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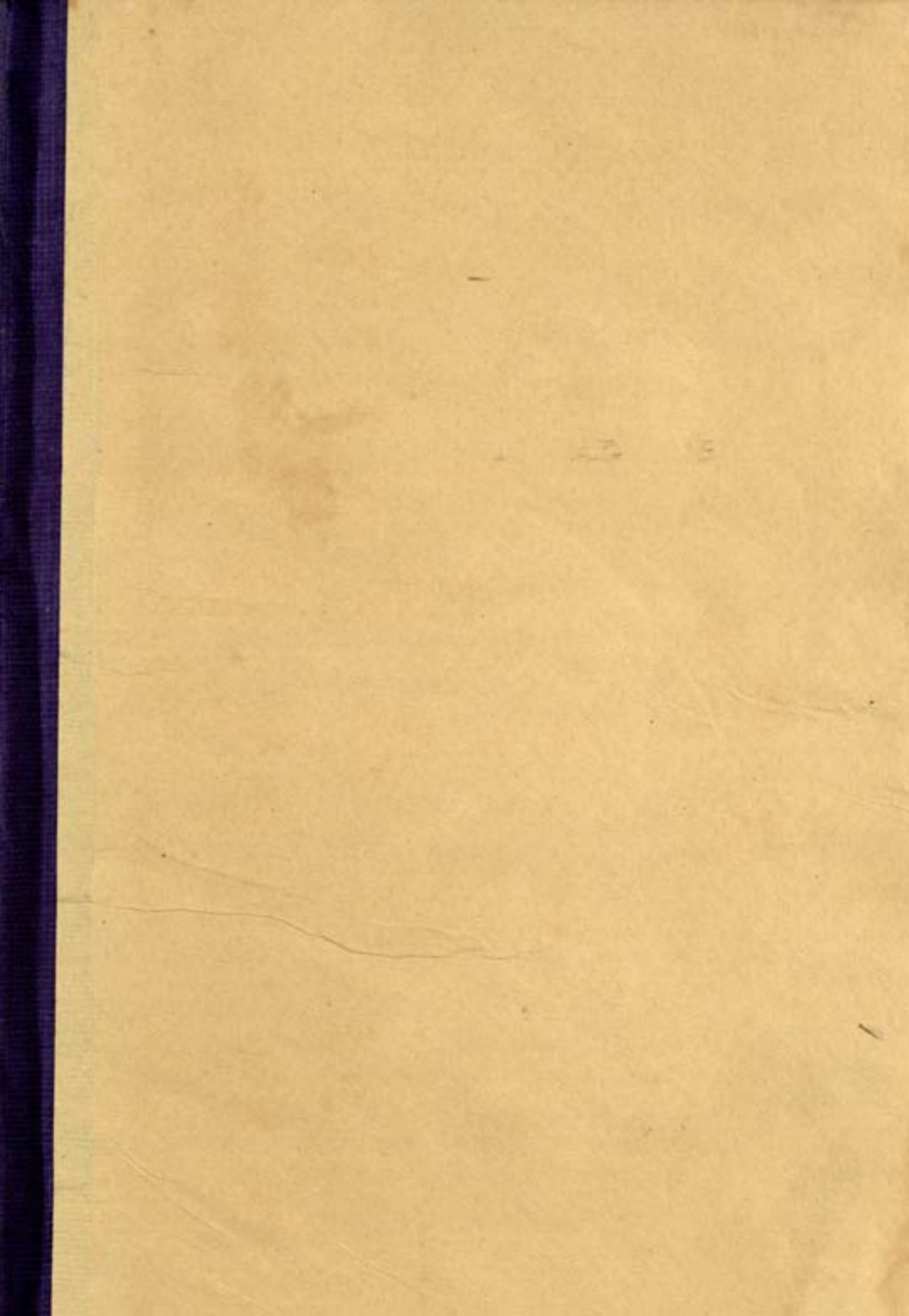
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CONSTRUCTION PLANNING,  
EQUIPMENT, AND METHODS

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# CONSTRUCTION PLANNING, EQUIPMENT, AND METHODS

44140

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*Professor of Construction Engineering  
Agricultural and Mechanical College of Texas*



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# CONSTRUCTION PLANNING, EQUIPMENT, AND METHODS

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## II

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## PREFACE

During the earlier years of construction, success frequently depended on one's ability to drive men, mules, and equipment in order to maintain a progress schedule and in order to complete a project at the lowest possible cost. Today, such practices have been replaced to a large degree by carefully planning each step for a project before construction is started, by selecting the most suitable construction equipment for a given project, by analyzing a project to determine the best methods of constructing it, and by maintaining adequate controls over a project through periodic reports showing progress, costs, and other desirable information. Although engineering principles are being applied to construction with excellent results, there is a need for the application of even more engineering knowledge.

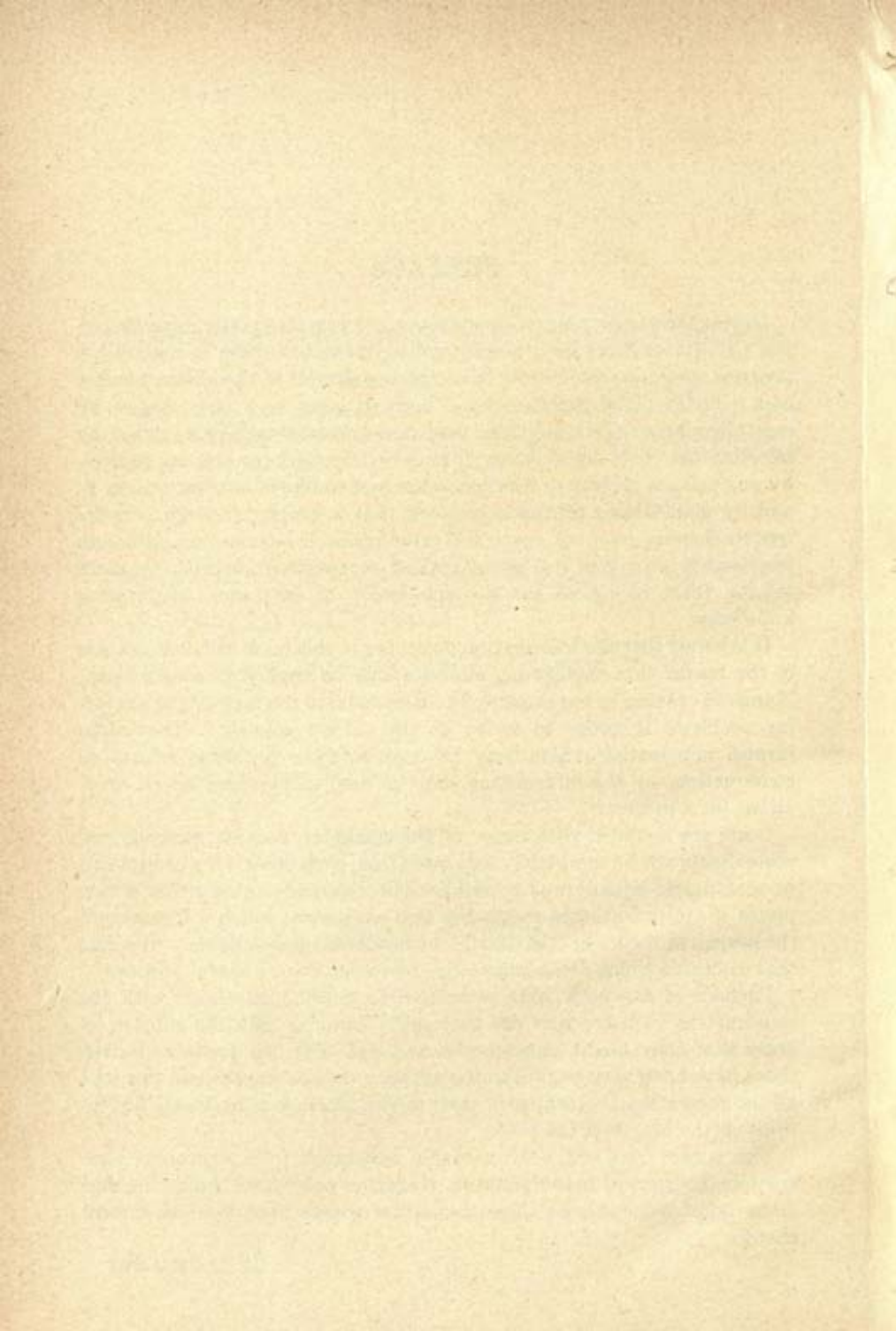
It is hoped that the information presented in this book will demonstrate to the reader that engineering analyses may be applied to construction. Numerous examples are presented to demonstrate the methods of analyzing problems in order to arrive at the correct solution. The tables furnish information which may be used to solve problems related to construction, or the information may be used in planning actual operations for a project.

Costs are included with many of the examples, because methods and costs should not be completely separated from each other. If a project can be constructed equally well by either of two methods, or by either of two pieces of equipment, that method or that equipment which will construct the project at the lower cost usually is considered more suitable. It seems that examples which demonstrate this reasoning serve a useful purpose.

Portions of the book were submitted to persons associated with the construction industry who are thoroughly familiar with the subject, in order that they might criticize the material. The comments and criticisms have been very helpful to the author. It was impractical to adopt all the suggestions, regardless of their merits, because of necessary limitations on the length of the book.

The author received very valuable assistance from numerous contractors, equipment manufacturers, magazine publishers, engineers, and many other persons. To these the author wishes to express his sincere thanks.

R. L. PEURIFOY





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## LIST OF ABBREVIATIONS AND SYMBOLS

AGC	Associated General Contractors of America, Inc.
bbl	barrel
bhp	brake horsepower
bm	bank measure, volume of earth prior to loosening
cfm	cubic feet per minute
const.	construction
cu ft	cubic foot
cu yd	cubic yard
cwt	100 pounds
deg	degree
est.	estimated
F	Fahrenheit temperature
f.o.b.	free on board
fpm	feet per minute
fps	feet per second
ft	foot
ft-lb	foot-pound
gal	gallon
gpm	gallons per minute
hp	horsepower
hr	hour
in.	inch
lb	pound
lin ft	linear foot
M	1,000
m	meter
max	maximum
M fbm	1,000 feet board measure of lumber
min	minute, minimum
mm	millimeter
mph	miles per hour
op	operation
plf	pounds per linear foot
psf	pounds per square foot
psi	pounds per square inch
rpm	revolutions per minute
S4S	smooth on 4 sides for lumber

sec	second
sq ft	square foot
sq in.	square inch
square	100 square feet of area
sq yd	square yard
std	standard
tph	tons per hour
wk	week
yd	yard
yr	year

## CHAPTER 1

### INTRODUCTION

**The Purpose of This Book.** The efforts of an engineer, who designs a project, and the constructor, who builds the project, are directed toward the same goal, namely, the creation of something which will serve the purpose for which it is built in a satisfactory manner. Construction is the ultimate objective of a design. It is hoped that this book will assist the reader in more fully understanding the construction industry. It is hoped that the material presented in the book will illustrate how the application of engineering fundamentals and analyses to construction activities may reveal methods of improving the quality, while at the same time reducing the costs, of construction.

**The Engineer and Construction.** When the prospective owner of a project under consideration recognizes a need for the project, he usually employs an engineer to make a study to determine whether the project is justified. If the study indicates that it is justified, an engineer will be engaged to prepare the plans and specifications and usually to supervise the construction of the project. It is the duty of the engineer to design that project which will most nearly satisfy the needs of the owner at the lowest practical cost. The engineer should study every major item to determine if it is possible to reduce the cost without unduly reducing the service which the project will furnish. It may be possible to change a design, modify the requirements for construction, or revise portions of the specifications in such a manner that the cost of the project will be reduced without sacrificing its essential value. An engineer who practices this philosophy is rendering a real service to his client. Thus, it seems evident that an engineer should be reasonably familiar with construction methods and costs if he is to design a project that is to be constructed at the lowest practical cost.

**The Construction Industry.** For most projects, after the design is completed and plans and specifications are prepared, professional constructors, usually referred to as contractors, are given an opportunity to submit bids to the owner, indicating the prices for which they will construct the project. It is common practice to award the contract to the lowest qualified bidder. If the project involves so many uncertainties that it is impossible to estimate the cost in advance, it may be desirable to award a contract on the basis of the cost plus a fixed fee. Under the terms of



cost-plus-a-fixed-fee contract the owner agrees to reimburse the contractor for all the costs incurred in constructing the project and to pay him an additional fee as a profit. The amount of the fee usually is agreed on in advance of entering into a contract. The contract may require the contractor to furnish all materials, construction equipment, labor, and supervision to complete the project.

Contractors frequently are required to furnish a performance bond for each project. The bond, which is issued by an approved surety, protects the owner by guaranteeing that the project will be completed satisfactorily for the contract price. In the event the original contractor fails to complete the project, it then becomes the responsibility of the surety to obtain satisfactory completion by engaging another contractor or in some other manner which is acceptable to the owner. The cost of the performance bond, amounting to approximately 1 per cent of the cost of the project, is paid for by the contractor.

Contractors tend to specialize in the types of work which they construct. While there are no uniform lines separating the fields of construction, they may be divided into building, highway, heavy, railroad, pipe line, municipal, marine, steel erection, etc. Several of these can be subdivided into smaller fields. The reasons for specializing are primarily a matter of business discretion. Few contractors, if any, can afford to own all the different types of equipment required for construction in all engineering fields. A contractor who attempts to own and operate such a large quantity of equipment might find himself "equipment-poor." As it costs money to own equipment, even though it is not working, idle equipment represents a continuing loss to the owner. The cost of owning equipment is discussed in Chap. 3.

**Construction Economy and the Engineer.** The cost of a project is influenced by the requirements of the design and the specifications. Prior to completing the final design the engineer should give careful consideration to the methods and equipment which may be used to construct the project. Requirements which increase the cost without producing commensurate benefits should be eliminated. The ultimate decisions of the engineer should be based on a reasonable knowledge of construction methods and costs.

The cost of a project may be divided into five or more items: materials, labor, equipment, overhead and supervision, and profit. While the last item is beyond the control of the engineer, he does have some control over the cost of the first four items.

If the engineer specifies materials which must be transported great distances, the costs may be unnecessarily high. Requirements for tests and inspections of materials may be too rigid for the purpose for which the materials will be used. Frequently substitute materials are available

nearly which are essentially as satisfactory as other materials whose costs are considerably higher.

The specified quality of workmanship and methods of construction have considerable influence on the amount and class of labor required and the cost of labor. Complicated concrete structures are relatively easy to design and reduce to drawings but may be exceedingly difficult to build. A high-grade concrete finish may be justified for exposed surfaces in a fine building, but the same quality of workmanship is not justified for a warehouse. The quality of workmanship should be in keeping with the type of project.

Engineers should keep informed on the developments of new construction equipment, as such information will enable them to modify the design or construction methods to permit the use of economical equipment. The use of a dual-drum concrete-paving mixer, instead of a single-drum mixer, will increase the production of concrete materially and for most projects will reduce the cost of the pavement. The use of the high-capacity earth loader and large trucks may necessitate a change in the location, size, and shape of a borrow pit, but the resulting economies may easily justify the change. The use of wellpoint systems for controlling ground water has eliminated the need of cofferdams for many projects. The development of underreamed footings has changed the foundation designs for many structures from load-bearing piles to less expensive types of supports.

The following are indicative of methods which an engineer may use to reduce the costs of construction:

1. Design concrete structures with as many duplicate members as practical in order to permit the reuse of forms without rebuilding.
2. Simplify the design of the structure where possible.
3. Design for the use of cost-saving equipment and methods.
4. Eliminate unnecessary special construction requirements.
5. Design to reduce the required labor to a minimum.
6. Specify a quality of workmanship that is consistent with the quality of the project.
7. Furnish adequate foundation information where possible.
8. Refrain from requiring the contractor to assume the responsibility for information that should be furnished by the engineer or for adequacy of design.
9. Use local materials when they are satisfactory.
10. Write simple, straightforward specifications which clearly state what is expected. Define the results expected, but within reason permit the contractor to select the methods of accomplishing the results.
11. Use standardized specifications, with which the contractors are familiar, where possible.



#### 4 CONSTRUCTION PLANNING, EQUIPMENT, AND METHODS

12. Hold prebidding conferences with contractors in order to eliminate uncertainties and to reduce change orders to a minimum.

13. Use inspectors who have sufficient judgment and experience to understand the project and have authority to make decisions.

Other examples, illustrating methods of effecting economy in construction, will be found in succeeding chapters of this book.

**Construction Economy and the Contractor.** One desirable characteristic of a successful contractor is a degree of dissatisfaction over the plans and methods under consideration for constructing a project. Complacency by members of the construction industry will not develop new equipment, new methods, or new construction planning, all of which are desirable for continuing improvements in the products of the industry at lower costs. A contractor who does not keep informed on new equipment and methods will soon discover that his competitors are underbidding him. It is hoped that the analyses and examples presented in this book will impress on the reader the value of carefully studying each project in order to select the methods and equipment that will produce the greatest construction economy.

Suggestions for possible reductions in construction costs by the contractor include, but are not limited to, the following:

1. Prebidding studies of the project and the site to determine the effect of:
  - a. Topography.
  - b. Geology.
  - c. Climate.
  - d. Sources of materials.
  - e. Access to the project.
  - f. Housing facilities if required.
  - g. Storage facilities for materials and equipment.
  - h. Labor supply.
  - i. Local services.
2. The use of substitute construction equipment, having higher capacities, higher efficiencies, higher speeds, more maneuverability, and lower operating costs.
3. The payment of a bonus to the key personnel for production in excess of a specified rate.
4. The use of radios as a means of communication between the headquarters office and key personnel on projects covering large areas.
5. The practice of holding periodic conferences with key personnel to discuss plans, procedures, and results. Such conferences should produce better morale among the staff members and should result in better coordination among the various operations.



