square inch gauge pressure at a rate of 10 gallons per hour. This mixture, when ignited, produces a burning gas at temperatures in the range of 3500 degrees Fahrenheit. The flame, when played on the granite surface, produces rapid local expansion, through heat transfer, with "popping" or spalling of granules of rock from the main body of the deposit. It is this eroding action which produces the "cut" or channel.

Exhibit 41 illustrates the general apparatus involved in the operation and shows the operator applying the torch flame to the rock surface. The fuel oil tank is mounted on the block to the left with the control gear immediately to the front. The air compressor is not shown. A closer view shows the torch emitting the flame on to the granite surface. A separate supply of water is utilized to wash the spalled granules to a lower level for later removal. It is noted that, in the application shown, only one operator is required.

The technique is being used here to form expansion space around large blocks of granite, so that they may be further subdivided into portable sections by normal quarrying methods.

The depth of cut is limited only by the length of pipe torch which can be efficiently handled by the operator. It may be noted from the pictures that the resultant surfaces in the background are relatively smooth, considering the manual operation involved. Some performance figures relating to the torch channeling techniques are as follows:

1. The rates of air and fuel oil at the pressures quoted above will produce from 15 to 30 square feet of exposed area per hour by the channel cut method.
2. Using these same specifics, a gallon of fuel oil will enable removal of from 5 to 10 cubic feet per hour.

Comprehensive cost data relating to the spalling of granite by this technique are not available. However, from the production rates quoted, we have prepared a cost study for the spalling of a unit area of granite based on the following assumptions:

1. Fifteen square feet of rock per hour would be exposed by a channel cut four inches in width.
2. One prime operator would be required together with a proportionate charge for supporting supervisory and labor time.



SEE ENLARGEMENT BELOW

GENERAL VIEW OF APPARATUS



TORCH

FLAME

CHANNEL

VIEW OF TORCH FLAME
PRODUCING CHANNEL SPALLING

DEEP CIVIL DEFENSE SHELTERS
IN URBAN AREAS
FLAME SPALLING OF ROCK

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3. All material and labor costs would be included as direct costs.
4. Equipment, including the necessary control gear, would be included as a part of the indirect cost, based on amortization over the usable life of this equipment.

Utilizing these assumptions, we estimate that the cost of exposing rock surface by the spalling of a four inch channel width would be in the range of \$0.75 per square foot.

It was quoted that this technique has been in use at this quarry for more than three years, with the operation still under study for determination of the optimum flame temperature and application methods. One point of study involves the establishment of the maximum flame temperature possible without fusing the rock and thus forming an insulating barrier to the optimum heat transfer.

Surface treatment of granite slabs was being performed by passing a multi-orifice burner, 12 inches in length, over the surface of rock slab at an automatically controlled rate and constant elevation, forming a stippled surface. At the same rates and quantities of air and fuel quoted before, the same number of cubic feet of rock per hour could be removed. When translated into removal of an average of 1/8 inch of depth of rock surface, from 500 to 1000 square feet per hour could be so treated.

A third technique, not yet demonstrated, would involve the removal of rock in a local area to considerable depth, for example, a hole 3 to 6 inches in diameter and 20 or more feet long, corresponding to a conventionally drilled blast hole. It is quoted that the operational specifics, mentioned in connection with the channel technique, would enable the production of a 3 inch hole at a rate of 50 feet per hour.

Possible Applications

A review of the capabilities of this technique disclosed that it has not been used on rock other than intact granite and consequently no comparative factual data are available with reference to its application to other rock types. The dependence of the technique on the spalling action through heat transfer and local expansion would appear to limit it to the classification of rocks possessing the least amount of joints, faults, fissures and disintegrated matter. It was conjectured that in these more intact rocks, of the types adaptable to the heat transfer-spalling action, that a mass removal attack utilizing jumbo mounted banks of burners played on the rock face or faces would provide a uniform rate of progress, worthy of economic comparison with conventional methods. However, if zones of rock would be encountered that would not be adaptable to this technique, other more conventional means would have to be resorted to in order to remove the materials in these areas.

One point of interest is worthy of mention involving the use of this technique. It was experienced that the torch burner develops a very high noise level in operation, requiring the operator to wear ear protective devices. It is assumed that the noise level, unless unforeseen improvements would be made, would increase correspondingly in a multi-burner application. Any such mode of attack, particularly in the driving of a tunnel, might require complete automation of the equipment with remote control operational features.

Arc-Gas (Plasma) Flames

This firm is also actively engaged in research and application relative to the use of extremely high temperature flames ranging up to 50,000 degrees Fahrenheit. These are produced by the release of burning gases of oxygen, helium, nitrogen or air in a combustion chamber, being acted on at that point by high frequency electrical power. The introduction of this energy changes the gas characteristics to a plasma, super-energizing the outflow and producing temperature ranges not otherwise feasible. It was quoted that the simplicity and efficiency of this process is unique, providing operation with a minimum of maintenance costs. No information was offered regarding its application to the field of excavation, since no experimentation has been conducted in this area to date. However, it was conjectured

that flame temperatures in the upper ranges possible in this process would reduce solids, such as rock, to a molten state. The application of this technique must await further research into the technological problems which would be presented thereby, involving method of attack, disposal and related features.

Conclusion

As stated before, this technique has not been applied to the mass removal of materials from open or tunnel excavations. Present trends in hard rock tunneling toward the use of larger, more costly equipment to increase production nearly restricts the application of that equipment to projects of considerable magnitude. Therefore, it is worthy of consideration that potential exists in the flame spalling technique from the standpoint of portability and simplicity of equipment for utilization in removal of the more intact igneous rocks. It would remain to be seen, through more appropriate applications, if these apparent benefits might be also contributory to economies and production rates comparable to techniques presently accepted and used. The further development of this technique should be encouraged with those considerations in mind.

ANALYSIS OF COST STUDIES

Open Cut Excavations

As discussed before, subsurface conditions exert a major influence on approach, techniques and the resultant costs of excavating in any material.

To illustrate the cost aspect, reference is made to the study examples presented herein. Those cases where the excavation could be carried to the full 150 feet of depth are regarded as feasible excavations in the sense that they could be performed. The studies for the largest size of excavation to this depth indicate a range in cost factors of from a minimum of 10 to a maximum of 25, or a 150 percent increase, brought about entirely by a change in the subsurface conditions. Likewise a range in cost factor of from 4 to 10, or the same differential percentage-wise, is exhibited for this largest size carried to a depth of 100 feet. At the 50 foot depth, the same differential is indicated.

Studies of the smallest size of bottom areas, which are more nearly comparable to shaft excavations than open cut, exhibit an even greater range of cost factors. At the maximum depth, the factors range from 23 to 80 for the cases feasible for excavation at that level, with the maximum being 250 percent over the minimum. It should be noted that in most instances the cost factors for the smallest excavation show a decided increase over those for the other sizes at all depths.

These studies also show how the unit cost factor decreases as the size of bottom area of these excavations increases.

The studies for Locations D, E, H and I, which indicate physical limitation of depth of excavation, also show, by their rapidly increasing cost factors with respect to depth, that the economies of excavation form another limitation in these unfavorable subsurface conditions.

Numerous similar comparisons may be made to point up the effect of the different assumed conditions on the cost factors. Other conditions could be assumed or, in actuality, be encountered which, although practically feasible, would increase even further this range in costs.

Tunnels in Earth

As could be expected, the costs per square foot of usable area for tunnels in earth are at a maximum in the more unfavorable soil conditions, decreasing as these conditions improve, with the single bore, single level tunnel costs, Sections A and D, exhibiting the greatest range, from \$124 to \$56 per square foot.

It was indicated that the costs per square foot of usable areas decrease with the increase in the size of the bore or section.

Section B, double interconnected bore, single level, and Section C, single bore, double level, yield approximately the same usable areas per linear foot. It is also of interest to note the close comparison in costs per unit of usable area between these sections. Either of the sections could produce more square footage per linear foot compared to the smaller single bore, but would, under poor conditions, present more problems during construction.

The enlargement of the twin bores to form a single tunnel area, in Section B, would complicate construction, particularly in poorer ground conditions. However, consideration of this section is warranted by its potentiality for multipurpose in conjunction with underground transport usage.

The maximum size characteristics of Section C could produce difficulties with respect to differential hydrostatic pressures in saturated ground conditions. This problem could be alleviated in the future, if the principle of the Robbins soft ground tunneling machine is proved successful.

Tunnels in Rock

These studies indicate results similar to earth tunnels in that as the tunnel section increases in size, the unit costs per square foot of usable area decrease, ranging from \$57 for the single level section to \$18 for the four-level section.

The portion of this report dealing with geology of rock indicates the wide variation in conditions which could be encountered in driving

rock tunnels. The quantitative effect of these conditions on cost is shown conclusively in Appendix C of Bulletin No. 78 "Investigation of Alternative Aqueduct Systems to serve Southern California", prepared by the California Department of Water Resources. This appendix, "Procedure for Estimating Costs of Tunnel Construction", presents the results of comprehensive analysis of conditions, techniques, progress and costs for 99 tunnel projects utilized for studies determining excavation and lining costs for tunnels. These tunnel studies include circular and horseshoe shapes in a range of diameters from 9 to 28 feet located in eight classifications of rock conditions. This document is highly recommended as a reference for those responsible for detailed studies of rock tunnel shelters, insofar as the included data would apply to the sizes under consideration;

A few excerpts from the tabular data included therein illustrate the effect of rock and ground water conditions on excavation costs. The cost of excavating a tunnel of unlined diameter of 12 feet would vary per linear foot from \$206 in stratified rock to \$315 in completely crushed rock, both under dry conditions. In wet conditions, this cost would increase sharply from \$574 in competent rock to \$833 in completely crushed rock. The complete range indicates a cost increase of 300 percent from the best to the poorest conditions studied.

An unlined tunnel of 24 feet diameter would exhibit an even greater range. The excavation cost per linear foot would vary from \$464 in dry stratified rock to \$3,628 in wet crushed rock, an increase of 700 percent.

These results point up the importance of adequate investigation and study preceding the cost and design analyses for any tunnel project.

CONCLUSIONS

The relatively limited time and parameters governing these studies, coupled with the extremely broad aspects of excavation included in the scope, preclude the development of hard and fast conclusions relative to the subject. Certain facts emerge, however, for consideration by the individuals and agencies responsible for further investigation of deep shelter feasibility.

Site Investigation

Of all the aspects of excavation experienced or encountered in the past or as a result of these studies, the one most important phase which enables success or failure, timely or delayed progress and budgeted or abnormal extra costs, is the adequate and properly designed program of investigation of subsurface conditions. There is no substitute for this phase, which at best provides only a basis for the exercise of sound and qualified judgment. Instances occur when the demand for accelerated programs make it difficult to apportion sufficient time for proper investigation. It is the engineer's responsibility to resist efforts to shortcut this phase, since he knows well that any subsurface problems will eventually have to be dealt with.

Equipment and Techniques

Rapid progress is currently being made in certain areas of excavation procedures with promise for even greater achievements in the future. The more notable of these is represented by excavating equipment, such as the bucket wheel excavator and the earth and rock tunneling machines. It is believed that as pioneering use of this equipment continues, refinements and added capabilities will allow their application to a yet wider range of materials resulting in considerable economic benefit to all.

The stabilization of water bearing materials, another major aspect of excavation, is also being given close attention. Companies at interest are refining procedures, developing equipment and controllable admixtures to bring about the lessening of the problems introduced by water in the excavation.

If a program for provision of deep shelters would be progressed to an active planning and design stage, it would be in the national interests to encourage and foster the development of these and related techniques. The potentialities for reduction in excavation time, together with the alleviation of difficult methods of operation, producing greater economies than possible at present, would warrant the attention and assistance of the responsible agencies in advancing this development.

Application of the Cost Studies

The range of subsurface conditions considered in these studies is limited, considering the possible combinations of conditions to be encountered in this country. However, the factors and costs derived herein could properly be used in the development of excavation costs in locations presenting close similarity to the governing conditions covered in the individual studies. It would be possible to extend these studies through additional parameters and conditions to further enlarge the range of costs of excavation, dependent only on the time and money available measured against the requirements of intended usage.

The necessity may arise for a general, nationwide, centrally conducted assessment of the costs of excavation for shelter areas, precluding, by the enormity of the task, a detailed site-by-site analysis. In this event, it is believed that the data contained herein, coupled with such other information as may be pertinent, such as the California rock tunnel studies and supported by the careful exercise of judgment, could allow the preparation of a general estimate of such costs within a tolerable degree of accuracy. It is not believed, however, that the data contained herein would apply except in a general sense to a site-by-site analysis, except where conditions closely fit the situations assumed herein.