6. The adoption of realistic safety practices on a project as a means of reducing accidents.
7. Considering the desirability of subcontracting specialized operations to other contractors who can do the work more economically than the general contractor.
8. Considering the desirability of improving shop and servicing facilities for better maintenance of construction equipment.

An estimator for a contracting firm prepared a bid for a project. When the bids were opened, it was discovered that this firm's bid was so low that the members of the firm feared that a serious error had been made in preparing the bid. They could foresee a substantial loss as the result of the bid. The estimator was called in and asked if he thought he could construct the project for the estimated cost. He replied that he could if he were permitted to adopt the construction methods which he used in estimating the cost. He was placed in charge of the project and completed it with a satisfactory profit.

## CHAPTER 2

### JOB PLANNING AND MANAGEMENT

**General Information.** This chapter deals with the planning that is necessary prior to starting actual construction on a project. Such planning should facilitate the construction by establishing:

1. The time for delivering materials
2. The types, quantities, and duration of equipment needs
3. The classification and numbers of laborers needed and the periods during which they will be needed
4. The extent to which financial aid, if any, will be needed
5. The time required to complete the project

A contractor should do some of this planning prior to bidding a project, as such planning frequently will reveal factors which will affect the cost of the project, and thus will influence the amounts shown in a bid.

**Construction Stages.** On large projects it may be desirable or essential to divide the project into several construction stages, which may be constructed independently or in conjunction with each other.

A new water supply for a city might include the following stages:

1. Clearing the reservoir site
2. The earth-fill dam
3. The concrete dam, spillway, and controls
4. The pump station
5. The transmission line, tunnels, etc.
6. The water-treatment plant

Each of the stages may be constructed under a separate contract. The quantities of work and the duration of construction for each stage must be known in advance in order that each may be constructed in the proper sequence. The reservoir site must be cleared before the dam is completed and the storage of water begins. It may be necessary to complete the concrete portion of the dam, and to install control gates in order that this structure may serve as a means of diverting the water in the stream while the earth fill is being placed. The pump station, transmission line, and treatment plant should be completed by the time the reservoir has stored a sufficient quantity of water to be usable.

**Construction Operations.** Most projects are divided into construction operations to facilitate job planning. A construction operation is a portion of a project which may be performed by a classification of laborers or perhaps a single type of equipment. For example, in constructing a



reinforced-concrete retaining wall the project might be divided into the following operations:

1. Excavation, machine
2. Excavation, hand
3. Forms
4. Reinforcing steel
5. Concrete
6. Backfill

In planning the construction of a highway requiring a new location the project might be divided into the following operations:

1. Moving to the project and setting up plant
2. Clearing and grubbing the right of way
3. Earthwork, cut and fill
4. Drainage structures, box culverts
5. Pavement
6. Cleanup and removal of plant

In order to estimate the progress in constructing the project, the job planner should determine the quantity of work to be constructed for each operation, expressed in an appropriate unit. Then he should estimate the probable rate at which the work will be performed, allowing for estimated loss in time due to bad weather or any other cause. From this information it will be possible to estimate the total time required to complete each operation. The estimated starting date and completion date for each operation can be shown on a bar chart. In scheduling the operations the job planner should consider the desirable sequential relationships between the operations. For example, in constructing a concrete foundation unit it will be necessary to complete the excavation before concrete can be placed.

**Construction Schedules.** A construction schedule is usually in the form of a bar chart, which shows for a given project the operations, quantity, unit, and rate of constructing for each operation, and the estimated date of starting and completing each operation. It is desirable to include on the schedule provision for reporting or indicating the actual amount of work completed on each operation at any given date, such as at the end of a week or a month. If the actual progress is indicated on the schedule, it is possible to determine very quickly whether construction is progressing according to the original plans.

Schedules for projects whose construction will require less than a year may be divided into weeks, while schedules for projects requiring more than a year generally will be divided into months. A schedule should show the dates clearly. If the time is divided into weeks, it is good practice to show the end of the week, Saturday, as the effective date, with a notation reading "For week ending."



Every construction schedule should be identified with the particular project by placing on it the name of the project, the name of the owner, possibly the name of the engineer, and the location. It may be desirable to include a code to assist in reading the schedule.

A construction schedule for driving piles and for pier improvements is illustrated in Fig. 2-1.

**Preparing a Construction Schedule.** Prior to preparing a construction schedule a project should be divided into the desirable operations. The amount of work to be performed should be determined, and the rate of performing the work should be estimated for each operation. An appropriate allowance should be made for loss of time due to bad weather. In estimating the rate at which the work will be performed consideration should be given to economy of construction. The number of laborers and units of equipment should be selected that will result in the most economical construction consistent with the particular operation and the entire project. After the schedule is completed, it should be studied carefully to determine whether changes are desirable. It may be possible to delay starting an operation in order that laborers and equipment can be transferred from another operation, thereby reducing the total number of laborers and units of equipment required to complete the project. Perhaps a delay in starting an operation may permit a unit of equipment to be transferred from another project to this project, thus eliminating the need of purchasing or renting additional equipment.

As an example illustrating the preparation of a construction schedule, let us consider a project requiring the relocation of a highway. The project will involve the following quantities and operations:

Length, 8.568 miles

Width of right of way, 100 ft

Clearing and grubbing, 64 acres, medium timber, distributed along the project

Drainage structures, 12 multibox concrete culverts, 3 openings, average length 32 ft

Earth fill in excess of cut, from borrow pits, with average haul distance  $\frac{3}{4}$  mile, a total of 136,800 cu yd bm

Concrete pavement, width 24 ft, average thickness 9 in., total area 120,636 sq yd

**Job Description.** The area in timber is slightly more than one-half of the total area of the right of way. The trees are principally oak and elm, with maximum size 14 in. The specifications require the contractor to remove all trees and roots to a depth of 18 in. and to burn the timber on the right of way.

The earth is a mixture of sand and clay, whose average borrow-pit weight is 93 lb per cu ft. The average swell will be 25 per cent. It is estimated that the average initial moisture content will be 8 per cent by weight. The specifications require a moisture content of 12 per cent

Job No. \_\_\_\_\_  
Project \_\_\_\_\_  
Owner \_\_\_\_\_  
Location \_\_\_\_\_

Date Jan 10, 1954

Indicates the per cent of work completed on the date of the report

[illegible]

Fig. 2-1. Construction schedule for driving piles and for pier improvements.



during compaction. The earth is to be placed in layers not exceeding 8 in. in thickness, when loose, and compacted to a density of 97 lb per cu ft. The earth will be excavated with a  $1\frac{1}{2}$ -cu-yd power shovel and hauled in trucks having a struck capacity of 6 cu yd.

The culverts will be constructed with concrete having a 28-day compressive strength of 3,000 psi. The forms will remain in place for not less than 7 days. Heavy equipment may not be moved across the culverts until 14 days after the concrete is placed. The earth on the sides and over the culverts will be compacted.

The concrete for the pavement will be mixed in a dual-drum 34E paving mixer, batch size 40.8 cu ft. A batch will require 3,112 lb of gravel, 2,042 lb of sand, 711 lb of cement, and 30.3 gal of water. Pavement may not be placed on any section of the highway until the earth base at that section has cured for at least 28 days after it is placed.

The sand, gravel, and cement will be delivered to a railroad siding, approximately 1 mile from the mid-point of the project. An existing gravel road will be used for a haul road. Water is available in several ponds located along the project, with an average haul distance of 2 miles. A gasoline-engine-driven pump, whose capacity is 400 gpm, will be used to pump the water into the sprinkler trucks.

**The Construction Schedule.** The construction schedule for the project is shown in Fig. 2-2.

In preparing the schedule for the project it is estimated that bad weather will cause delays amounting to approximately 20 per cent of the actual time required for construction. The calculations, which give the probable rate of construction and time required for each operation, are illustrated in the following articles. In preparing the schedule a week is considered to be 5 days of 8 hr each.

**Moving In.** The moving-in operation consists of transporting to and setting up at the project a two-compartment aggregate-batching bin, a bulk-cement storage bin, a tool warehouse, offices, testing laboratory, equipment-repair and -servicing facilities, and the construction equipment required for the project. It may not be desirable to transport all the construction equipment to the project prior to beginning construction, but the equipment that will be needed first should be moved to the project as early as possible.

It is estimated that 1 week will be required for moving in and setting up the plant.

**Clearing and Grubbing.** The timber will be cleared from the right of way with bulldozers mounted on diesel crawler tractors, developing 130 drawbar hp. It is estimated that a bulldozer can clear 1 acre per day or 5 acres per week. Two bulldozers will be used to clear 10 acres per week,



Job No. 148  
Project SH 1764  
Owner State Highway Dept of Texas  
Location Brazos County  
Date of project 1953  
Date of report June 27, 1953



actual working time. The total time to remove the timber will be

Actual operating time, 64 acres @ 10 acres per wk	= 6.4 wk
Add 20% for lost time	= 1.3 wk
Total time	= 7.7 wk
Round time out to	8 wk

A single crawler tractor, pulling a rooter, should be able to remove all tree roots to a depth of 18 in. at the same rate that the bulldozers push down the trees. The roots will be stacked on the piles of trees, by laborers, and burned.

It should be possible to start this operation before moving in is completed. Therefore the construction schedule provides for clearing to start immediately.

**Drainage Structures.** If the water courses are dry during the construction of the drainage structures, the operations will consist of excavating to the required grade and constructing the concrete slab, walls, roof, and wing walls. If flowing water is expected, it may be desirable to construct a temporary earth dam above and below the structure to divert the water through a ditch, dug to the side of the culvert, using a bulldozer to excavate the earth from the ditch.

The actual time required to construct a culvert should be about as follows:

Bulldozer constructing temporary dams, excavating diversion ditch, and excavating for base of a culvert	1 day
Fine grading, by hand	1 day
Erecting forms and placing reinforcing steel for base	1½ days
Placing concrete for base	½ day
Erecting forms for walls and roof and placing reinforcing steel	3 days
Placing concrete for walls and roof	1 day
Removing and cleaning forms	1 day
Total working time	9 days
Add 20% for lost time	2 days
Total time	11 days.

Although it will require an estimated 11 days to construct one culvert, it is probable that necessary interruptions will increase the total elapsed time to 3 weeks. However, it is not necessary to complete one culvert prior to starting another. Assume that three culverts are under construction at the same time. Under this schedule one culvert should be finished each week, with time allowed for the curing of the concrete. Thus, a total of 12 weeks will be required to complete the culverts.

Construction on the culverts should not be started until a sufficient portion of the right of way is cleared to permit the construction to be continued without interruption. The culverts will be started 2 weeks after clearing is started.



**Earth Fill.** The earth for the fill will be excavated with a  $1\frac{1}{2}$ -cu-yd power shovel, whose output should be approximately 150 cu yd per hr bank measure (see Table 6-2). A truck having a struck capacity of 6 cu yd should haul 6 cu yd bank measure if the heaped capacity is 7.5 cu yd loose measure.

Under reasonably good haul-road conditions a truck should make a round trip in 12 min. If it is assumed that a truck will operate an average of 50 min per hr, because of necessary delays, the number of trucks required will be obtained as follows:

$$\begin{aligned}\text{Trips per hr per truck, } 50 \div 12 &= 4.17 \\ \text{Volume hauled per hr per truck, } 4.17 \times 6 &= 25 \text{ cu yd} \\ \text{No. trucks required, } 150 \div 25 &= 6\end{aligned}$$

An extra truck should be provided as a stand-by unit for use in the event of a breakdown by one of the trucks.

The volume of earth placed in a week will be  $150 \text{ cu yd per hr} \times 40 \text{ hr per week} = 6,000 \text{ cu yd}$ . The time required to complete the fill will be

$$\begin{aligned}\text{Working time, } 136,800 \text{ cu yd @ } 6,000 \text{ cu yd per wk} &= 22.8 \text{ wk} \\ \text{Add for lost time due to weather} &= 3.4 \text{ wk} \\ \text{Total time} &= 26.2 \text{ wk} \\ \text{Round time out to} &= 26 \text{ wk}\end{aligned}$$

In addition to the shovel and trucks, it will be necessary to provide one or more heavy-duty graders to smooth and shape the earth in the fill and to maintain the haul roads, one or more sheep's-foot rollers to compact the earth, and enough sprinkler trucks to supply the required water.

In 1 hr the volume of earth hauled will be 150 cu yd bank measure. Owing to the increased density in the fill, the volume in the fill will be  $150 \times \frac{93}{97} = 143.8 \text{ cu yd}$ . The earth will be placed in layers whose compacted thickness will be 6 in. The area covered in an hour will be  $143.8 \times 27 \div \frac{1}{2} = 7,765 \text{ sq ft}$ . If it is necessary for the grader to make four passes for each layer, the effective area to be covered in an hour will be  $4 \times 7,765 = 31,060 \text{ sq ft}$ . A tandem-drive heavy-duty grader with a 12-ft blade, set at an angle, should give an effective blade width of 9 ft and should average 1.5 mph, allowing for turns and other lost time. The total area covered will be  $1.5 \times 5,280 \times 9 = 71,280 \text{ sq ft per hr}$ . As the grader will be required less than one-half time for shaping the earth fill, it may be used to maintain the haul roads.

Assume that it will be necessary for a sheep's-foot roller drum to make 12 passes in compacting a 6-in. layer of fill. The effective area to be covered will be  $7,765 \times 12 = 93,180 \text{ sq ft per hr}$ . A single-drum roller, 4.5 ft wide, with an average speed of 1.5 mph will cover

$$1.5 \times 5,280 \times 4.5 = 35,640 \text{ sq ft per hr}$$



Thus, the compaction will require

$$93,180 \div 35,640 = 3 \text{ drums}$$

pulled by a crawler tractor.

The quantity of water required per hour may be determined as follows:

$$\begin{aligned} \text{Weight of earth per cu yd bm, } 27 \times 93 &= 2,511 \text{ lb} \\ \text{Weight of earth placed per hr, } 150 \times 2,511 &= 376,650 \text{ lb} \\ \text{Quantity of water added, by weight, } 12 - 8 &= 4\% \\ \text{Weight of water per hr, } 376,650 \times 0.04 &= 15,066 \text{ lb} \\ \text{Quantity of water required per hr, } 15,066 \div 8.33 &= 1,810 \text{ gal} \end{aligned}$$

If a 2,000-gal truck can make a round trip in an hour, it will supply sufficient water for the fill.

It is estimated that placing the fill can be started 2 weeks after clearing and grubbing is started.

**Pavement.** The area of the concrete pavement will be

$$8.568 \times 5,280 \times 24 \div 9 = 120,630 \text{ sq yd}$$

The volume will be  $120,630 \div 4 = 30,157$  cu yd, with no allowance for wastage or overrun in thickness. A 34E dual-drum paving mixer, producing 40.8 cu ft of concrete per batch, can mix a batch in 1 min under favorable conditions. If it is assumed that the mixer will actually operate 50 min per hr, an average of 50 batches per hour can be mixed and placed. This is equal to  $50 \times 40.8 \div 27 = 75.6$  cu yd per hr, or 12,096 sq yd per week.

The time required to complete the operation will be

$$\begin{aligned} \text{Working time, } 120,630 \text{ sq yd @ } 12,096 \text{ sq yd per wk} &= 10 \text{ wk} \\ \text{Add } 20\% \text{ for lost time} &= 2 \text{ wk} \\ \text{Total time} &= 12 \text{ wk} \end{aligned}$$

Dump trucks, hauling 2 batches per load, will be used to transport the aggregate and cement from the batching plant to the mixer. The dry weight of a batch will be 5,865 lb, and the volume will be approximately 1.5 cu yd. It will require trucks with a capacity of at least 3 cu yd, with a removable partition to separate the two batches. For an average round-trip haul distance of 4 miles a truck should make 2.5 trips per hour, with allowance for delays. The number of trucks needed will be

$$\begin{aligned} \text{Batches hauled per truck per hr, } 2 \times 2.5 &= 5 \\ \text{Trucks required, } 50 \text{ batches per hr at } 5 \text{ batches per truck} &= 10 \\ \text{Use 11 trucks, with 1 for a stand-by} \end{aligned}$$

The major equipment for this operation will include

- 1 34E dual-drum paver
- 1 subgrader

## For week ending

Date of project 1953

[illegible]

FIG. 2-3. Equipment use schedule



- 1 vibrator spreader
- 1 finisher
- 3,000 lin ft of steel forms
- 1 water tank on trailer, 1,500 gal
- 1 water truck, 2,000 gal
- 1 flat-bed truck, 4-6 tons
- 1 cement-storage silo, 750 bbl
- 1 two-compartment aggregate bin and batcher, 40 tons
- 1 clamshell, 2-cu-yd bucket
- 11 dump trucks, 3 cu yd capacity
- 1 pickup truck

Unless it is practical to increase the rate of placing the fill, the concrete pavement should not be started until 18 weeks after the construction of the fill is started. This will permit the last portion of the fill to cure 4 weeks prior to placing the pavement, as required by the specifications.

**Equipment-use Schedule.** Figure 2-3 illustrates a method of scheduling the equipment to be used on a project. The equipment is that which will be used on the project for Fig. 2-2. Such a schedule should assure that equipment will be used efficiently.