Availability of Data

From the experience gained in these studies, it appears that there could be industry-wide improvement in the collection, collation and dissemination of recent data pertinent to the general area of excavation. Several reference works have been published recently on the

subject of excavation, which improve greatly the availability of data relating to normal practices for common benefit. Data relating to recent advances in the art are only available through engineering journals, with only minor coverage on methods of interest derived from foreign innovations. It is possible that the new developments in machine storage of information will provide an easy means to make more data available to the general engineering and construction profession relative to all phases of excavation in all areas of the world.

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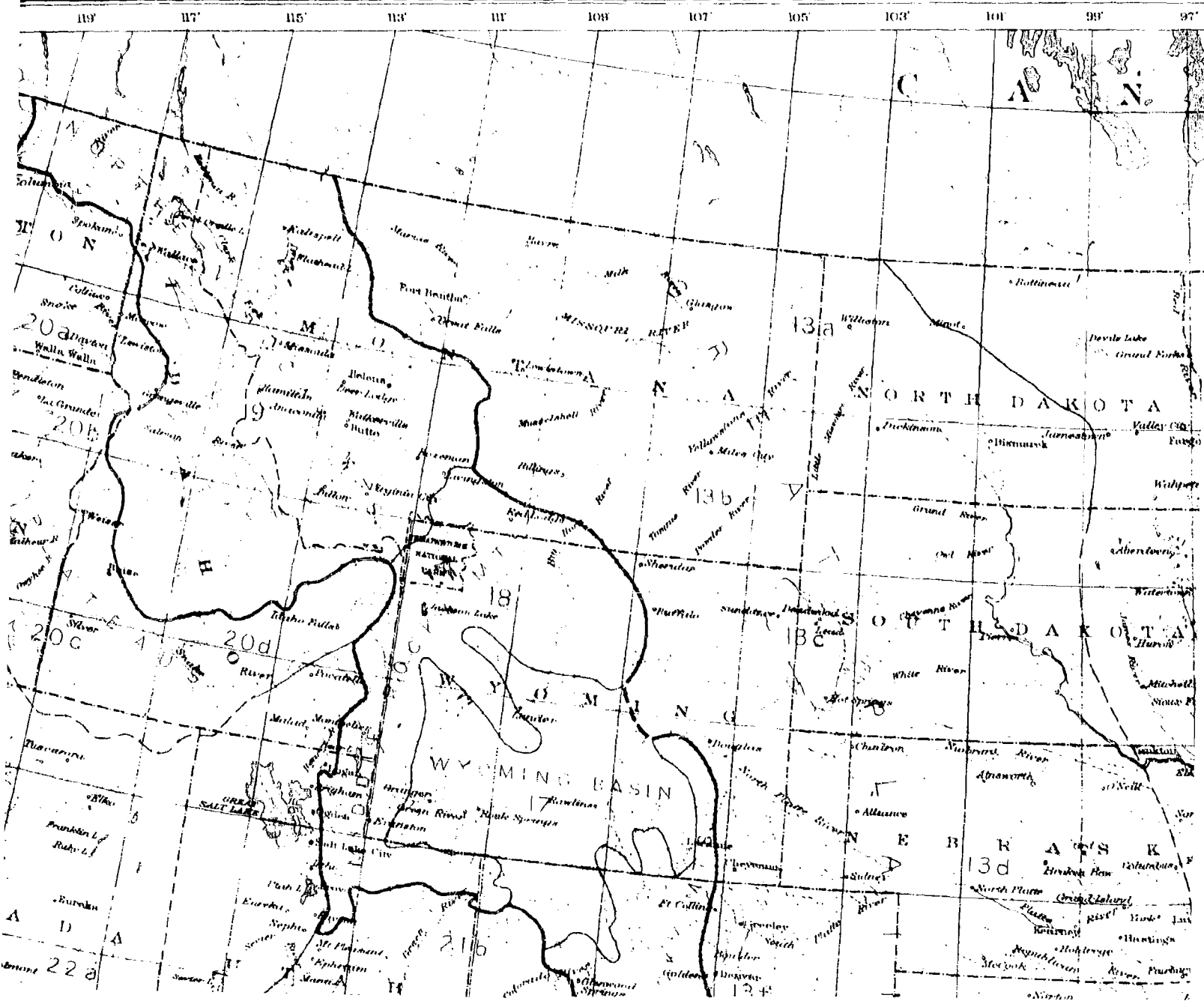
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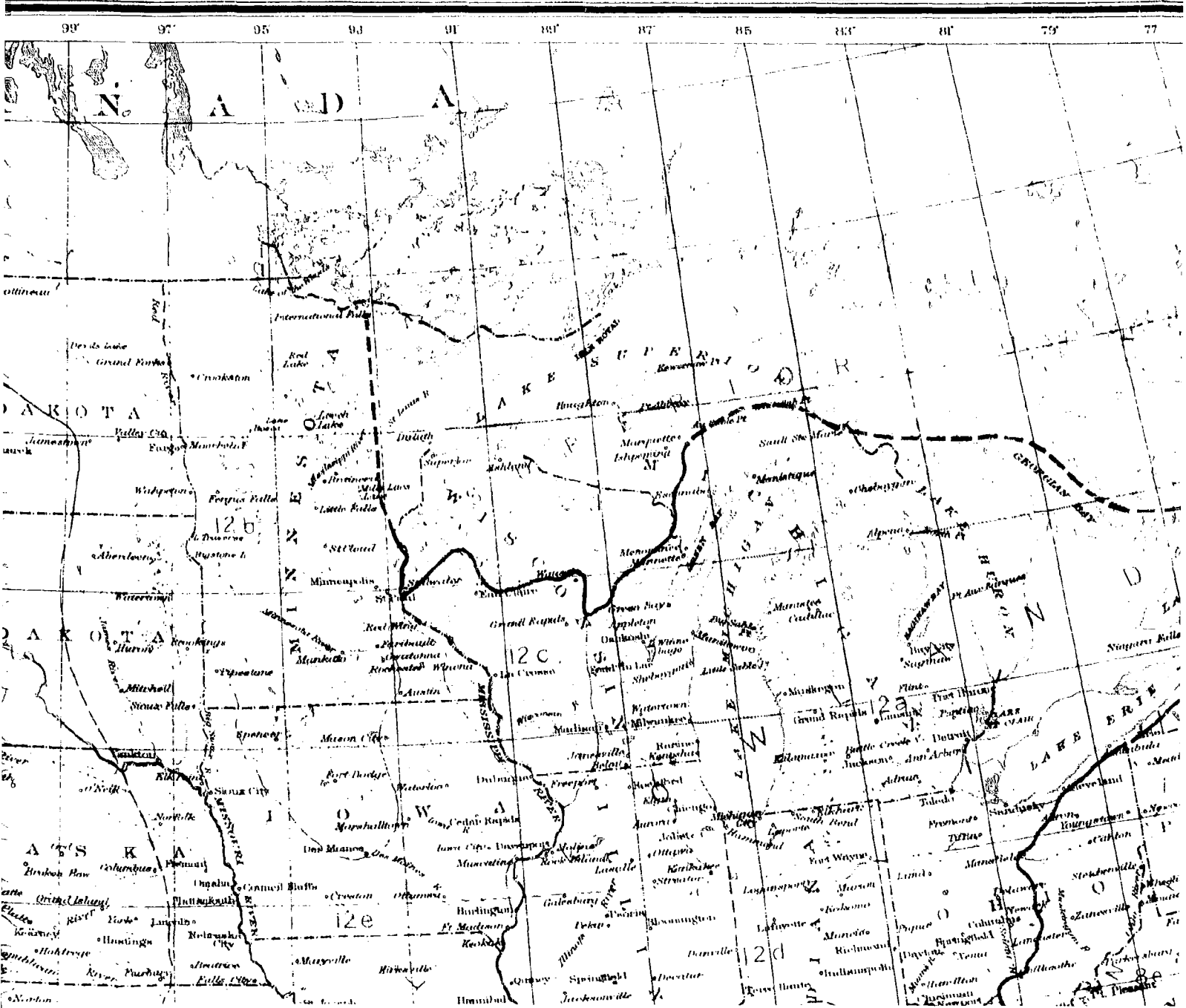
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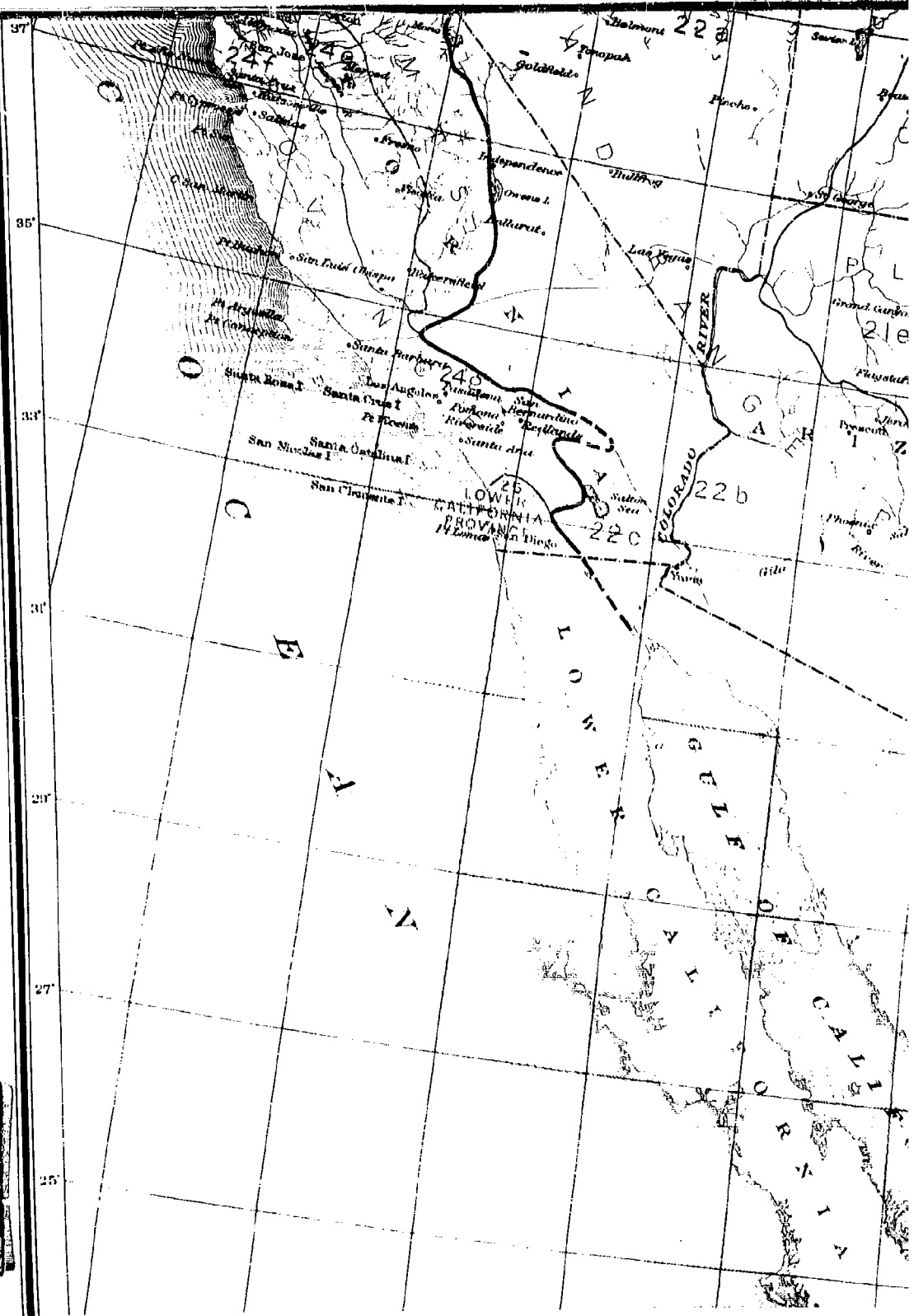
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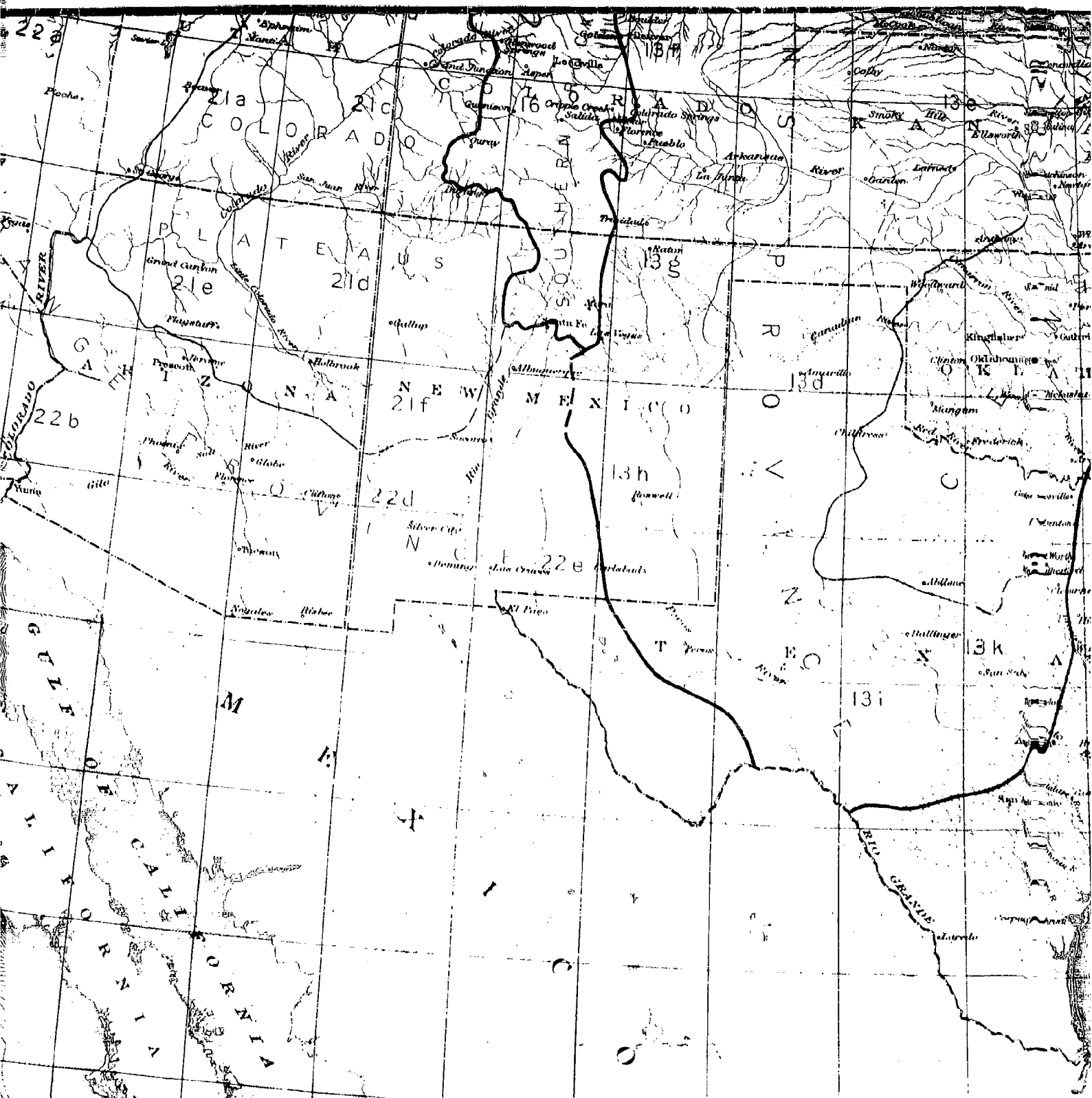
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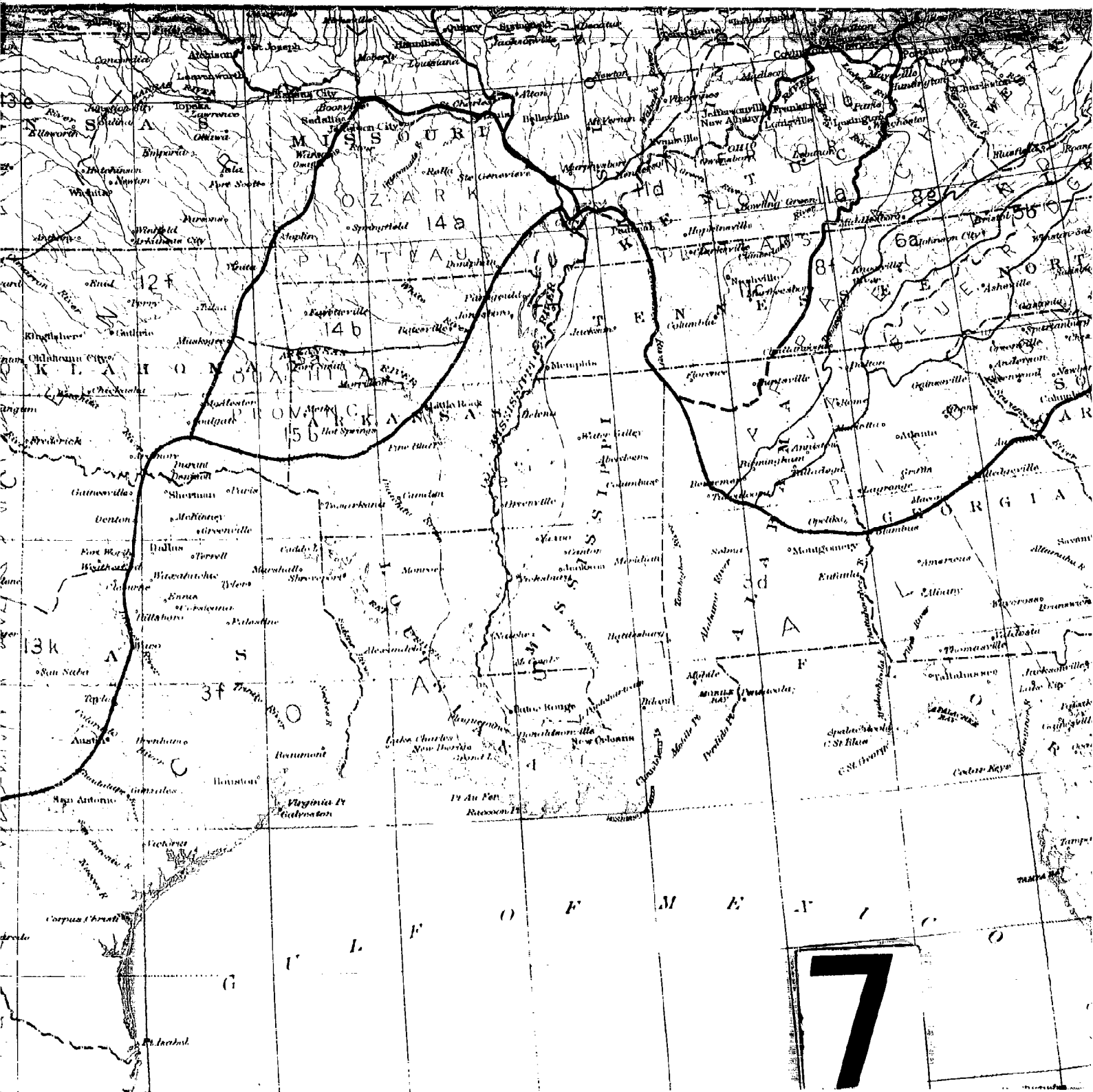