**Comments on the Construction Schedule.** After the initial schedule is completed, it should be examined critically to determine whether revisions are desirable. If so, they should be made prior to completing the final schedule.

An examination of the schedule for Fig. 2-2 reveals that the time required to place the earth fill is estimated to consume 26 weeks. It may be desirable to increase the rate of placing the earth, either by using a larger power shovel, or by using two power shovels, and more trucks.

**Ordering Materials.** The construction schedule may be used as a guide in specifying the delivery dates for materials for the project. Materials should be delivered to a project far enough in advance of their need to assure that there will be no delays. However, it is not good practice to have materials at the job site too far in advance of their need, as they may deteriorate, may be damaged or lost, or may congest the working area.

For the project illustrated by the construction schedule in Fig. 2-2 the first materials needed will be the lumber for forms, reinforcing steel, cement, sand, and gravel for the drainage structures. If guaranteed delivery dates are obtainable, it may be satisfactory to arrange for deliveries to be made a week in advance of the estimated starting date for each structure.

The deliveries of materials for the concrete paving should be started about a week prior to the beginning of construction, around Aug. 3, and should be continued at a uniform rate for the duration of the paving operation. The probable rate of delivery should be 3,780 bbl of cement, 2,042 tons of sand, and 3,112 tons of gravel per week. If this information



Operation	Classification	No. men
Moving in.....	Foreman	1
	Mechanics	2
	Truck drivers	2
	Crane operator	1
	Crane oiler	1
	Laborers	3
Clearing and grubbing.....	Foreman	1
	Tractor operators	3
	Laborers	2
Drainage structures.....	Foreman	1
	Carpenters	4
	Ironworkers	2
	Tractor operator	1
	Clamshell operator	1
	Mixer operator	1
	Cement finisher	1
	Truck drivers	2
	Laborers	10
	Foreman	1
Earth fill.....	Shovel operator	1
	Shovel oiler	1
	Truck drivers, earth	6
	Truck driver, water	1
	Grader operator	1
	Tractor operator	1
	Utility laborers	2
	Foremen	2
Pavement.....	Cement batcher	1
	Clamshell operator	1
	Clamshell oiler	1
	Aggregate batcher	1
	Truck drivers, aggregate	10
	Mixer operator	1
	Subgrader operator	1
	Vibrator spreader operator	1
	Finisher operator	1
	Truck driver, water	1
	Truck driver, forms	1
	Laborers	14
	Surveyor	1
	Helpers	2
	Foreman	1
General cleanup.....	Grader operator	1
	Mechanics	2
	Truck drivers	2
	Crane operator	1
	Crane oiler	1
	Laborers	3

is given to the material suppliers, they can arrange to furnish the materials as they are needed. Revisions in delivery dates can be made, if necessary, during the construction of the project.

**Scheduling Laborers.** The number of laborers required during the construction of the project can be determined by estimating the number required for each operation. The number required for each operation of Fig. 2-2 might be as shown in the tabulation on page 17.

If the laborers are consolidated, by classification, for the entire project, it will be possible to determine the estimated number of workers for each classification for any period of time during the construction of the project. This information can then be used as the basis for arranging in advance for the laborers needed. Figure 2-4 illustrates the consolidation of the laborers for the project of Fig. 2-2.

**Financing the Project.** A construction schedule may be used to estimate the amount of funds that a contractor must provide in financing a project during construction. Most construction contracts specify that the owner shall pay to the contractor a stated per cent of the value of work completed during each month. The payment for work completed during a month is usually made by the tenth of the following month. Upon the completion of the project the retained funds are paid to the contractor. The amount retained by the owner is commonly 10 per cent. An analysis of the construction schedule will indicate the probable total expenditures and receipts through any desired dates. The excess of expenditures over receipts indicates the amount of financing which the contractor must provide from sources other than the owner.

The estimated expenditures and receipts are determined as shown in Tables 2-1 to 2-3.

Figure 2-5 is a chart showing the estimated cumulative expenditures and receipts for the highway project of Fig. 2-2. The expenditures are based on end-of-week payments for costs incurred. This assumption is not entirely correct, as costs will be paid in some instances at the times of purchases, while other costs will be paid at the end of the month.

The chart shows that payment to the contractor for the work completed in any given month is received on the tenth of the following month.

The difference between the amount of money spent and the amount received at any time during the period of construction is indicated by the vertical distance between the two graphs for that time.

**Job Layout.** One of the first duties of a superintendent when he assumes the responsibility of starting construction is to prepare a job layout for the project. On this layout he will draw to scale the area available for offices, warehouses, storage of materials, equipment, and earth, and constructing forms and fabricating reinforcing steel. In preparing the job layout the superintendent should endeavor to arrange all



## For week ending

Project SH1764

Project SH1764Owner State Highway Dept of TexasLocation Brazos County

Date of project 1953

Fig. 2-4. Employment schedule.



TABLE 2-1. FORM FOR ESTIMATING EXPENDITURES DURING CONSTRUCTION

Weeks after starting	Operations under const.	Expenditure per wk	Cumulative expenditures	Weeks after starting	Operations under const.	Expenditure per wk	Cumulative expenditures
1	1, 2	\$5,920	\$ 5,920	18	4	\$ 3,683	\$111,888
2	2	1,920	7,840	19	4	3,683	115,571
3	2, 3, 4	8,403	16,243	20	4	3,683	119,254
4	2, 3, 4	8,403	24,646	21	4, 5	33,839	153,093
5	2, 3, 4	8,403	33,049	22	4, 5	33,839	186,932
6	2, 3, 4	8,403	41,452	23	4, 5	33,839	220,771
7	2, 3, 4	8,403	49,855	24	4, 5	33,839	254,610
8	2, 3, 4	8,403	58,258	25	4, 5	33,839	288,449
9	3, 4	6,483	64,741	26	4, 5	33,839	322,288
10	3, 4	6,483	71,224	27	4, 5	33,839	356,127
11	3, 4	6,483	77,707	28	4, 5	33,839	389,966
12	3, 4	6,483	84,190	29	5	30,156	420,122
13	3, 4	6,483	90,672	30	5	30,156	450,278
14	3, 4	6,483	97,156	31	5	30,156	480,434
15	4	3,683	100,839	32	5	30,156	510,590
16	4	3,683	104,522	33	6	1,500	512,090
17	4	3,683	108,205				

areas to reduce the time consumed in carrying materials from the storage areas to the project. Materials that are similar in use should be stored close together, where possible. The general office and warehouse should be located near the main entrance in order that persons visiting the project for business purposes will not have to travel around the construction areas to reach the office. This should reduce the danger of injuries to visitors and the confusion that frequently is associated with the presence of strangers around a project. If the general warehouse is near the entrance, it will facilitate the delivery of material to be stored in the warehouse and also it will permit closer supervision of materials removed from the warehouse. However, if a warehouse is needed to store heavy materials, such as machines that will be incorporated into the project, it may be desirable to consider using additional warehouses, located nearer the project.

Figure 2-6 illustrates a job layout for a multistoried reinforced-concrete frame building. The contractor is fortunate in having adequate area for easy storage of all materials at the job site. This is not commonly the case for buildings erected in congested cities, where storage areas at the job site are limited or nonexistent. If area is not available at the job site, the contractor must obtain storage area as near the job site as possible.

TABLE 2-2. ESTIMATED RECEIPTS DURING CONSTRUCTION

Date of receipt	No. wk per month	Operations under const.	Wk under const. for period	Units completed per wk	Unit price received during const.	End-of-period receipts per operation	Est. end-of-period receipts	Cumulative receipts
5/10	4.8	1	1	1	\$ 0	\$ 0		
		2	4.8	8	252.00	9,677		
		3	2.8	1	2,799.00	7,837		
		4	2.8	5,261	0.712	10,488	\$ 28,002	\$ 28,002
6/10	4.2	2	3.2	8	252.00	6,451		
		3	4.2	1	2,799.00	11,756		
		4	4.2	5,261	0.712	15,733	33,940	61,942
7/10	4.4	3	4.4	1	2,799.00	12,316		
		4	4.4	5,261	0.712	16,482	28,798	90,740
8/10	4.6	3	0.6	1	2,799.00	1,679		
		4	4.6	5,261	0.712	17,231	18,910	109,650
9/10	4.2	4	4.2	5,261	0.712	15,733		
		5	2.2	10,052	2.99	66,122	81,855	191,505
10/10	4.4	4	4.4	5,261	0.712	16,482		
		5	4.4	10,052	2.99	132,244	148,726	340,231
11/10	4.4	4	1.4	5,261	0.712	5,244		
		5	4.4	10,052	2.99	132,244	137,488	477,719
12/10		5	1	10,052	2.99	30,055		
		6	2		0	0	30,055	507,774
Amount retained by owner during const.							\$ 54,662	\$562,436

Figure 2-7 illustrates the job layout for the construction of the Narrows Dam in Arkansas.

**Project Control during Construction.** At specified time intervals, daily, weekly, or monthly, reports should be submitted by the project superintendent to the headquarters office, showing the actual progress on each operation during the specified time interval or through the effective date of the report. This procedure permits a close control of progress on a project. If the progress on one or more operations or on the entire project is behind schedule, such information will be known early enough to take corrective steps. If the progress on one operation is found to be out of balance with the progress on a related operation, it will be pos-



TABLE 2-3. ESTIMATED EXPENDITURES AND RECEIPTS

Job No. 148  
 Project SH 1764  
 Owner State Highway Department of Texas  
 Location Brazos County  
 Date 1953

Operation	Quantity	Units completed per wk	Const. expenditures		Unit contract price	Unit price received during const.	Total expenditures	Total receipts	
			Per unit	Per wk				During constr.	Following completion
1	1	1	\$4,000	\$ 4,000	\$ 0	\$ 0	\$ 4,000	\$ 0	\$ 0
2	64	8	240	1,920	280	252	15,360	16,128	17,920
3	12	1	2,800	2,800	3,100	2,799	33,600	33,588	37,320
4	136,800	5,261	0.70	3,683	0.78	0.712	95,758	97,402	106,704
5	120,630	10,052	3.00	30,156	3.32	2.99	361,872	360,684	400,492
6	1	.....	1,500	1,500	0	0	1,500	0	0
Totals	.....	.....	.....	.....	.....	.....	\$512,090	\$507,802	\$562,436

sible to bring operations back into balance before serious damage results.

Figure 2-2 illustrates one method of recording actual progress on a

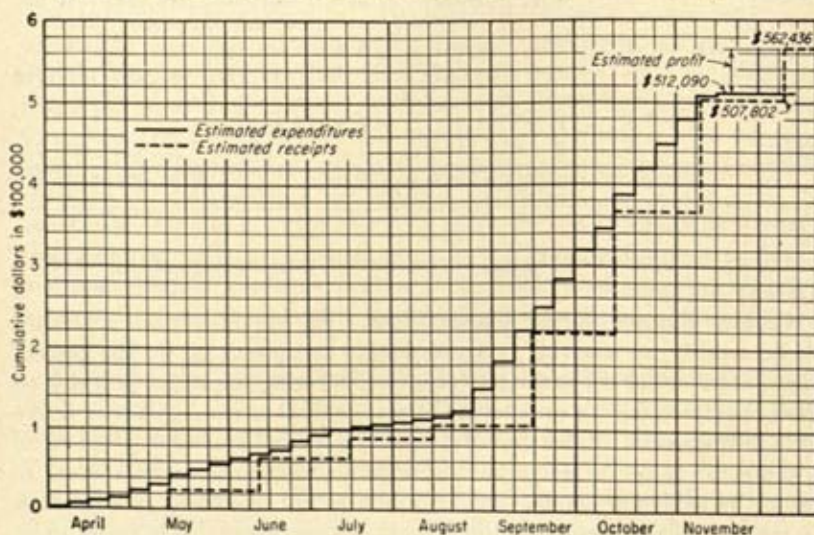


FIG. 2-5. Financial schedule.

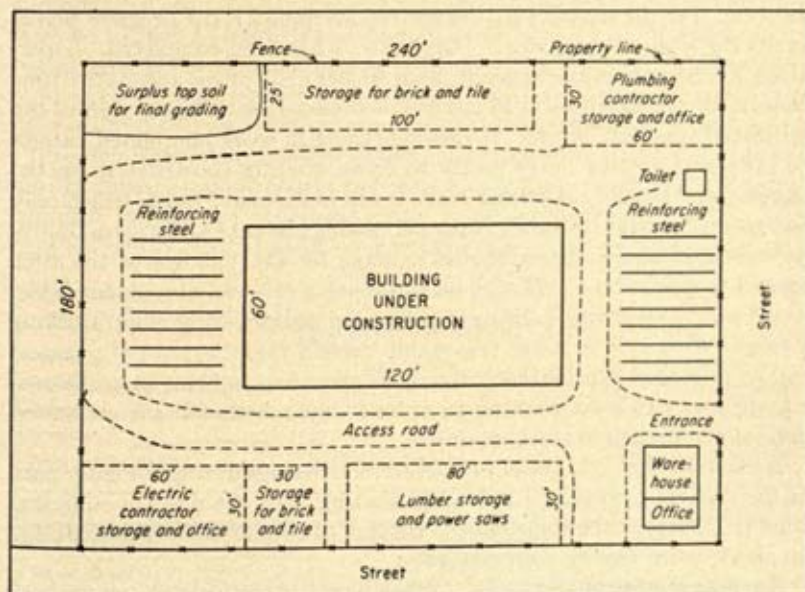


FIG. 2-6. Job layout.

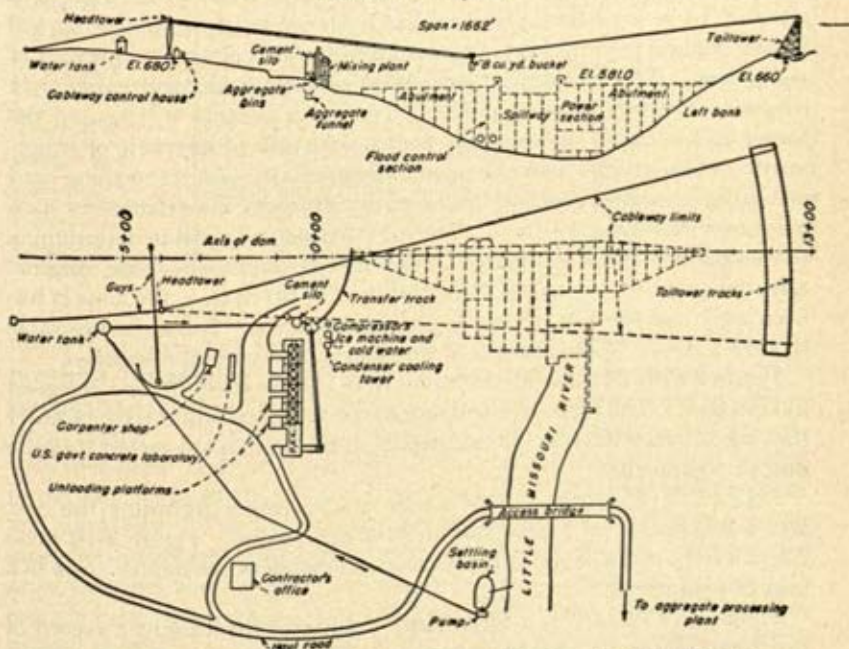


FIG. 2-7. Job layout for the Narrows Dam. (Engineering News-Record.)



project. For the effective date of the report, June 27, the progress report shows the work accomplished. Operation 2 has been completed. Operation 3 is 83 per cent completed, with 92 per cent of the estimated time consumed. Operation 4 is 36 per cent completed, with 42 per cent of the estimated time consumed. If this deficiency in work completed cannot be overcome, it may be necessary to delay starting construction on the pavement. Possibly the delay is the result of excessive rains, which may be expected to end by July 1, thus permitting the rate of construction to be increased above the estimated average for the balance of the time allotted to operation 4. If such an assumption is reasonably dependable, it will not be necessary to bring in additional equipment or begin working a longer week. If it is not reasonably certain that the rate of progress can be increased substantially, it will be necessary to bring in additional equipment or to start working on an overtime schedule if the estimated date of completion is to be retained.

Remember that it is better to take corrective steps during the early part of the construction period instead of waiting until there is not sufficient time to overcome the difficulties. Having to correct serious time delays on short notice can be very expensive.

**Keeping Equipment Records.** When a unit of construction equipment is purchased, it should be assigned a suitable identification, such as a number, to be used throughout its life. An owner of equipment should have a definite plan for keeping a record of the cost of each major unit of equipment. The record may be kept on suitable equipment cards or in ledgers. The information obtained from such records will enable the owner to determine the complete financial history of any unit of equipment. Comparisons may be made between the cost of owning and operating comparable units furnished by different manufacturers as a guide in selecting future units. The information will assist in determining the economic life of equipment. The record should show the original cost, delivered to the owner, the schedule of depreciation, the time it has been used, and the cost of repairs and maintenance. It may be desirable to keep a record of the amount of fuel and lubricating oil consumed.

Figure 2-8 illustrates a form for reporting the use of equipment assigned to a project. The forms are sent to general headquarters weekly in order that the information may be transferred to the permanent record for each unit of equipment.

Figure 2-9 illustrates a form which is suitable for recording the cost and a description of a complete unit of equipment. Figure 2-10 illustrates a form which is suitable for keeping a record of the depreciation of a unit of equipment.

Figure 2-11 illustrates a form which is suitable for keeping a record of the use or rental of a unit of equipment. The contractor who owned this

equipment rented it to his jobs at established monthly rates. The form may be revised to show rental periods in days or weeks, if desirable, or rental periods less than a month may be shown on the form, as illustrated.