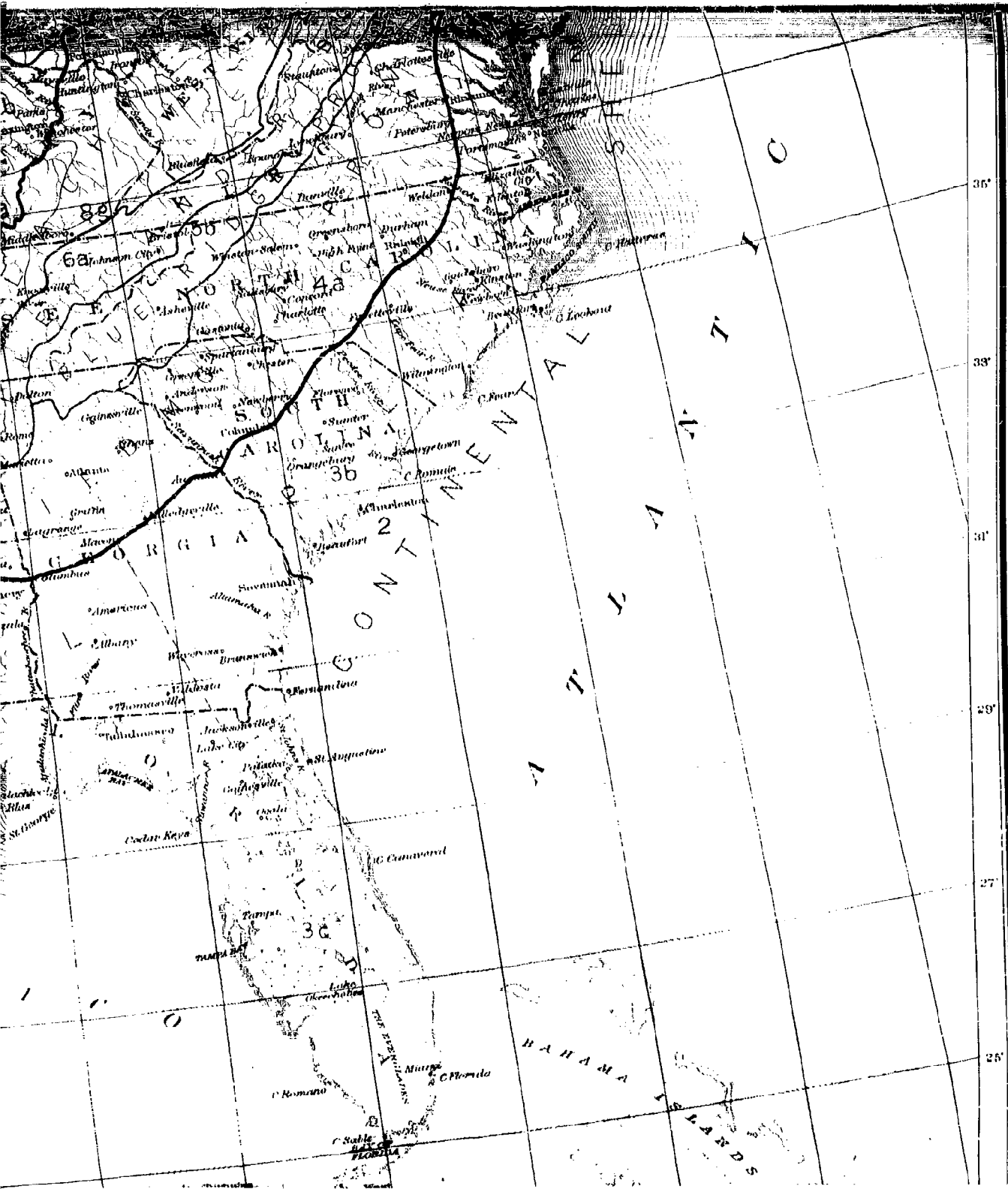


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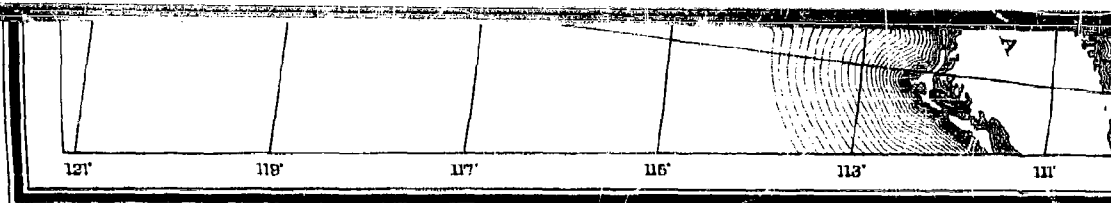






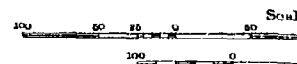
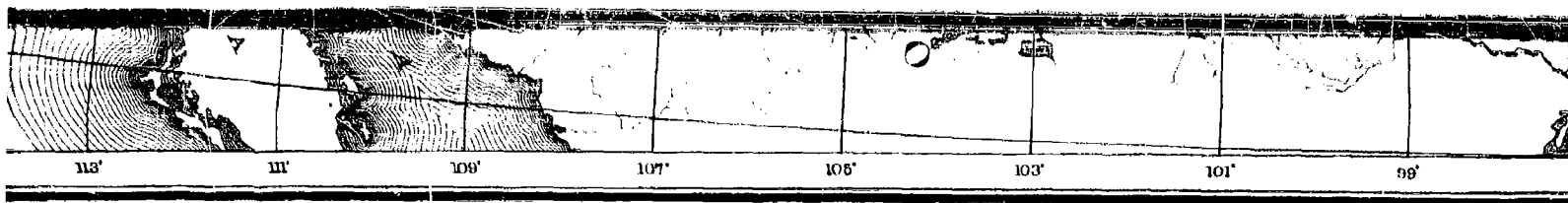