Figure 2-12 illustrates a form which is suitable for keeping a record of the cost of repairs and other operating expenses for a unit of equipment.

**Project Supervision.** The extent and type of supervision required during construction varies considerably with the project. For a small, compact project the supervision may be relatively simple, while a large project, which is spread out over considerable area, such as a dam or a

EQUIPMENT USE REPORT							
Job No. _____							
Project _____							
Owner _____							
Location _____							
Week ending date _____							
	<div style="text-align: right;">Code</div> <div style="display: flex; justify-content: flex-end; gap: 10px;"> <div>Working </div> <div>Idle </div> <div>Under repair </div> </div>						
Equipment	Monday	Tuesday	Wednesday	Thursday	Friday	Saturday	Sunday
Clamshell No. 2							
Power shovel No. 4							
Truck No. 16							
Truck No. 17							
Truck No. 18							
Truck No. 19							
Truck No. 20							
Truck No. 21							
Tractor No. 4							
Tractor No. 5							
Tractor No. 6							
Tractor No. 7							

FIG. 2-8. Equipment use report.

major pipe line, may introduce many supervisory problems. The relationships between all personnel from the contracting company down through the superintendent, foreman, and working crews must be understood clearly. On a project where many men are working side by side with each other there are opportunities for misunderstandings and friction to develop. Jurisdictional arguments may arise regarding responsibility and authority. The foreman should recognize these problems at their inception and should take immediate steps to correct them. If problems arise between the foremen, the superintendent should be prepared to correct them before they reach a serious stage. One practice which has proved successful is to hold regular staff conferences to promote harmony



DESCRIPTION										ENGINE NO. _____		EQUIPMENT NO. 14-82	
D7 Caterpillar diesel tractor Arrangement 7B 9435, 7B1713 heavy duty track roller guards, 4F1867 lighting system, 3F9549 starting system, 7B4373 crankcase guard, 7B4464 front pull hook, MD7 trackson pipelayer with 3500 lb counterweight, serial MD7-681													
PURCHASE RECORD					INVOICE RECORD								
Date acquired	New Used	Vendor	P.O. No.	V.O. date	V.O. No.	Purchase price	Freight	Tax	Total cost				
11/25/52	N	A. T. Fisher Co.	BE108D	12/3/53	JV11-8	13,884 56	316 40	334 84	14,535 80				
LICENSE DATA					SALES RECORD								
Year	State	Number	Cost	Title No.	Date sold	To							
					Sales price	\$							
					Book value	\$							
					Gain or loss	\$							
					Remarks								
ITEM					SERIAL NO.		EQUIPMENT NO.						
D7 Caterpillar tractor with pipelayer					3T 6612		14-82						

FIG. 2-9. Equipment ownership cost.

DEPRECIATION RECORD										
EQUIPMENT NO. 14-82										
DESCRIPTION <u>D7 Caterpillar tractor with pigelay</u>										
Date	Depreciation			Book value	Total cost	Date	Depreciation			Total cost
	Rate per mo	Amount	To date				Rate per mo	Amount	To date	
	302 83			14,535 80			302 83			14,535 80
11/30/52				14,535 80		11/30/53		302 83	3,633 96	10,901 84
12/31/52		302 83	302 83	14,232 97		12/31/53		302 83	3,936 79	10,599 01
1/31/53		302 83	605 66	13,930 14		1/31/54		302 83	4,239 62	10,296 08
2/28/53		302 83	908 49	13,627 31		2/28/54		302 83	4,542 45	9,993 35
3/31/53		302 83	1,211 32	13,324 48		3/31/54		302 83	4,845 28	9,690 52
4/30/53		302 83	1,514 15	13,021 65		4/30/54		302 83	5,148 13	9,387 69
5/30/53		302 83	1,816 98	12,718 82		5/30/54		302 83	5,450 94	9,084 86
6/30/53		302 83	2,119 81	12,415 99						
7/31/53		302 83	2,422 64	12,113 16						
8/31/53		302 83	2,725 47	11,810 33						
9/30/53		302 83	3,028 30	11,507 50						
10/31/53		302 83	3,331 13	11,204 67						
Depreciation rate <u>25</u> per cent per year										

FIG. 2-10. Equipment depreciation record.



## EQUIPMENT NO. 14-82

DESCRIPTION: D7 Caterpillar tractor with pipelayer.

[illegible]

FIG. 2-11. Equipment rental record.





and understanding between key personnel by permitting each one to understand better the problems of the other.

Figure 2-13 illustrates an organization chart which shows the relationship between all major departments of a construction gang. The actual organization of a chart will vary with the particular project.

**The Use of Two-way Radio in Supervising a Project.** It is becoming increasingly common practice to use two-way radios, transmitters, and receivers in supervising construction projects. The use of such equipment is especially desirable for a project that is spread out over a large

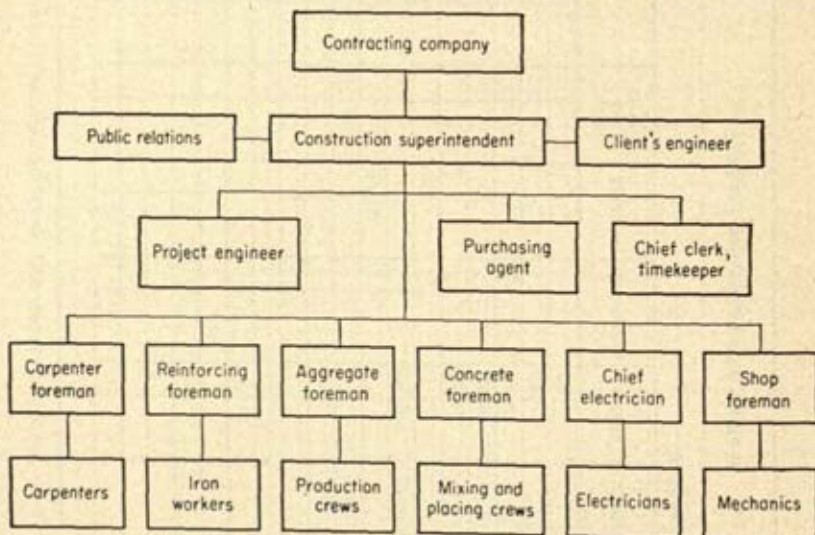


FIG. 2-13. Typical organization chart for a construction project.

area, where one operation is dependent on another. Among the advantages are the following:

1. Permits quick contacts with the home office, field office, and key personnel on the job.
2. Reduces the time spent by key personnel, such as the superintendent, in rushing from one operation to another.
3. Saves time and cost by increasing the efficiency on a project.
4. Permits equipment to be shifted quickly from one operation to another, thereby reducing delays due to equipment failures, or reduces the amount of equipment required on a project because of the increased efficiency of use.
5. Permits quick contact with the shop in the event emergency repairs are required for equipment.
6. Expedites the distribution of materials to the different operations.

7. Gives excellent control between concrete-mixing plant and placing operations. In the event of a failure at either location the other can be notified immediately.

8. Permits quick calls for first aid or an ambulance in the event of injuries to personnel.

Three types of two-way radio sets are available:

1. Portable
2. Mobile
3. Base station

**Portable Sets.** Portable sets may be divided into two types, handset radiophones and packsets.

Handset radiophones weigh 10 to 20 lb and are equipped with wet or dry batteries having a net operating life of about 8 hr. The reception is good for a range of 1 to 2 miles, depending on the terrain and topography.

Packsets are larger than handsets and have longer-life batteries. Some have loudspeakers to permit an entire crew to listen to instructions. The range is about the same as for the handset.

**Mobile Units.** Mobile units are installed in trucks, automobiles, scrapers, loaders, at mixer plants, etc. They are operated by the vehicle battery. The power usually ranges from 10 to 60 watts. They have a greater range than the portable sets.

**Base Stations.** Base-station transmitters and receivers are installed at headquarters for the duration of the project. If a 50- to 60-ft antenna is used, it is possible to send and receive messages up to 60 miles or more. The power may be as high as 250 watts, with frequency modulation (FM). Such stations are classified as industrial applications and come under the supervision of the Federal Communications Commission (FCC). The assigned frequencies are so high that the waves travel essentially in a straight line.

In order to obtain a permit to install and operate a base station, it is necessary to submit to the FCC a request for approval, citing the need for a unit. It will be necessary to provide a second-class licensed radio operator at the main station to maintain the radios. Other persons who use the main station or mobile units must be issued licenses as third-class radio operators. The FCC requires weekly frequency checks and periodic reports on the station. Specific call letters and numbers are assigned and must be used at the beginning and end of each conversation.

**Construction-cost Control.** Few businesses can survive without a knowledge of costs and without an intelligent control of costs. Certainly this is true in the construction industry. A contractor may be an excellent builder, but unless he knows his construction costs, he will never survive the vigorous competition in the industry. If a manufacturer finds that he has lost money on certain items, he may be able to raise the



prices enough to assure a profit. However, a contractor who discovers after a project is finished that he has lost money may not have an opportunity to raise the price on the next project, especially if his losses were so great that he cannot finance another project. He may lose money because of one or several reasons, such as:

1. Low bid
2. Insufficient knowledge of job conditions
3. Increases in the costs of materials and labor
4. Adverse weather conditions
5. Improper selection of construction equipment
6. Inefficient management and supervision

While it may not be possible to correct the first four difficulties after the project is started, there may be some opportunity to improve item 5, and certainly an alert businessman should correct item 6, or better still he should not let it occur. Cost engineering or cost control will assist in correcting losses resulting from inefficient management and supervision. Cost control is more than mere bookkeeping. Bookkeeping will enable a contractor to determine whether he made a profit after a project is finished. Cost control during the period of construction will enable a contractor to analyze intelligently the performance of labor and equipment. It will show costs and production for labor and equipment. If the costs are higher than were estimated, either the estimate was too low or the costs are too high. If the latter condition is found to exist, it may be corrected while the project is still in operation, thereby providing a profit instead of a loss.

The owner of equipment should use an equipment ledger to provide information concerning each type of major equipment, showing an assigned number, with a description giving the size or capacity and any auxiliary equipment, date of purchase, name of seller, original total cost, estimated total life, and a depreciation schedule (see Figs. 2-9 and 2-10). He should use an equipment-operating ledger to keep a complete record of the cost of each type of equipment (see Fig. 2-12).

Prior to starting construction on a project a contractor should set up a classification of construction accounts in which specific item numbers are assigned to each construction operation. The item numbers that were used in estimating the cost of the project should be used in preparing the classification of construction accounts. This procedure will facilitate the comparison of costs with the original estimates. In setting up the items for which costs are to be estimated and reported during construction it is well to consider the desirability of dividing an operation into subitems. For example, the cost of concrete in a structure might be subdivided into the costs of producing aggregate, hauling aggregate, mixing and placing concrete, and finishing and curing concrete. If a concrete structure

includes various sizes and shapes whose costs vary considerably, it may be desirable to divide the project into subitems for cost purposes.

Cost accounts should provide for the showing of the costs of materials, labor, and equipment separately for each operation if they are to serve the purpose for which they are used. Some contractors follow the practice of grouping the cost of all equipment into one item. This practice is not good, as it does not permit a determination of the true complete cost of a given operation on which the equipment is used. This is especially true of engineering construction for which the cost of equipment may represent a major portion of the total cost. If the cost of equipment includes rental or depreciation, maintenance and repairs, fuel, supplies, etc., a record of the time that the equipment is used on each operation will permit the total cost to be prorated correctly between the several operations. It is not correct to charge to an operation the cost of major repairs because the equipment was assigned to that operation when the repairs were made.

Cost-accounting methods should be realistic, simple, and understandable. They are not an end product, but a means of managing a project. If the men who are supposed to use the information understand it, they will use it. If the information is too complicated, it will be disregarded or used incorrectly.

**Cost-control Records.** Experience gained on construction projects indicates that it is desirable to use simplified records for obtaining cost information. The forms illustrated in Fig. 2-14 to Fig. 2-19 are intended to show how cost information may be recorded and used in a simplified manner.

If the forms are made on the same-size ledger sheets, such as  $8\frac{1}{2}$  by 11 in., with two holes punched on the  $8\frac{1}{2}$ -in. side, all the records from a project may be assembled in one ledger binder when the project is completed. Such information will be of considerable value to estimators in preparing estimates for future projects.

Figure 2-14 is a form for keeping a record of the costs of materials purchased. As the net costs shown in the last column are cumulative through the last entry, it is possible to determine at a glance what the total cost of materials is for a given item.

Figure 2-15 is a form for keeping a record of the man-hours of labor used weekly and to date, the cost of labor per week and to date, the quantity of work completed per week and to date, the unit cost of work completed to date, and the estimated total cost based on the quantity and the indicated unit cost. The indicated saving or overrun is obtained by subtracting from the estimated total cost the product of the budget quantity times the unit cost to date for any desired date. It will be noted that the form is designed to show the item for which the costs apply, such as making forms for footings, piers, walls, and grade beams.



MATERIAL COST RECORD															
Item		Bought from		Description		Number		Budget		Unit		Total			
<u>Floor plank and nailers</u>								Estimated		quantity <u>136M</u>		cost <u>123</u>		cost <u>16,730</u>	





Figure 2-16 illustrates a daily timekeeper's field sheet which is designed to record the number of hours worked during a day by any number of men up to 50. If there are more than 50 men on a project, additional sheets may be prepared with numbers from 51 to 100, 101 to 150, etc. The numbers used on the sheets should coincide with the numbers assigned to the workers. The hours for each worker are shown under the proper classification of work performed, as indicated by the operation symbol near the top of the sheet. Near the bottom of the sheet the total number of man-hours for each operation and the corresponding cost are shown. The sheet provides a space for showing the wage rate per hour and the daily wage earned for each worker. The number of hours worked at regular wage rates and at overtime rates may be shown. The total man-hours shown across the bottom of the sheet should agree with the total hours shown in the vertical column as a check on the accuracy of entries and computations. In a similar manner the total of the amounts shown near the bottom of the sheet should agree with the total amount shown in the vertical column.

In using these forms it is suggested that a suitable system of symbols be adopted to indicate the various operations. In order to eliminate confusion, the system of symbols should be used uniformly throughout a contractor's operations on all projects. Thus, F1 might indicate forms for foundations, piers, and grade beams; F2, forms for floor beams and girders; F3, forms for floor and roof slab. If a further breakdown is desired, the symbols may be modified to F1M, making forms, F1E, erecting forms, F1S, stripping forms. These symbols are used on the timekeeper's field sheet.

Figure 2-17 illustrates a payroll record on which the information from the timekeeper's sheet may be permanently recorded. It will be noted that the extreme left and right columns are numbered in groups of 50 to correspond with the numbers used on the timekeeper's sheet. If the spacings of the numbers on the two sheets are made exactly the same, it will be possible to superimpose one sheet partially on the other to simplify the transfer of information from the timekeeper's sheet to the payroll record. In transferring overtime hours from the timekeeper's sheet the hours are shown in the vertical column headed "O.T." for the particular day. When the overtime hours are shown in the payroll-record column headed "Hours," they should be placed in the column headed " $1\frac{1}{2}$ " or "2," depending on which overtime rate applies. When overtime hours are extended to the column headed "Pay hr," they are multiplied by the appropriate factor, either  $1\frac{1}{2}$  or 2, before they are added to the hours at regular rates. This permits all pay hours to be multiplied by the regular wage rate, thereby eliminating the need of showing more than one wage rate for each worker. Under the column headed "Deductions," the

# TIMEKEEPER'S FIELD SHEET

Sheet No. 3  
Date 3/5/53

Job No. 531

Man's No.	Rate	Amount	Hours		No.
			Reg	O.T.	
1	1.50	12.00	8		1
2	1.50	12.00	8		2
3	1.50	12.00	8		3
4	1.50	12.00	8		4
5	1.50	12.00	8		5
48	1.50	12.00	8		48
49	1.50	12.00	8		49
50	1.00	8.00	8		50
Total man hours		584.00	400		Total

FIG. 2-16. Timekeeper's field sheet. (Courtesy of Irving H. Winslow.)



WEEKLY PAYROLL RECORD															Sheet No. <u>2</u>																	
Job No. <u>537</u>															For period from <u>3/5</u> to <u>3/11</u>																	
Payroll No. <u>10</u>																																
No.	OT brt fed	Thu		Fri		Sat		Sun		Mon		Tue		Wed		Hours		Pay hr	Rate	Gross amt	Deductions		Net amt	Name	WT class	Trade	Check No.	Date paid off	No.			
		R	OT	R	OT	R	OT	R	OT	R	OT	R	OT	R	OT	R	OT				R	OT								OAB	WT	
1		8		8		8		8		8		8		8		8		40	8	52	150	78.00	1.17	520	71.63	J.T. Brown	5	Carpenter	521			
2		8		8		8		8		8		8		8		8		40	8	60	150	90.00	1.35	10.20	78.45	D. Jones	3	Carpenter	522			
3		8		8		8		8		8		8		8		8		40	8	48	150	72.00	1.08	9.40	61.52	C.L. Smith	1	Carpenter	523			
4																																
5																																
6																																
7																																
8																																
9																																
10																																
44																																
45																																
46																																
47																																
48																																
49																																
50																																

R, hours worked at regular rates  
 OT, hours worked at overtime rate  
 OAB, old age benefits  
 WT, withholding tax

Fig. 2-17. Weekly payroll record. (Courtesy of Irving H. Winalow.)

LABOR STATEMENT											
Job No. <u>531</u>			Statement No. <u>11</u> through <u>3/11</u>						Sheet No. <u>2</u>		
Symbol	Description	Week's cost	Quantity			Unit cost		Cost		Estimated	
			Unit of meas	Est total	Actual to date	Est	Actual to date	Est total	Actual to date	Probable final	Saving
	<i>Forms (Make, erect, strip)</i>										
F1M	<i>Footings,</i>		Sq	63	63	8.00	5.94	504	374	130	
F1E	<i>piers, walls,</i>		Sq	173	173	30.00	27.00	5190	4671	519	
F1S	<i>grade beams</i>	36.00	Sq	173	124	6.00	6.49	1038	805		85
F2M	<i>Slabs, beams,</i>	160.99	Sq	60	32	10.00	10.32	600	330		19
F2E	<i>girders</i>	210.68	Sq	93	17	27.00	28.09	2511	478		101
F2S	<i>Slabs, beams, girders</i>		Sq	93		10.00		930			

FIG. 2-18. Weekly labor-cost statement. (Courtesy of Irving H. Winalow.)



MATERIAL STATEMENT										Sheet No. <u>2</u>
Job No. <u>531</u>		Statement No. <u>1</u> through <u>4/1</u>								
Description	Unit of meas	Estimated			Actual to date			Probable final cost	Indicated	
		Quantity	Unit cost	Total cost	Quantity	Unit cost	Total cost		Saving	Overrun
Steel and iron										
Steel reinforcement and sundry	Ton	27	110	2,970	26	100	2,612	2,700	270	
Steel sash	Sq ft	5,580	0.75	4,180	5,580	0.69	3,866	3,866	314	
Miscellaneous iron				1,600			116	1,600		
Total				8,750			6,594	8,166	584	
Concrete and masonry										
Ready-mixed concrete	Cu yd	449	12.70	5,700	275	12.10	3,332	5,433	267	
Brick-common red	M	194	27.60	5,360				5,360		

FIG. 2-19. Material-cost statement. (Courtesy of Irving H. Winslow.)

amounts deducted for old-age benefits and for withholding taxes are shown.

The days shown at the head of the several columns may be revised, as desired, to permit any given day to indicate the end of a week, or the names of the days may be replaced by numbers such as 3/15, 3/16, 3/17, etc.

Figure 2-18 illustrates a weekly labor-cost statement on which there are recorded the number of units of work completed, the estimated and actual unit cost to date, and the probable final cost of each operation. The probable final cost is obtained by multiplying the estimated total quantity by the actual unit cost to date. From the probable final cost of a given operation it is possible to estimate the saving or overrun, based on the original estimated cost of the operation. This information may be of considerable value to a contractor in determining the cost status of a project at any time during the period of construction. It will be useful to the estimating department in preparing estimates for future projects.

Figure 2-19 illustrates a material-cost statement which may be prepared weekly or monthly. It assists a contractor by enabling him to determine the status of his material costs through any desired date during construction.

### PROBLEMS

**2-1.** Prepare a construction schedule for a project involving the drilling of water wells, installing pumps in the wells, laying cast-iron water pipe, building a pump house, installing booster pumps, and constructing a storage reservoir at ground level. The project will be started on the first Monday in March of the current year. A nominal week will be 5 days of 8 hr each. For each operation add 20 to 30 per cent for time lost due to weather.

In preparing the schedule show the operation, quantity, rate, total time in weeks, and the schedule by months and weeks. Use Saturday to designate the end of each week.

The quantities and rates of construction are as follows:

Operation	Quantity	Rate of const.
Move in.....	1	1 wk
Drill wells.....	2 each	3 wk each
Install pumps.....	2 each	1 wk each
Lay cast-iron pipe.....	28,860 lin ft	40 ft per hr
Build pump house.....	1 each	10 wk
Install pumps.....	2 each	1 wk each
Build reservoir.....	1 each	12 wk
Run tests.....	1 each	1 wk
Cleanup.....	1 each	1 wk

**2-2.** Prepare a construction schedule for a given highway project. The project will be started on the second Monday in April of the current year. A week will be



## 42 CONSTRUCTION PLANNING, EQUIPMENT, AND METHODS

6 days of 8 hr each. For each operation add 20 to 30 per cent for the time lost due to weather. Divide the time into months and weeks, showing each week to end on Saturday.

The concrete pavement will be exactly 8 miles long, 24 ft wide, and 9 in. average thickness. The quantity of pavement should be expressed in square yards.

The specifications require that each drainage structure shall be completed at least 14 days before any earth fill is placed on it. There must be a lapse of at least 30 days between the completion of earthwork and the placing of pavement for any section of the highway. The pavement must be cured for at least 7 days after it is placed.

Base your schedule on the following information:

Operation	Quantity	Unit	No. units of equipment used	Output per unit of equipment
Move in.....	.....	.....	..	Assume 1 wk required
Clearing and grubbing..	84	Acres	2	$\frac{1}{2}$ acre per day
Earthwork.....	105,600	Cu yd	3	80 cu yd per hr
Drainage structures....	16	Each	..	1 per wk
Concrete pavement.....	.....	Sq yd	1	80 cu yd per hr
Cleanup and move out..	.....	.....	..	Assume 3 wk required

**2-3.** Prepare a construction schedule for a reinforced-concrete building consisting of the following operations and quantities:

Operation	Quantity	Rate of const.
Move in.....	1	1 wk
Excavation, machine.....	460 cu yd	20 cu yd per hr
Excavation, hand.....	32 cu yd	2 cu yd per hr
Foundation concrete.....	286 cu yd	50 cu yd per wk
Structural concrete.....	1,160 cu yd	90 cu yd per wk
Masonry, concrete blocks.....	240 M	12 M per wk
Masonry, brick.....	360 M	15 M per wk
Windows.....	180 each	20 per wk
Doors and millwork.....	96 each	16 per wk
Plastering.....	3,680 sq yd	240 sq yd per wk
Painting, 3 coats.....	398 squares	Along with job
Roofing.....	86 squares	40 squares per wk
Electrical.....	1 lot	Along with job
Plumbing.....	1 lot	Along with job
Cleanup and move out.....	1	1 wk

The project will be started on the third Monday in June of the current year. A week will be 5 days of 8 hr each. Allow 20 to 30 per cent extra time for each operation through doors and millwork for time lost due to weather.

The brick and concrete blocks will be installed at approximately the same time. The windows, which are steel sash, will be installed with the masonry walls. The

doors and millwork will be installed after the building is substantially finished. The painting, which will be applied to all interior surfaces, doors, and windows, will be started when the roofing is completed.

The construction schedule should show the following items:

Operation

Quantity

Rate per week

Total time for each operation, in weeks

Chart showing the time of starting and finishing each operation

**2-4.** Prepare a project layout showing where you would locate the office, warehouse, form lumber, reinforcing steel, sand, gravel, and cement, concrete mixer, and hoisting tower for the project described below. The 300-ft side of the site lies along a paved street. All materials will be delivered by truck. Concrete will be mixed at the job.

The following information will apply:

Size of site,  $300 \times 200$  ft

Size of structure,  $100 \times 80$  ft

Depth of excavation for structure, 10 ft

Height of structure above ground, 26 ft

The structure will be located near center of site

The earth will be hauled away as it is excavated



## CHAPTER 3

### FACTORS AFFECTING THE SELECTION OF CONSTRUCTION EQUIPMENT

**General Information.** A problem which frequently confronts a contractor as he plans to construct a project is the selection of the most suitable equipment. He should consider the money spent for equipment as an investment which he can expect to recover, with a profit, during the useful life of the equipment. A contractor does not pay for construction equipment: the equipment must pay for itself by earning for the contractor more money than it costs. Unless it can be established in advance that a unit of equipment will earn more than the cost, it should not be purchased.

A contractor can never afford to own all types or sizes of equipment that might be used for the kind of work he does. It may be possible to determine what kind and size of equipment seems most suitable for a given project, but this information alone will not necessarily justify the purchase of the equipment. Perhaps the project under consideration is not large enough to justify the purchase, as the cost cannot be recovered before the completion of the project and it may not be possible to dispose of the equipment at the completion of the project at a reasonable price. A contractor may own a type of equipment, which is presently idle, that is less desirable than the proposed equipment, but, considering the probable heavy depreciation for the proposed equipment and the uncertainty that it can be used on future projects, the apparently ideal equipment may prove to be more expensive than equipment now owned by the contractor.

Any time a unit of equipment will pay for itself on work that is certain to be done it is good business to purchase it. For example, if a unit of equipment costing \$25,000 will save \$50,000 on a project, a contractor is justified in purchasing it, regardless of the prospects of using it on additional projects or the prospects of selling it at a favorable price when the project is finished.

**Standard Types of Equipment.** There is no clear definition of standard equipment. Equipment that is standard for one contractor may be special equipment for another contractor. It depends on the extent to which a contractor will use it in his construction operations. Another

method which is sometimes used to distinguish between standard and special equipment is the extent to which it is commonly manufactured and available to prospective purchasers. Thus, a 1-cu-yd diesel-powered crawler-mounted power shovel is standard equipment, whereas a 30-cu-yd shovel is classified as special equipment. The larger shovel is manufactured for a specific purchaser.

Contractors should confine their purchases to standard equipment unless a project definitely justifies the purchase of special equipment. Delivery of standard equipment may be obtained more quickly. Standard equipment can be used economically on more than one project. Repair parts for standard equipment may be obtained more quickly and

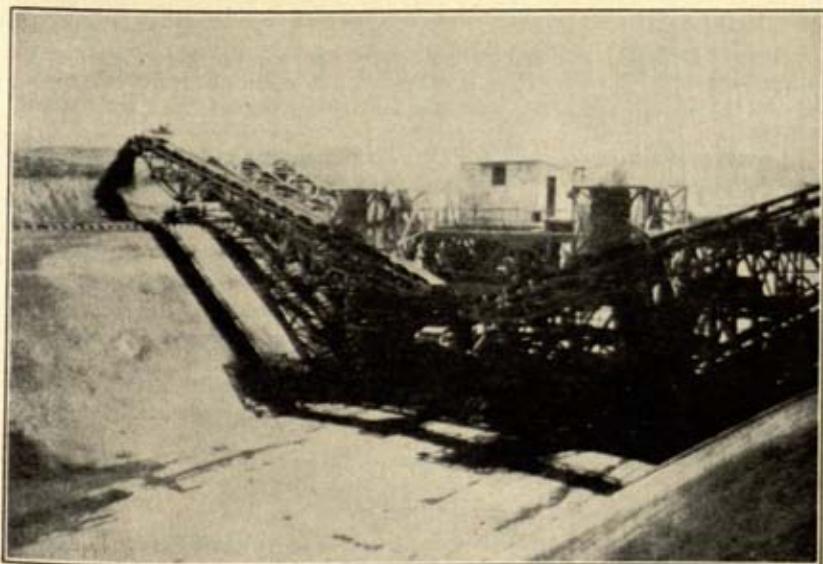


FIG. 3-1. Canal trimmer. (*Western Contracting Corporation.*)

economically than for special equipment. If a contractor no longer needs a unit of standard equipment, he can usually dispose of it more easily and at a more favorable price.

**Special Equipment.** One definition of special equipment is equipment that is manufactured for use on a single project or for a special type of operation. Such equipment may not be suitable or economical for use on another project. An example of special equipment is a 40-cu-yd power shovel used to remove the overburden in strip mining coal. Another example is the hydraulic dredge which was constructed primarily for use in building the Ft. Randall Dam. Another special type of equipment is



the canal trimmer illustrated in Fig. 3-1. This equipment is used for final trimming the bottom and sides of an earthen canal, prior to placing the watertight membrane, such as concrete or asphalt surfacing. However, as this type of equipment is becoming more common in canal construction, it might be considered as standard. Although belt-conveyor systems sometimes are used to transport aggregates several miles in constructing dams, such installations probably should be considered as special equipment.

An example of a study to determine the economy of transporting aggregate by trucks or by a belt-conveyor system is given below:

**EXAMPLE.** The project is a concrete dam requiring 1,200,000 tons of crushed-stone aggregate. If the aggregate is hauled from the quarry to the project by trucks, it will be necessary to build a haul road with several bridge structures at an estimated total cost of \$280,000. It is doubtful that any salvage value can be realized from the bridge structures after the project is completed. It is estimated that the cost of hauling the aggregate, including the cost of maintaining the haul road, will be \$0.10 per ton.

If the aggregate is transported by a belt-conveyor system, it is estimated that the cost of the system will be \$390,000. As the belts, rollers, electric motors, and parts of the supporting structures can be used on other projects it is estimated that the system will have a salvage value of \$60,000.

In estimating the cost of hauling the aggregate with trucks the job planner should include in his estimate all costs of operating the trucks, such as depreciation, maintenance, repairs, investment, tires, fuel, lubrication, and operator's wages, as illustrated on pages 178 to 181. Also, the planner should include the estimated costs of maintaining the haul road during the period of construction.

The cost of operating the belt-conveyor system should include depreciation, investment, maintenance, and repairs for the entire system, plus the cost of electrical energy and labor.

The total estimated cost and the cost per ton will be

Using trucks:	
Haul road	= \$280,000
Hauling, 1,200,000 tons @ \$0.10 per ton	= 120,000
Total cost	= \$400,000
Cost per ton, $\$400,000 \div 1,200,000$ tons	= 0.333
Using a belt conveyor:	
Belt-conveyor system	= \$390,000
Transporting, 1,200,000 tons @ \$0.07 per ton	= 84,000
Total cost	= \$474,000
Less salvage value of system	= 60,000
Net total cost	= \$414,000
Cost per ton, $\$414,000 \div 1,200,000$ tons	= 0.345

The analysis indicates that for this project the use of a belt-conveyor system is not justified. However, if the quantity of aggregate is increased to 2,400,000 tons, the belt-conveyor system is justified. The total cost and the cost per ton will be

## Using trucks:

Haul road	= \$280,000
Hauling, 2,400,000 tons @ \$0.10 per ton	= 240,000
Total cost	= \$520,000
Cost per ton, $\$520,000 \div 2,400,000$ tons	= 0.217

## Using a belt conveyor:

Belt-conveyor system	= \$390,000
Transporting, 2,400,000 tons @ \$0.07 per ton	= 168,000
Total cost	= \$558,000
Less salvage value of system	= 60,000
Net total cost	= \$498,000
Cost per ton, $\$498,000 \div 2,400,000$ tons	= 0.207

**EXAMPLE.** Another example of a study which was made to determine the economy of using special equipment involved trimming a canal prior to placing a concrete lining. The soil was a sandy clay. The canal had a 14-ft-wide bottom, 14 ft depth, and 1:1 side slopes. On similar projects the cost of trimming the bottom and slopes, using draglines, bulldozers, graders, and hand labor, had varied from \$0.30 to \$0.40 per square yard. A specially constructed self-propelled trimmer, similar to the one illustrated in Fig. 3-1, was estimated to cost \$68,000. Auxiliary equipment was estimated to cost \$24,000. It was estimated that the equipment could be used 1,600 hr per year for a total of 4 years. The equipment should trim an average of 50 lin ft of canal per hour.

The estimated cost of trimming per square yard was determined as shown below:

Width of canal, including slopes and bottom, 52 ft	
Speed, 50 ft per hr	
Area, $50 \times 52 \div 9 = 289$ sq yd per hr	
Total cost of equipment, \$92,000	
Cost per hr	
Equipment	= \$39.30
Labor	= 24.00
Total cost	= \$63.30
Cost per sq yd, $\$63.30 \div 289$	= 0.219

From the above analysis it is evident that the purchase of the special trimmer is justified provided the equipment will be used 4 years. During this period the equipment will trim  $1,600 \text{ hr} \times 50 \text{ ft per hr} \div 5,280 \text{ ft per mile} = 15.15$  miles per year. How many miles of canal must be trimmed to justify the purchase of the equipment?

The number of miles of canal to be trimmed to justify the purchase of a mechanical trimmer will vary with the cost of trimming by other equipment and methods, the cost of trimming with the mechanical trimmer, the cross section of the canal, and the number of hours the trimmer will be used per year. For the equipment described in the previous analysis, the determination would be made as follows:

Estimated cost of trimming without using special trimmer, \$0.30 per sq yd  
 Cost of special trimmer, \$92,000  
 Est. life, 4 yr  
 Est. hr used per yr, 1,600



## Annual costs

Depreciation, $\$92,000 \div 4$	= \$23,000
Maintenance and repairs @ 100% of depreciation, $\$23,000 \times 1.0$	= 23,000
Investment, 10% of average value, $\$92,000 \times 0.625^* \times 0.10$	= 5,750
Fuel, lubrication, and other expenses, 1,600 hr @ \$7.00 per hr	= 11,200
Total annual cost	= \$62,950
Cost per hr, $\$62,950 \div 1,600$ hr	= 39.30

The annual costs excluding depreciation are

Maintenance and repairs	= \$23,000
Investment	= 5,750
Fuel, lubrication, and other expenses	= 11,200
Total annual cost	= \$39,950
Cost per hr, $\$39,950 \div 1,600$	= 24.96

As the annual cost of depreciation, amounting to \$23,000, represents a loss in the value of the equipment to the owner, it will be necessary for the trimmer to save \$23,000 per year for 4 years to just break even on the initial cost of the equipment. Based on an hourly cost of \$24.96, excluding depreciation, the cost of trimming the canal will be

Area trimmed per hr, $50 \times 52 \div 9$	= 289 sq yd
Equipment cost per hr	= \$24.96
Labor cost per hr	= 24.00
Total cost per hr	= \$48.96
Cost per sq yd, $\$48.96 \div 289$	= 0.169
Cost per sq yd without trimmer	= 0.30
Saving in cost per sq yd due to trimmer	= 0.131
No. sq yd required per year, $\$23,000 \div 0.131$	= 175,572
No. sq yd per ft of canal, $52 \div 9$	= 5.78
Length of canal to give 175,572 sq yd, $175,572 \div 5.78$	= 30,400 ft
= $30,400 \div 5,280$ ft per mile	= 5.78 miles

Thus, it is determined that it will be necessary to trim at least 5.78 miles per year for 4 years, or a total of 23.0 miles, to justify the purchase of the special trimmer. If the conditions are different from those given, a revised analysis should be made in a similar manner. This method may be used to determine the justification for purchasing any type of special equipment.