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MAJOR DIVISION	PROVINCE	SECTION	CHARA
Laurentian Upland	1. Superior Upland	1.	Submaturely dissected, recent complex structure.
	2. Continental Shelf	2.	Sloping submarine plain of se
Atlantic Plain	3. Coastal Plain	a. Embayed section	3a. Submaturely dissected and pa
		b. Sea Island section	3b. Young to mature terraced con
		c. Floridian section	3c. Young marine plain, with sar
		d. East Gulf Coastal Plain	3d. Young to mature belted coast
		e. Mississippi Alluvial Plain	3e. Flood plain and delta.
		f. West Gulf Coastal Plain	3f. Young grading inland to mat
	4. Piedmont province	a. Piedmont Upland	4a. Submaturely dissected penepl relief
		b. Piedmont Lowlands	4b. Less uplifted peneplain on wa
	5. Blue Ridge province	a. Northern section	5a. Maturely dissected mountain
		b. Southern section	5b. Subdued mountains of disorc
	6. Valley and Ridge province	a. Tennessee section	6a. Second-cycle mountains of predominately over even-er
		b. Middle section	6b. The same, but even-ersted east side.
		c. Hudson Valley	6c. Glaciated peneplain on weak
	7. St. Lawrence Valley	a. Champlain section	7a. Rolling lowland, glaciated; i
		b. Northern section	7b. Young marine plain with loc
Appalachian High-lands	8. Appalachian Plateaus	a. Mohawk section	8a. Maturely dissected glaciated
		b. Catskill section	8b. Maturely dissected plateau (glaciated).
		c. Southern New York section	8c. Mature glaciated plateau of
		d. Allegheny Mountain section	8d. Mature plateau of strong r folds.
		e. Kanawha section	8e. Mature plateau of fine textu
		f. Cumberland Plateau section	8f. Submaturely dissected plat
		g. Cumberland Mountain section	8g. Higher mature plateau and tion.
	9. New England province	a. Seaboard Lowland section	9a. Peneplains below 500 feet p nocks.
		b. New England Upland section	9b. Dissected and glaciated pen nocks.
		c. White Mountain section	9c. Subdued glaciated mountai
		d. Green Mountain section	9d. Linear ranges of subdued a
		e. Taconic section	9e. Maturely dissected and gl folded strata.
	10. Adirondack province		10. Subdued mountains and di

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CHARACTERISTICS^a

1. Submaturely dissected, recently glaciated peneplain on crystalline rocks of complex structure.
2. Sloping submarine plain of sedimentation.
- 3a. Submaturely dissected and partly submerged, terraced coastal plain.
- 3b. Young to mature terraced coastal plain with submerged border.
- 3c. Young marine plain, with sand hills, swamps, sinks, and lakes.
- 3d. Young to mature belted coastal plain.
- 3e. Flood plain and delta.
- 3f. Young grading inland to mature coastal plain.
- 4a. Submaturely dissected peneplain on disordered resistant rocks; moderate relief^b.
- 4b. Less uplifted peneplain on weak strata; residual ridges on strong rocks.
- 5a. Maturely dissected mountains of crystalline rocks; accordant altitudes.
- 5b. Subdued mountains of disordered crystalline rocks.
- 6a. Second-cycle mountains of folded strong and weak strata; valley belts predominate over even-crested ridges.
- 6b. The same, but even-crested ridges predominate over valleys except on east side.
- 6c. Glaciated peneplain on weak folded strata.
- 7a. Rolling lowland, glaciated; in part covered by young marine plain.
- 7b. Young marine plain with local rock hills.
- 8a. Maturely dissected glaciated plateau; varied relief and diverse altitudes.
- 8b. Maturely dissected plateau of mountainous relief and coarse texture (glaciated).
- 8c. Mature glaciated plateau of moderate relief.
- 8d. Mature plateau of strong relief; some mountains due to erosion of open folds.
- 8e. Mature plateau of fine texture; moderate to strong relief.
- 8f. Submaturely dissected plateau of moderate to strong relief.
- 8g. Higher mature plateau and mountain ridges on eroded open folds.
- 9a. Peneplains below 500 feet postmaturely eroded and glaciated; few monad-nocks.
- 9b. Dissected and glaciated peneplains on complex structural features; monad-nocks.
- 9c. Subdued glaciated mountain masses of crystalline rocks.
- 9d. Linear ranges of subdued and glaciated mountains and residual plateaus.
- 9e. Maturely dissected and glaciated mountains and peneplain on resistant folded strata.
10. Subdued mountains and dissected peneplain, glaciated.

MAJOR DIVISION

PROVINCE

SECTION

Interior Plains	11. Interior Low Plateaus	a. Highland Rim section
		b. Lexington Plain
		c. Nashville Basin
		d. Possible western section (not delimited).
	12. Central Lowland	a. Eastern lake section
		b. Western lake section
		c. Wisconsin Driftless section
		d. Till Plains
	13. Great Plains province	e. Dissected Till Plains
		f. Osage Plains
		g. Missouri Plateau, glaciated
		h. Missouri Plateau, unglaciated
		i. Black Hills
j. High Plains		
k. Plains Border		
l. Colorado Piedmont		
m. Raton section		
Interior Highlands	14. Ozark Plateaus	a. Springfield-Salem plateau
		b. Boston "Mountains"
	15. Ouachita province	a. Arkansas Valley
		b. Ouachita Mountains
	Rocky Mountain System	16. Southern Rocky Mountains
17. Wyoming Basin		
18. Middle Rocky Mountains		
19. Northern Rocky Mountains		

^aPrepared by Nevin M. Fennem

^bDegrees of relief are herein specified. As used here *high* relief is measured in hundreds of feet. *Strong* relief with a wide latitude on both sides.