**Replacement of Parts.** A factor which may be overlooked by a prospective purchaser of equipment is the ease and speed with which replacement parts may be obtained. All equipment parts are subject to failure, regardless of the care which they receive. A truck with a broken axle is useless until the axle is replaced. A broken part in a power shovel may delay an entire project for weeks, while waiting for the part to be manufactured and shipped. Prior to purchasing equipment, the buyer should determine where spare parts are obtainable. If parts are not obtainable quickly, it may be wise to purchase other equipment, for which parts are quickly available, even though the latter seems less desirable. This is an argument for standard equipment.

\* See Table 3-1.

**The Cost of Owning and Operating Construction Equipment.** There are several methods of determining the probable cost of owning and operating construction equipment. No known method will give exact costs under all operating conditions. At best the estimate is only a close approximation of the cost. Carefully kept records for equipment previously used should give information which may be used as a guide for the particular equipment. But there is no assurance that similar equipment will have a similar cost experience, especially if it is used under different conditions. Factors which affect the cost of owning and operating equipment include the cost of the equipment delivered to the owner, the severity of the conditions under which it is used, the number of hours it is used per year, the number of years it is used, the care with which the owner maintains and repairs it, and the demand for used equipment when it is sold, which will affect the salvage value.

When it is necessary to estimate the cost of owning and operating equipment prior to purchasing it, cost records, based on past performance, will not be available. The costs which should be considered include depreciation, maintenance, repairs, investment, lubrication, and fuel, if fuel is required to operate it.

**Depreciation.** When a unit of equipment is placed in operation, it begins to wear out. Regardless of the care in maintaining and repairing it, the equipment will ultimately wear out or become obsolete and should be replaced. Thus, depreciation is the certain march to the junk yard. The owner of equipment should provide a reserve fund to replace it when it is worn out. The most common method of determining the cost of depreciation is to assume a useful life for the equipment, expressed in years, hours, or units of production, whichever seems most appropriate or desirable. If the total cost of the equipment is divided by the estimated useful life, the result will be the depreciation per year, hour, or unit of production. If a unit of equipment costs \$10,000 and is estimated to have a useful life of 10,000 hr, the cost of depreciation will be \$1.00 per hour, or if the equipment will produce an estimated 100,000 units during its life, the cost of depreciation will be  $\$10,000 \div 100,000 = \$0.10$  per unit. This is referred to as straight-line depreciation.

A contractor who fails to include in his estimate an appropriate allowance for depreciation of his equipment will find that he has worn out his equipment and has no funds with which to replace it. The amount charged for depreciation should not be considered as a profit.

**Maintenance and Repairs.** The cost of maintenance and repairs will vary considerably with the type of equipment, the service to which it is assigned, and the care which it receives. If a bearing is greased and adjusted at frequent intervals, its life will be much longer than if it is neglected.



The annual cost of maintenance and repairs may be expressed as a per cent of the annual cost of depreciation, or it may be expressed independently of depreciation. In any event, it should be sufficient to cover the cost of keeping the equipment operating. The annual cost of maintenance and repairs for a power shovel may vary from 80 to 120 per cent of the annual cost of depreciation, with 100 per cent a fair average value. The annual cost for certain types of rock-crushing equipment may be much higher, while for an electric motor it will be lower. Experience records serve as a guide in estimating these costs. Appendix A gives representative estimates of the costs of maintenance and repairs for construction equipment.

**Investment Costs.** It costs money to own equipment, regardless of the extent to which it is used. These costs, which are frequently classified as investment costs, include interest on the money invested, taxes of all types which are assessed against the equipment, insurance, and storage. The rates for these items will vary somewhat among different owners, with location, and for other reasons.

There are several methods of determining the cost of interest paid on the money invested in equipment. Even though the owner pays cash for the equipment, he should charge interest on the investment, as the money spent for the equipment could be invested in some other asset which would return interest to the owner. The possibility of earning interest on money is lost to the equipment owner when he spends the money for equipment.

Some equipment owners charge a fixed rate of interest against the full purchase cost of the equipment each year it is owned. This method gives an annual interest cost which is higher than it should be. Each year that equipment is used the owner should deduct from its earnings an amount equal to the annual cost of depreciation. Since this money is retained by the owner, it reduces his net investment in the equipment. After the equipment has been used for the estimated depreciation period, expressed in years or units of production, the owner will have recovered its original cost through the reserve for depreciation. In any event, the interest charged should be based on a realistic value for the equipment, instead of its original cost.

The average annual cost of interest should be based on the average value of the equipment during its useful life. This value may be obtained by establishing a schedule of values for the beginning of each year that the equipment will be used. The calculations given below illustrate a method of determining the average value of equipment:

Original cost of equipment, \$25,000

Est. useful life, 5 yr

Average annual cost of depreciation,  $\$25,000 \div 5 = \$5,000$

Beginning of year	Cumulative depreciation	Value of equipment
(1)	(2)	(3)
1	0	\$25,000
2	\$ 5,000	20,000
3	10,000	15,000
4	15,000	10,000
5	20,000	5,000
6	25,000	0

Total of values in col. 3 = \$75,000

Average value, \$75,000 ÷ 5 = \$15,000

Average value as % of original cost,  $\frac{\$15,000 \times 100}{\$25,000} = 60$

Thus, the average value of equipment having an estimated life of 5 years is 60 per cent of the original cost.

In a similar manner it can be shown that the average value of equipment having a life of 4 years is 62.5 per cent of the original cost, and for equipment having a life of 6 years it is 58.33 per cent.

A formula which will give the average value of equipment as a per cent of the original cost is

$$\text{Average value} = \frac{(1 + n) \times 100}{2n} = \% \text{ of original cost} \quad (3-1)$$

where  $n$  is the number of years in the depreciation period.

A schedule of average values for equipment for various years of life is given in Table 3-1.

TABLE 3-1. AVERAGE VALUE OF EQUIPMENT

Estimated life, yr	Average value as % of original cost
2	75.00
3	66.67
4	62.50
5	60.00
6	58.33
7	57.14
8	56.25
9	55.55
10	55.00
11	54.54
12	54.17

Insurance and taxes are usually paid on the depreciated value of the equipment. Therefore, it is proper to use the average value of equipment in determining the average annual cost of insurance and taxes.



It is common practice to combine interest, insurance, taxes, and storage costs and to estimate them as a fixed per cent of the average investment in the equipment. Surveys made on a nationwide basis indicate an average combined rate of 10 per cent, which includes interest at 6 per cent and insurance, taxes, and storage at 4 per cent.

**Operating Costs.** Construction equipment which is driven by internal-combustion engines requires fuel and lubricating oil, which should be considered as an operating cost. While the amounts consumed and the unit cost of each will vary with the type of equipment, the conditions under which it is used, and location, it is possible to estimate the cost reasonably accurately for a given project.

The person who is responsible for selecting the equipment should estimate the conditions under which the equipment will operate. There are at least two conditions which will apply to most projects, the extent to which the engine will operate at full power all the time, and the actual time that the unit will operate in an hour or a day.

While the power unit in a piece of equipment may be capable of developing a given horsepower when operating at maximum output, it is well known that maximum output usually will not be required at all times. For example, the full power of an engine may be required while a power shovel is loading the dipper, but during the balance of the cycle the demands on the engine are reduced considerably. The full power of a tractor will be required while it is loading a scraper with earth, and possibly while it is climbing an embankment, but for the rest of the round-trip cycle it is probable that less than the maximum power will be required. Consider the gasoline-engine-driven air compressor that is heard so frequently. For a short time the engine will operate at full power, then it will idle for a while, these conditions alternating as the air is used.

**Fuel Consumed.** As the quantity of fuel consumed in an hour by an internal-combustion engine depends on the average power supplied, it is necessary to estimate the average power that will be required of the engine. This may be expressed as a per cent of the maximum power. For most construction equipment the actual average power will vary from 50 to 90 per cent of the maximum power.

Another factor which will affect the amount of fuel consumed in an hour is the length of time that the equipment will actually operate in an hour. On most projects there will be delays and interruptions which prevent the equipment from operating continuously. The extent of the delays will vary with the project. The failure of a wire rope on a power shovel may delay the shovel and trucks for an hour or more. Operators must stop for relief periodically, to get a drink of water, or to check the equipment. The actual time that equipment is operated may vary from 45 to 55 min per hr, or in some instances it may be less.

A gasoline engine will consume approximately 0.06 gal of fuel per horsepower per hour. This quantity is subject to variation with altitude, temperature, and climatic conditions. The power of a gasoline engine will decrease approximately 3 per cent for each 1,000 ft of altitude above sea level.

It is desired to estimate the quantity of fuel consumed per hour by a 2-cu-yd power shovel with a 160-hp gasoline engine. An analysis of the job conditions indicates that the engine will operate at 80 per cent of its rated horsepower and that the unit will operate 50 min per hr on the average. The time factor is  $\frac{50}{60} \times 100 = 83.3$  per cent. The combined operating factor =  $0.8 \times 0.833 \times 100 = 66.6$  per cent. The probable quantity of fuel consumed per hour will be

At 100% operating factor,  $0.06 \times 160 = 9.6$  gal

At 66.6% operating factor,  $0.666 \times 9.6 = 6.4$  gal

A diesel engine will consume approximately 0.04 gal of fuel per horsepower per hour. As for a gasoline engine, this quantity will vary with altitude, temperature, and climatic conditions. The power of a four-cycle diesel engine will decrease about 3 per cent, and a two-cycle about 1 per cent, for each 1,000 ft of altitude above sea level. If a supercharger is used, this loss will not occur.

If the power shovel discussed heretofore is equipped with a diesel engine of the same horsepower and the operating conditions remain the same, the probable quantity of fuel consumed per hour will be

At 100% operating factor,  $0.04 \times 160 = 6.4$  gal

At 66.6% operating factor,  $0.666 \times 6.4 = 4.3$  gal

For other operating factors the quantity of fuel consumed should be determined in a similar manner.

**Lubricating Oil.** The quantity of lubricating oil used by an engine will vary with the size of the engine, the capacity of the crankcase, the condition of the piston rings, and the number of hours between oil changes. For extremely dusty operations it may be desirable to change oil every 50 hr, but this is an unusual condition. It is common practice to change oil every 100 to 200 hr. The quantity of oil consumed by an engine per change will include the amount added during the change plus the make-up oil between changes.

A formula which may be used to estimate the quantity of oil required is

$$q = \frac{\text{hp} \times 0.6 \times 0.006 \text{ lb per hp-hr}}{7.4 \text{ lb per gal}} + \frac{c}{t} \quad (3-2)$$



where  $q$  = quantity consumed, gph  
 hp = rated horsepower of engine  
 $c$  = capacity of crankcase, gal  
 $t$  = no. hours between changes

The above formula is based on an operating factor of 60 per cent. It assumes that the quantity of oil consumed per rated hp-hr, between changes, will be 0.006 gal. Using the formula, for a 100-hp engine with a crankcase capacity of 4 gal, requiring a change every 100 hr, the quantity consumed per hour will be

$$q = \frac{100 \times 0.6 \times 0.006}{7.4} + \frac{4}{100} = 0.049 + 0.04 = 0.089 \text{ gal}$$

### Examples Illustrating the Cost of Owning and Operating Construction Equipment

**EXAMPLE.** Determine the probable cost per hour of owning and operating a 2-cu-yd diesel-engine-powered crawler-type power shovel. The following information will apply:

Engine, 160 hp	
Crankcase capacity, 6 gal	
Hr between oil changes, 100	
Operating factor, 60%	
Fuel consumed per hr, $0.6 \times 0.04 \times 160 = 3.9$ gal	
Lubricating oil consumed per hr, $\frac{160 \times 0.6 \times 0.006}{7.4} + \frac{6}{100} = 0.138$ gal	
Shipping weight, 136,000 lb	
Useful life, 6 yr	
Hr used per yr, 2,000	
Cost to owner:	
List price f.o.b. the factory	= \$54,450
Freight, 136,000 lb @ \$1.50 per cwt	= 2,040
Unloading and assembling	= 110
Total cost to owner	= \$56,600
Average investment, $0.5833 \times \$56,600$	= 33,015
Annual cost:	
Depreciation, $\$56,600 \div 6$ yr	= \$ 9,433
Maintenance and repairs, 100% of depreciation	= 9,433
Investment, 10% of \$33,015	= 3,301
Total annual fixed cost	= \$22,167
Hourly cost:	
Fixed cost, $\$22,167 \div 2,000$ hr	= \$11.08
Fuel, 3.9 gal @ \$0.15	= 0.59
Lubricating oil, 0.138 gal @ \$1.00	= 0.14
Total cost per hr, excluding labor	= \$11.81

EXAMPLE. Determine the probable cost per hour of owning and operating a 15-cu-yd heaped-capacity bottom-dump wagon with six rubber tires. The following information will apply:

Engine, 200 hp, diesel	
Crankcase capacity, 11 gal	
Hr between oil changes, 80	
Operating factor, 67 %	
Fuel consumed per hr, $0.67 \times 0.04 \times 200 = 5.6$ gal	
Lubricating oil consumed per hr, $\frac{200 \times 0.67 \times 0.006}{7.4} + \frac{11}{80} = 0.246$ gal	
Shipping weight, 33,000 lb	
Useful life, 5 yr	
Hr used per yr, 2,000	
Maintenance and repairs, 50% of depreciation	
Life of tires, 5,000 hr	
Repairs to tires, 15% of depreciation of tires	
Cost to owner:	
List price f.o.b. the factory	= \$25,860
Freight, 33,000 lb @ \$1.50 per cwt	= 495
Total cost to owner	= \$26,355
Less value of tires	6,560
Net cost, less tires	= \$19,795
Average investment, $0.6 \times \$26,355$	= 15,813
Annual cost:	
Depreciation, $\$19,795 \div 5$ yr	= \$ 3,959
Maintenance and repairs, 50% of \$3,959	= 1,980
Investment, 10% of \$15,813	= 1,581
Total annual fixed cost	= \$ 7,520
Hourly cost:	
Fixed cost, $\$7,520 \div 2,000$ hr	= \$ 3.76
Tire depreciation, $\$6,560 \div 5,000$ hr	= 1.31
Tire repairs, 15% of \$1.31	= 0.20
Fuel, 5.6 gal @ \$0.15	= 0.84
Lubricating oil, 0.246 gal @ \$1.00	= 0.25
Total cost per hr, excluding labor	= \$ 6.36

EXAMPLE. Determine the probable cost per hour of owning and operating an earth loader and the crawler tractor required to pull it. The following information will apply:

Tractor:

Engine, 144 hp, diesel	
Crankcase capacity, 9 gal	
Hr between oil changes, 100	
Operating factor, 70 %	
Fuel consumed per hr, $0.7 \times 0.04 \times 144 = 4.1$ gal	
Lubricating oil consumed per hr, $\frac{144 \times 0.7 \times 0.006}{7.4} + \frac{9}{100} = 0.172$ gal	



Shipping weight, 35,000 lb

Useful life, 5 yr

Hr used per yr, 2,000

Maintenance and repairs, 75% of depreciation

Cost to owner:

List price f.o.b. the factory = \$14,600

Freight, 35,000 lb @ \$1.50 per cwt = 525

Total cost to owner = \$15,125

Average investment,  $0.6 \times \$15,125$  = 9,075

Annual cost:

Depreciation,  $\$15,125 \div 5$  yr = \$ 3,025

Maintenance and repairs, 75% of \$3,025 = 2,269

Investment, 10% of \$9,075 = 908

Total annual fixed cost = \$ 6,202

Hourly cost:

Fixed cost,  $\$6,202 \div 2,000$  hr = \$ 3.10

Fuel, 4.1 gal @ \$0.15 = 0.62

Lubricating oil, 0.172 gal @ \$1.00 = 0.17

Total cost per hr, excluding labor = \$ 3.89

Earth loader:

Cutting edge, maximum width, 9 ft 6 in.

Cut, maximum depth, 48 in.

Average cut, 18 in. deep, 4 ft wide

Belt width, 54 in.

Belt length, 74 ft

Engine, 190 hp, diesel

Crankcase capacity, 5 gal

Hr between oil changes, 100

Operating factor, 70%

Fuel consumed per hr,  $0.7 \times 0.04 \times 190 = 5.3$  galLubricating oil consumed per hr,  $\frac{190 \times 0.7 \times 0.006}{7.4} + \frac{5}{100} = 0.158$  gal

Shipping weight, 55,000 lb

Useful life, 7 yr

Hr used per yr, 2,000

Cost to owner:

List price f.o.b. the factory = \$33,500

Freight, 55,000 lb @ \$1.50 per cwt = 825

Total cost to owner = \$34,325

Less original cost of belt = 2,300

Net cost, less belt = \$32,025

Average investment,  $0.571 \times \$34,325$  = 19,600

Annual cost:

Depreciation,  $\$32,025 \div 7$  yr = \$ 4,575

Investment, 10% of \$19,600 = 1,960

Total annual fixed cost = \$ 6,535

Hourly cost:

Fixed costs,  $\$6,535 \div 2,000$  hr = \$ 3.27

Operating costs will vary with conditions, which may be adverse, average, or favorable. The estimated cost is given for each of these conditions, respectively.