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Scale 1:100,000
1 inch = 1 mile
1 centimeter = 100 meters

CHARACTERISTICS

- 11a. Young to mature plateau of moderate relief.
- 11b. Mature to old plain on weak rocks, trenched by main rivers.
- 11c. Mature to old plain on weak rocks, slightly uplifted and moderately dissected.
- not 11d. Low, maturely dissected plateau with silt-filled valleys.
- 12a. Maturely dissected and glaciated cuestas and lowlands; moraines, lakes, and lacustrine plains.
- 12b. Young glaciated plain, moraines, lakes, and lacustrine plains.
- on 12c. Maturely dissected plateau and lowland invaded by glacial outwash. (Margin of old eroded drift included.)
- 12d. Young till plains, morainic topography rare, no lakes.
- 12e. Submaturely to maturely dissected till plains.
- 12f. Old scarped plains beveling faintly inclined strata; main streams intrenched.
- ed 13a. Glaciated old plateaus; isolated mountains.
- ated 13b. Old plateau, terrace lands; local badlands; isolated mountains.
- 13c. Maturely dissected domed mountains.
- 13d. Broad intervalley remnants of smooth fluvial plains.
- 13e. Submaturely to maturely dissected plateau.
- 13f. Late mature to old elevated plain.
- 13g. Trenched peneplain surmounted by dissected, lava-capped plateaus and buttes.
- 13h. Late mature to old plain.
- 13i. Young plateau with mature margin of moderate to strong relief.
- 13k. Plateau in maturity and later stages of erosion.
- ous 14a. Submature to mature plateaus.
- 14b. Submature to mature plateau of strong relief.
- 15a. Gently folded strong and weak strata; peneplain with residual ridges.
- 15b. Second-cycle mountains of folded strong and weak strata.
- 16. Complex mountains of various types; intermont basins.
- 17. Elevated plains in various stages of erosion; isolated low mountains.
- 18. Complex mountains, mainly anticlinal ranges; intermont basins.
- 19. Deeply dissected mountain uplands, not anticlinal ranges; intermont basins.

MAJOR DIVISION

PROVINCE

Intermontane Plateaus

20. Columbia Plateaus

- a. Walla Walla
- b. Blue Mountains
- c. Payette
- d. Snake River
- e. Harney

21. Colorado Plateaus

- a. High Plateau
- b. Uinta Mountains
- c. Canyon
- d. Navajo
- e. Grand Canyon
- f. Datil section

22. Basin and Range province

- a. Great Basin
- b. Sonoran
- c. Salton River
- d. Mexican
- e. Sacramento

23. Cascade-Sierra Mountains

- a. Northern
- b. Middle
- c. Southern
- d. Sierra Nevada

Pacific Mountain System

24. Pacific Border province

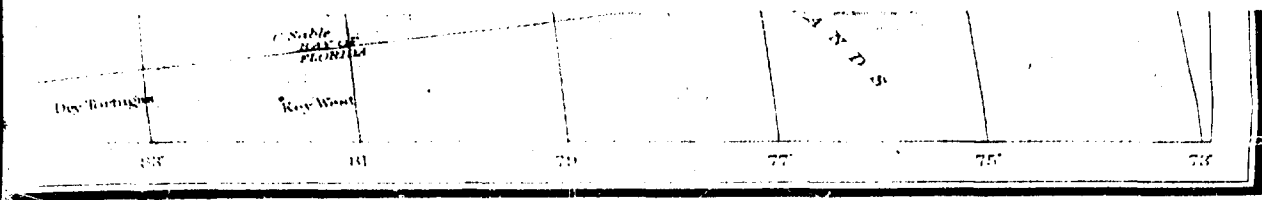
- a. Puget Trench
- b. Olympic
- c. Oregon
- d. Klamath
- e. California
- f. California
- g. Los Angeles

25. Lower Californian province

- a. Los Angeles

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man and Douglas W. Johnson
taken of as low, moderate, strong, and high.
sured in thousands of feet, moderate relief
of may be anything approaching 1,000 feet



UNITED STATES GEOLOGICAL SURVEY WASHINGTON D. C. 1962

Scale 1:500,000

Edition of 1946

SECTION	CHARACTERISTICS
a. Walla Walla Plateau	20a. Rolling plateau with young incised valleys.
b. Blue Mountain section	20b. Complex mountains and dissected volcanic plateaus.
c. Payette section	20c. Young plateaus of prevailingly weak rocks; broad alluvial terraces. (Applies to northern part only.)
d. Snake River Plain	20d. Young lava plateau.
e. Harney section	20e. Young lava plateau; features of recent volcanism; ineffective drainage.
a. High Plateaus of Utah	21a. High block plateaus, in part lava-capped; terraced plateaus on south side.
b. Uinta Basin	21b. Dissected plateau; strong relief.
c. Canyon Lands	21c. Young to mature canyoned plateaus; high relief.
d. Navajo section	21d. Young plateaus; smaller relief than 21c, into which it grades.
e. Grand Canyon section	21e. High block plateaus, trenched by Grand Canyon.
f. Datil section	21f. Lava flows entire or in remnants; volcanic necks.
a. Great Basin	22a. Isolated ranges (largely dissected block mountains) separated by aggraded desert plains.
b. Sonoran Desert	22b. Widely separated short ranges in desert plains.
c. Salton Trough	22c. Desert alluvial slopes and delta plain; Gulf of California.
d. Mexican Highland	22d. Isolated ranges (largely dissected block mountains) separated by aggraded desert plains.
e. Sacramento section	22e. Mature block mountains of gently tilted strata; block plateaus; bolsons.
a. Northern Cascade Mountains	23a. Sharp alpine summits of accordant height; higher volcanic cones.
b. Middle Cascade Mountains	23b. Generally accordant summits; higher volcanic cones.
c. Southern Cascade Mountains	23c. Volcanic mountains variously eroded; no very distinct range.
d. Sierra Nevada	23d. Block mountain range tilted west; accordant crests; alpine peaks near east side.
a. Puget Trough	24a. Lowlands of diverse character; in part submerged.
b. Olympic Mountains	24b. Generally accordant crests; local alpine peaks.
c. Oregon Coast Range	24c. Uplifted peneplain on weak rocks, dissected; monadnocks of igneous rock.
d. Klamath Mountains	24d. Uplifted and dissected peneplain on strong rocks; extensive monadnock ranges.
e. California Trough	24e. Low fluvial plain.
f. California Coast Ranges	24f. Parallel ranges and valleys on folded, faulted, and metamorphosed strata; rounded crests of subequal height.
g. Los Angeles Ranges	24g. Narrow ranges and broad fault blocks; alluviated lowlands.
	25. Dissected westward-sloping granite upland (in northern part)

NOTE.—Major divisions are separated by the heaviest lines. Provinces are named on map and also distinguished by numbers. Sections are indicated by letters. Broken lines indicate boundaries much generalized or poorly known.

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