	Cost per hr		
	Adverse conditions	Average conditions	Favorable conditions
Belt @ \$2,300 for 500-1,000-1,500 hr.....	\$ 4.60	\$2.30	\$1.54
Points @ \$26.00 for 50-125-200 hr.....	0.52	0.21	0.13
Plow @ \$280.00 for 150-375-600 hr.....	1.87	0.74	0.47
Cutting edges @ \$150.00 for 150-375-600 hr....	1.00	0.40	0.25
Repairs, parts and labor.....	1.45	1.04	0.83
Fuel @ \$ 0.15 per gal.....	0.80	0.60	0.40
Lubricating oil and grease.....	0.20	0.20	0.20
Total operating cost per hr.....	\$10.44	\$5.49	\$3.82
Fixed cost per hr.....	3.27	3.27	3.27
Total cost per hr.....	\$13.71	\$8.76	\$7.09

The combined cost per hour for tractor and loader is

	Cost per hr		
	Adverse conditions	Average conditions	Favorable conditions
Tractor.....	\$ 3.89	\$ 3.89	\$ 3.89
Loader.....	13.71	8.76	7.09
Total cost.....	\$17.60	\$12.65	\$10.93

The hourly cost of owning and operating construction equipment, as illustrated in the previous examples, will vary with the conditions under which the equipment is operated, and the job planner should analyze each job to determine the probable conditions.

If a power shovel is used to excavate a soft material, the life of the dipper teeth, wire rope, and other parts which are affected by the wear and strain will be relatively long. Repair costs will be relatively low. However, if the shovel is used to excavate rock or other hard materials, the dipper teeth, wire rope, clutch linings, and certain gear parts will be subjected to greater strains and the life of each will be reduced. Repair costs will be correspondingly increased. Likewise, the consumption of fuel will be affected by digging conditions.

If trucks are operated over straight, reasonably level, smooth roads, the cost of repairs will be lower than when the same trucks are operated over poorly maintained roads, with steep hills, ruts, or deep sand. A study of statistical information showing the cost per mile for operating automotive equipment over roads having different types of surfaces will reveal sur-



prisingly large variations in the costs. These variations will apply to trucks used for hauling materials.

**Economic Life of Construction Equipment.** When new construction equipment is purchased, it is in good physical condition. As it is used, many parts are subjected to varying degrees of wear. Although careful maintenance will reduce the rapidity of wear, parts will fail and must be replaced, usually at an increasing rate as the life of the equipment is extended. The costs of maintenance and repairs may be grouped into two classifications, minor repairs, which are made as they become necessary, and major repairs, such as overhauling a complete unit. If records of the cost of operating equipment are kept, they will ultimately disclose that after the equipment has been used for a certain period of time the cost per hour for continuing to operate it will be greater than the average hourly cost up to that time. When it is established that the future cost per hour for owning and operating equipment will be higher than the average hourly cost for previous operation, the equipment has reached the end of its economic life and should be disposed of. It is then cheaper to replace the old equipment with new equipment. Contractors who do not keep accurate records of the cost of owning and operating equipment may use such equipment beyond its useful life, then wonder how competitors can do work at a lower price and still earn a profit. With the heavy investments frequently required for equipment the owner must keep cost records if he expects to conduct his business on a sound financial basis. No contractor can bid on a project intelligently if he does not know the cost of owning and operating his equipment.

There are at least two methods of determining the hourly cost of owning and operating equipment. One method is to keep accurate records of the initial cost of the equipment, the cost of maintenance and repairs, and the cost of fuel and lubrication if fuel is required. This information will establish the average hourly cost of owning and operating equipment. Another method is to add to the average hourly cost of owning and operating equipment the average hourly cost of lost time resulting from equipment failures. The latter item will be very significant for such equipment as a power shovel that is used to load earth into trucks. When the shovel breaks down, all trucks stop, but the truck costs may go on. In determining the economic life of a key unit of equipment it is proper to consider all costs that will be affected by a stoppage of the unit.

**Economic Life of Equipment, Neglecting the Cost of Lost Time.** Figure 3-2 illustrates a graphical method of determining the average hourly cost of owning and operating a 2-cu-yd dragline whose initial cost was \$52,000. The average costs per hour are determined by neglecting any possible salvage value of the equipment at the particular time. Salvage or sale value has the effect of reducing the average hourly cost of

the equipment. The costs through point *G* are determined from records, while costs beyond point *G* are estimated.

An examination of the information shown in Fig. 3-2 reveals that the lowest hourly cost, amounting to \$14.30, is obtained by disposing of the equipment after using it 11,200 hr. This is the end of its economic life. However, if there is some question about the need for a new unit of this type for future work, or if money is not available to purchase a new unit, the average hourly cost will not be increased seriously if the equipment is

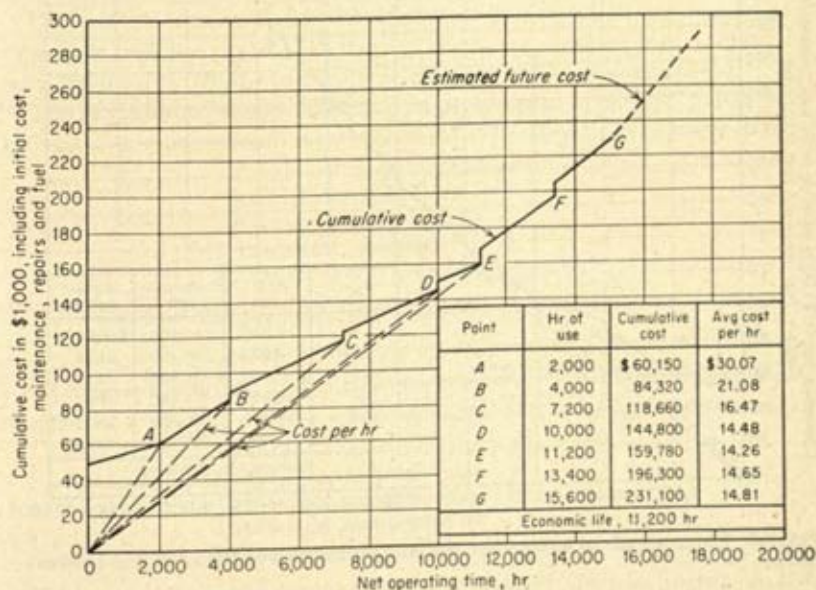


FIG. 3-2. Economic life of equipment, neglecting the cost of time lost due to breakdowns.

continued in use to points *F* or *G*, but thereafter it is estimated that the costs of maintenance and repairs will increase more rapidly.

**Economic Life of Equipment, Considering the Cost of Lost Time.** When a unit of construction equipment contributes to the operation of other units of equipment, such as a dragline loading trucks, the cost of a breakdown will be more serious than for equipment that operates independently of other equipment. If one tractor-scraper unit out of six breaks down, the production will be reduced by one-sixth during the breakdown but the job will go on. However, if a dragline which is loading six trucks breaks down, the entire job will be stopped. All the costs during the breakdown, including dragline rental, truck rental, labor, overhead, etc., should be charged against the failure of the dragline. Under



these conditions the economic life of a dragline may be reached sooner than when the dragline is operating independently.

Figure 3-3 illustrates the effect of charging to the dragline of Fig. 3-2 the costs of lost time resulting from breakdowns by the dragline. Curve 1 is a reproduction of the curve of Fig. 3-2. Curve 2 shows the cumulative effect of adding to curve 1 the total costs of breakdowns by the dragline.

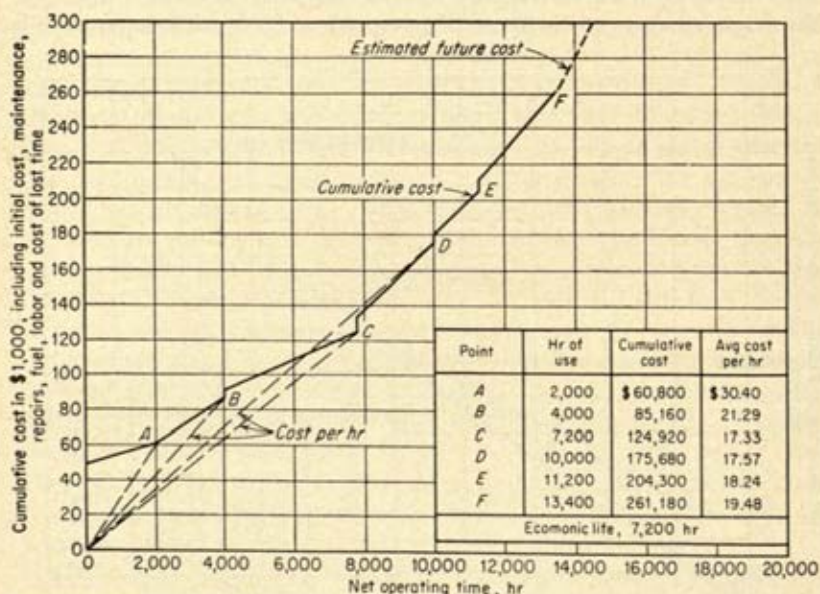


FIG. 3-3. Economic life of equipment, considering the cost of time lost due to breakdowns.

The costs used in plotting curve 2 are based on the following data:

Equipment	Cost per hr each, including operators	Total cost per hr
1 dragline, 2 cu yd. ....	\$18.10	\$18.10
6 trucks, 10 cu yd. ....	6.95	41.70
Job overhead. ....	.....	5.00
Total cost per hr. ....	.....	\$64.80

The costs of job overhead and operating the trucks are not charged to the dragline except during periods of breakdown by the dragline. If the trucks are rented, it may be possible to eliminate most of the truck costs for breakdowns of considerable duration, say more than 1 day. However, for breakdowns of short duration, less than 1 day, it is probable that the truck costs will continue while the dragline is being repaired.

**Equipment Used on Project.** The amount of equipment used on construction projects will vary considerably with the type of project. With transit-mixed concrete available in most areas of the nation the operating cost of equipment for constructing a building may amount to less than 5 per cent of the cost of the project, whereas for heavy construction, such as an earth-filled dam, the operating cost of equipment may amount to as much as 80 per cent of the cost of the project. The extent to which equipment is required on engineering construction is illustrated by the four lists given below.

For constructing a \$1,000,000 storm-sewer project in Denver, Colo., requiring trenches up to 17 ft deep in earth, for concrete pipe varying in sizes from 12 to 78 in., two contractors used the equipment listed.

The first contractor operated in congested areas, with utility lines and ground water to contend with. He used the following equipment:

- 3 clamshell, backhoes, 1 cu yd
- 6 centrifugal pumps, 4 in.
- 2 air compressors, 365 cfm
- 4 pneumatic tampers
- 2 D4 tractors and dozers, for backfill
- 1 motor grader, 12 ft
- 1 hydrocrane
- 1 hydra-hammer

The second contractor operated in open country, with no utility pipes or ground water to contend with. He used the following equipment:

- 1 Parsons 310 trench liner, 5 ft 0 in. wide
- 1 truck crane to handle pipe
- 1 D4 tractor and dozer
- 1 D2 tractor and dozer
- 750 trench jacks

For constructing a 24-in. gas pipe line in western Canada the contractor used the following major equipment for one spread, excluding grading equipment:

- |                                     |                               |
|-------------------------------------|-------------------------------|
| 9 D8 tractor angledozers            | 1 clean-and-prime machine     |
| 9 D8 tractors with side-boom cranes | 1 coat-and-wrap machine       |
| 3 D7 tractors with side-boom cranes | 5 dope pots                   |
| 3 D7 bulldozers                     | 1 portable generating plant   |
| 1 D6 backfiller                     | 6 portable air compressors    |
| 2 D4 tractors                       | 12 water pumps, various sizes |
| 1 HD7 tractor                       | 7 wagon drills                |
| 1 ripper                            | 2 paving breakers             |
| 1 Tournapull scraper                | 10 light plants               |
| 1 motor grader, 12 ft               | 4 power wagons                |
| 1 Buckeye 48 ditcher                | 2 semitrailers                |
| 1 Cleveland 320 ditcher             | 18 trucks                     |
| 18 welders, 300 amp                 | 15 pickup trucks              |
| 11 backhoes and cranes              | 4 jeeps                       |
| 1 cable-pipe bender                 | 1 ambulance                   |



For constructing the Chief Joseph Dam the contractor used the following major equipment:

#### Excavating

- 1 shovel, 6 cu yd
- 1 shovel, 5 cu yd
- 3 shovel draglines,  $3\frac{1}{2}$  cu yd
- 1 dragline, 3 cu yd
- 6 shovel draglines,  $2\frac{1}{2}$  cu yd
- 1 shovel dragline,  $1\frac{1}{2}$  cu yd
- 1 truck crane, 15-ton

#### Hauling

- 37 rear-dump trucks, 15- and 22-ton
- 8 rear-dump trucks, 21 cu yd
- 12 D8 tractors
- 2 water trucks
- 1 tractor low-bed haul unit
- 32 pickups and flat-bed trucks

#### Aggregate and concrete plants

- 1 primary jaw crusher,  $30 \times 42$  in.
- 1 secondary jaw crusher,  $24 \times 36$  in.
- 9 vibrating screens
- 1 rod mill for sand,  $6 \times 12$  ft
- 1 sand-sizing tank
- 1 rotary vacuum sand-drying wheel
- 1 feed-o-weight weighing blender
- 1 aggregate batching plant
- Assortment of belt conveyors, various lengths and sizes
- 3 concrete mixers, 4 cu yd
- 3 consistency meters
- Assortment of controllable concrete buckets, up to 8 cu yd
- 18 pneumatic vibrators, various sizes

#### Cooling plant

- 1 ammonia compressor,  $10 \times 10$  in.
- 1 ammonia compressor,  $9 \times 9$  in.
- 1 ammonia compressor,  $7 \times 7$  in.
- 2 Vilber VMC8 ammonia compressors
- 11 radial Freon compressors, 50 hp
- 4 Vilber Pak-ice machines, 30-ton

#### Cableways

- 2 25-ton cableways, with 500-hp hoists, carriages, and fittings
- 1 tail tower, 240 ft high
- 2 head towers, 110 ft high

#### Cofferdams

- 9 vertical turbine pumps, 20 in., 200 hp
- 2 McKiernan-Terry 9B3 pile hammers
- 2 McKiernan-Terry No. 7 hammers

#### Compressor plant

- 1 stationary compressor,  $23 \times 13 \times 16$  in.
- 1 stationary compressor,  $19 \times 11 \times 10$  in.
- 1 stationary compressor,  $15\frac{1}{2} \times 9\frac{1}{2} \times 7$  in.

- 2 stationary compressors,  $15\frac{1}{2} \times 9\frac{1}{2} \times 7$  in.
- 2 stationary compressors,  $13 \times 8 \times 7$  in.
- 2 stationary compressors,  $8 \times 5 \times 6\frac{1}{2}$  in.
- 1 stationary compressor, 365 cfm
- 2 portable compressors, 365 cfm
- 1 portable compressor, 105 cfm

## Drilling

- 2 Ingersoll-Rand Quarrymasters
- 25 Ingersoll-Rand wagon drills, model FM3
- 24 Ingersoll-Rand J50 jackhammers

**Sources of Equipment.** The previous article lists considerable equipment that was used on several projects. The list for each project indicates a substantial investment in equipment. A contractor may secure the use of equipment through ownership or through rental. If equipment will be used a great deal over an extended period of time, usually it will be cheaper and more satisfactory to purchase it. However, for equipment whose use will be limited, it frequently will be cheaper to rent the equipment than to purchase it.

## PROBLEMS

**3-1.** A  $2\frac{1}{2}$ -cu-yd dragline, which will cost \$71,680, can excavate earth for \$0.115 per cubic yard, including all costs. A 5-cu-yd dragline, which will cost \$166,490, can excavate earth for \$0.098 per cubic yard. Determine the minimum number of cubic yards of earth in a job to justify the purchase of the larger shovel if neither unit will have any salvage value after the completion of the job.

**3-2.** A 1-cu-yd power shovel cost \$22,650. A record of the cost of operating the shovel is as follows:

Fuel, lubricating oil, minor repairs, wire rope, \$1.45 per hr

No. hr after purchase	Cost of major repairs
1,820	\$1,240
3,260	760
4,130	1,430
4,980	965
5,820	1,320
6,740	865
8,150	1,975
9,360	1,260
10,610	1,395
12,240	3,250
13,560	3,680
15,225	3,760
16,195	3,525

Plot a graph showing the number of hours of use as the abscissa and the total cumulative cost as the ordinate. Has this shovel been used beyond its economic life? Determine the economic life of the shovel in hours. Determine the minimum cost per hour obtainable with this shovel. The calculated cost per hour neglects any salvage value of the shovel.



**3-3.** Analyze each of the plans given below to determine whether the 6-cu-yd trucks or the 13-cu-yd bottom-dump wagons should be used to haul the earth for a project requiring 4,640,000 cu yd bank measure. It will be necessary to haul earth at a rate of 500 cu yd per hr, working three shifts per day, 6 days per week.

Plan 1. Use diesel-engine-operated dump trucks with a capacity of 6 cu yd each.

No. trucks required, 14  
 Cost per truck, \$12,700  
 Est. salvage value per truck at end of job, \$4,650  
 Combined loading and hauling cost, \$0.397 per cu yd

Plan 2. Use diesel-engine-operated bottom-dump wagons with a capacity of 13 cu yd each.

No. units required, 8  
 Cost per unit, \$28,500  
 Est. salvage value per unit at end of job, \$9,250  
 Combined loading and hauling cost, \$0.345 per cu yd

**3-4.** Determine the probable cost per hour for owning and operating a  $1\frac{1}{2}$ -cu-yd gasoline-engine-operated crawler-type power shovel. The following information will apply:

Engine, 120 hp  
 Crankcase capacity, 4 gal  
 Time between oil changes, 100 hr  
 Operating factor, 60%  
 Useful life, 5 yr  
 Hours used per yr, 2,000  
 Delivered cost, \$41,620  
 Maintenance and repairs cost 100% of depreciation  
 Cost of fuel, \$0.20 per gal  
 Cost of lubricating oil, \$1.00 per gal

**3-5.** Determine the probable cost per hour for owning and operating a four-wheel diesel-engine-operated tractor. The following information will apply:

Engine, 200 hp  
 Crankcase capacity, 6 gal  
 Time between oil changes, 80 hr  
 Useful life, 5 yr  
 Hours used per yr, 2,000  
 Maintenance and repairs cost 80% of depreciation  
 Life of tires, 5,000 hr  
 Repairs to tires cost 15% of depreciation for tires  
 Delivered cost, \$17,850  
 Cost of tires, \$3,780  
 Cost of fuel, \$0.15 per gal  
 Cost of lubricating oil, \$1.00 per gal

**3-6.** A contractor owns construction equipment which will be tied up for 2 or 3 months on a project now in operation. After that the equipment will be free. This contractor needs similar equipment for use on another project to be started immediately. Should he purchase additional equipment, or should he rent the equipment that will be needed on the project to be started? Give your reasons. Discuss this subject at length if you wish.

## CHAPTER 4

### ENGINEERING FUNDAMENTALS

**General Information.** In this chapter many problems related to excavating, hauling, and placing earth will be discussed. With the constantly growing volume of earthwork for dams, levees, highways, airports, and other projects the need for selecting the most suitable construction equipment is becoming increasingly important. Persons in the construction industry, including contractors and engineers, should understand the effects which the selection of equipment and methods have on the cost of handling earth. It is hoped that the analyses of problems related to earthwork will assist in demonstrating how effective the application of engineering can be in determining the cost of earthwork.

**Physical Properties of Earth.** Prior to discussing earth handling or analyzing problems involving earthwork it is desirable to become more familiar with some of the physical properties of earth. These properties have a direct effect on the ease or difficulty of handling earth, the selection of equipment, and the production rates of the equipment.

**Swell and Shrinkage.** It is well known that the volume and density of earth undergo considerable changes when the earth is excavated, hauled, placed, and compacted. Because of these changes it is necessary to specify whether the volume is measured in its original position, in the loose condition, or in the fill after compaction.

The bank-measure volume is the volume of the earth measured in the borrow pit, trench, canal, or cut prior to loosening. This is the volume on which payment usually is based.

The loose-measure volume is the volume of the earth after it has been removed from its natural position and deposited in trucks, scrapers, or spoil piles.

The compacted volume, or fill volume, is the volume of the earth after it has been placed in a fill, such as a dam, and compacted. For projects requiring compacted earth fill the volume in the fill may be used as the basis of payment.

The volume of earth should be expressed in cubic yards, regardless of whether it is bank measure, loose, or compacted.

When the volume of earth increases because of loosening, this increase is defined as swell. It is expressed as a per cent of the original undisturbed volume. Thus, if the earth removed from a hole having a volume of 1 cu yd is found to have a loose volume of 1.25 cu yd, the gain in volume



is 0.25 cu yd, or 25 per cent. This particular earth is then said to have a swell of 25 per cent. The values of swell vary considerably for different classes of earth, as indicated in Table 4-1.

When earth is placed in a fill and compacted under modern construction methods, it usually will have a smaller volume than in its original condition. This reduction in volume is the result of an increase in the density and is illustrated by the difficulty frequently encountered in driving wood stakes into a fill after the earth has been thoroughly compacted by sheep's-foot tamping rollers, pneumatic tires, or other compacting equipment. This reduction in volume from the bank measure volume is defined as shrinkage. It is expressed as a per cent of the original undisturbed volume. Thus, if the earth removed from a hole having a volume of 1 cu yd is found to have a compacted volume of 0.9 cu yd, the loss in volume is 0.1 cu yd, or 10 per cent. For this condition the earth is said to have a shrinkage of 10 per cent. For any given class of earth the per cent of shrinkage will vary with the extent and degree of compaction and the amount of moisture present during compaction.

Table 4-1 gives representative values for swell for different classes of earth. These values will vary with the extent of loosening and compaction. If more accurate values are desired for a specific project, tests should be made on several samples of the earth, taken from different depths or from different locations within the proposed cut. The test may be made by weighing a given volume of undisturbed, loose, and compacted earth. A container having the same volume should be used in determining the weight for each of the three conditions.

The per cent swell and shrinkage may be determined from equations (4-1) and (4-2), respectively.

$$S_w = \left( \frac{B}{L} - 1 \right) \times 100 \quad (4-1)$$

$$S_h = \left( 1 - \frac{B}{C} \right) \times 100 \quad (4-2)$$

where  $S_w$  = % swell

$S_h$  = % shrinkage

$B$  = weight of undisturbed earth

$L$  = weight of loose earth

$C$  = weight of compacted earth

The weights of earth usually are expressed in pounds per cubic foot.

**EXAMPLE.** Determine the per cent swell and shrinkage for earth whose weights are as follows:

Undisturbed, 92 lb per cu ft

Loose, 76 lb per cu ft

Compacted, 108 lb per cu ft

The per cent swell will be

$$\begin{aligned} S_w &= (92\frac{1}{2}\% - 1) \times 100 \\ &= (1.21 - 1) \times 100 = 21\% \end{aligned}$$

The per cent shrinkage will be

$$\begin{aligned} S_s &= (1 - 92\frac{1}{2}\%) \times 100 \\ &= (1 - 0.85) \times 100 = 15\% \end{aligned}$$

Similar results may be obtained by using a calibrated container to measure the volumes of a given quantity of earth in the undisturbed, loose, and compacted states.

TABLE 4-1. REPRESENTATIVE SWELL FOR DIFFERENT CLASSES OF EARTH

Class of earth	Per cent swell
Clean sand or gravel	5-15
Top soil	10-25
Loamy soil	10-35
Common earth	20-45
Clay	30-60
Solid rock	50-80

To illustrate the use of Table 4-1, 10 cu yd of earth in a borrow pit may occupy 12.5 cu yd in a truck and 9 cu yd in a compacted fill.

**Soil Compaction.** If loose soil is deposited in a fill without compaction, it will contain many voids of varying sizes, which render it unfit for most engineering projects. Therefore, it is necessary to reduce the per cent of voids by compacting the soil with sheep's-foot rollers, pneumatic-tired units, smooth-wheeled rollers, or pneumatic tampers, used separately or in combination. The increased density will enable the soil to support heavier loads without excess settlement. When the soil is used for a dam, the increased density will make the structure more stable and will reduce the danger of seepage through the dam. While much information is available on soil compaction, the subject is not an exact science and it is difficult to specify in advance just what equipment and methods will produce the desired results. If exact results are required for a given project, the equipment and methods should be determined by actual field tests conducted under the supervision of an experienced soils engineer.

The density of a soil is increased by expelling air and moisture. This expulsion is accomplished by forcing the particles closer together as the result of applied pressure and thus to rearrange their positions in the mass of soil. The application of vibration combined with pressure is effective in increasing the density of many soils.

Tests reveal that if a soil is deficient in moisture the internal friction of the particles is high and it will be difficult to obtain good compaction.



If moisture is added, it will serve as a lubricant, which will reduce the internal friction and facilitate the compaction of the soil. If too much moisture is present, it is necessary to expel the excess moisture in compacting the soil. The amount of moisture necessary for maximum compaction, which varies with the type of soil, is expressed as a per cent, by weight, of the soil and is referred to as the optimum moisture content. For most soils encountered in construction the optimum moisture content varies from 8 to 25 per cent of the weight of the soil. The actual amount for any given soil can be determined by tests, as described below.

Soil should be deposited in sufficiently thin layers to permit a uniform distribution of the moisture within each layer and also to permit the pressure produced by the compacting equipment to penetrate to the full depth of the layer. A soil such as sand may be deposited in relatively thick layers, whereas clay should be deposited in thin layers.

Tests have been developed as an aid in determining the most satisfactory methods of compacting soils. While there are differing opinions regarding the test procedures, those established by the American Association of State Highway Officials (AASHO) will usually give satisfactory results. These tests are described in the next three paragraphs.

The *moisture-content test* is used to determine the ratio of the weight of the water contained in a given sample to the dry weight of the sample. The answer is expressed in per cent. The test is conducted by weighing a moist sample of earth, drying it in an oven, then noting the loss in weight due to the water evaporated. The weight of water lost divided by the weight of the dry sample and multiplied by 100 equals the per cent of moisture content.

The *unit-weight determination* is a test for determining the weight of a unit volume, which is expressed in pounds per cubic foot.

The *compaction test for optimum moisture content* (modified AASHO method) is an important test used to determine the quantity of moisture required to permit the greatest compaction. If too much water is present, more work must be done to expel the excess water. If insufficient water is present, the soil will not compact easily. This test is made by compacting, in a standard test machine, a quantity of the sample soil which has been thoroughly mixed with water. After compaction the weight per unit volume of the compacted material is determined. Next, samples of the compacted soil are taken, and the moisture content is determined as in the moisture-content test discussed above. From this information the moisture content for a unit weight of soil is determined. This same procedure is repeated on several samples with varying amounts of water added until the addition of more water does not give any weight increase for a given volume. The moisture content which results in the greatest weight per volume is the optimum moisture content.

The compaction test for optimum moisture content is similar in purpose to the *standard Proctor test*. The two tests differ in details of procedure as to the number of soil layers and thickness of soil, weight of the tamper used for compacting, and the distance through which the tamper is moved.

It may be necessary to obtain earth for a fill from a borrow pit which has stratified layers of sand, clay, gravel, or other materials. For many projects neither material alone is satisfactory for a fill. Sand and gravel have excellent load-bearing properties, but they will not bond together into a stable earth structure. Clay and loam may be stable when dry, but when they are wet, they lose their stability and become spongy. A combination of sand and gravel with the correct proportion of fine material, such as clay, will produce a structure with good load-bearing properties plus the stability resulting from the bonding together of the larger particles by the clay. A blended soil of this type can be compacted satisfactorily, and it retains its stability when wet or dry. In producing earth from a stratified borrow pit it may be desirable to select equipment that is capable of mixing the earth from the several layers as it is excavated.

**Rolling Resistance.** Rolling resistance is a resistance which is encountered by a vehicle in moving over a road or surface. This resistance varies considerably with the type and condition of the surface over which a vehicle moves. Soft earth offers a higher resistance than hard-surfaced roads such as concrete pavement. For vehicles which move on rubber tires the rolling resistance varies with the size, pressure, and tread design of the tires. For equipment which moves on crawler tracks, such as tractors, the resistance varies primarily with the type and condition of the road surface. If a truck is driven off a hard-surfaced highway into a field of soft earth, the resistance to moving is increased materially, as all drivers know. If a loaded wheelbarrow has a well-inflated pneumatic tire, it is much more easily pushed along a concrete sidewalk than when the tire is semideflated or soft. The difference is due to changes in the rolling resistance. A narrow-tread high-pressure tire gives lower rolling resistance than a broad-tread low-pressure tire on a hard-surfaced road. This is the result of the smaller area of contact between the tire and the road surface. However, if the road surface is soft and the tire tends to sink into the earth, a broad-tread low-pressure tire will offer a lower rolling resistance than a narrow-tread high-pressure tire. The reason for this condition is that the narrow tire sinks into the earth more deeply than the broad tire and thus is always having to climb out of a deeper hole, which is equivalent to climbing a steeper grade. As explained later in this book, the type and size tires selected for earth-hauling equipment should be determined after the condition of the haul road is known.

The rolling resistance of an earth haul road probably will not remain



constant under varying climatic conditions or for varying types of soil which exist along the road. If the earth is stable, highly compacted, and well maintained with a grader, and if the moisture content is kept near the optimum, it is possible to provide a surface with a rolling resistance about as low as for concrete and asphalt. It is possible to add moisture, but following an extended period of rain it may be difficult to remove the excess moisture, and the haul road will become muddy, with an increase in rolling resistance. Providing good surface drainage will speed the removal of the water and should permit the road to be reconditioned more quickly. For a major earth project it is good economy to provide a patrol grader, sprinkler trucks, and probably sheep's-foot rollers to keep the haul road in good condition. As illustrated under the subject of trucks, the maintenance of low rolling resistance is one of the best financial investments an earth-moving contractor can make.

Although it is impossible to give completely accurate values for the rolling resistances for all types of haul roads and wheels, the values given in Table 4-2 are reasonably accurate and may be used for estimating pur-

TABLE 4-2. REPRESENTATIVE ROLLING RESISTANCES FOR VARIOUS TYPES OF WHEELS AND SURFACES, IN POUNDS PER TON OF GROSS LOAD

Type of surface	Steel tires, plain bearings	Crawler-type track and wheels	Rubber tires, anti-friction bearings	
			High pressure	Low pressure
Smooth concrete.....	40	55	35	45
Good asphalt.....	50-70	60-70	40-65	50-60
Earth, compacted and well maintained...	60-100	60-80	40-70	50-70
Earth, poorly maintained, rutted.....	100-150	80-110	100-140	70-100
Earth, rutted, muddy, no maintenance...	200-250	140-180	180-220	150-200
Loose sand and gravel.....	280-320	160-200	260-290	220-260
Earth, very muddy, rutted, soft.....	350-400	200-240	300-400	280-340

poses. Rolling resistance is expressed in pounds of tractive pull required to move each gross ton over a level surface of the specified type or condition. For example, if a loaded truck having a gross weight equal to 20 tons is moving over a level road whose rolling resistance is 100 lb per ton, the tractive effort required to keep the truck moving at a uniform speed will be

$$20 \text{ tons} \times 100 \text{ lb per ton} = 2,000 \text{ lb}$$

If it is desirable to obtain the rolling resistance of a haul road, this can be done by towing a truck or other vehicle whose gross weight is known along a level section of the haul road at a uniform speed. The tow cable

should be equipped with a dynamometer or some other device which will permit the average tension in the cable to be determined. This tension is the total rolling resistance of the gross weight of the truck. The rolling resistance in pounds per gross ton will be

$$R = \frac{P}{W} \quad (4-3)$$

where  $R$  = rolling resistance, lb per ton

$P$  = total tension in tow cable, lb

$W$  = gross weight of truck, tons

If it is necessary to tow the loaded truck up or down a sloping haul road, an appropriate correction for the effect of the slope may be applied to the tension in the tow cable, as explained in the following article. In

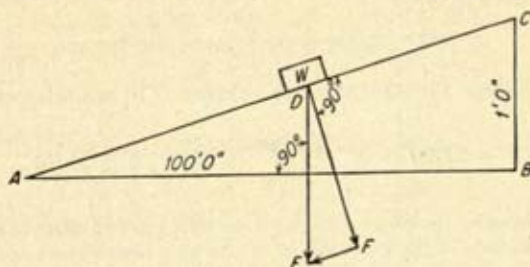


FIG. 4-1. The effect of grade on the performance of a tractor or a truck.

order to apply a correction it is necessary to know the grade of the haul road over which the test is being conducted.

**The Effect of Grade on Required Tractive Effort.** When a vehicle moves up a sloping road, the total tractive effort required to keep the vehicle moving is increased approximately in proportion to the slope of the road. If a vehicle moves down a sloping road, the total tractive effort required to keep the vehicle moving is reduced in proportion to the slope of the road. The most common method of expressing a slope is by per cent. A 1 per cent slope is one where the surface rises or drops 1 ft vertically in a horizontal distance of 100 ft. If the slope is 5 per cent, the surface rises or drops 5 ft per 100 ft of horizontal distance. If the surface rises, the slope is defined as plus, while if it drops, the slope is defined as minus. All automobile drivers know that a plus slope retards, while a minus slope aids, an automobile traveling along a highway. The same forces apply to construction equipment moving over a road. This is a physical property which is not affected by the type of equipment or the condition or type of the road.

The effect of grade is to increase, for a plus slope, or decrease, for a



minus slope, the required tractive effort by 20 lb per gross ton of weight for each 1 per cent of grade. While this amount is not strictly correct for all slopes, it is sufficiently accurate for most construction projects.

Figure 4-1 illustrates the method of determining the effect of grade on tractive effort. The line  $AB$  is horizontal. The slope of  $AC$  is +1 per cent.  $DE$  is perpendicular to  $AB$ .  $DF$  is perpendicular to  $AC$ .  $EF$  is parallel to  $AC$ . Triangle  $DEF$  is similar to triangle  $ABC$ . For practical purposes the length of  $AC$  is 100 ft.  $W$  is a 1-ton weight, represented by the vector  $DE$ .  $P$  is the component of  $W$  parallel to  $AC$ .

From the similarity of triangles,

$$\frac{P}{W} = \frac{BC}{AC} \quad \text{or} \quad P = W \frac{BC}{AC} = 2,000 \text{ lb} \times \frac{1}{100} = 20 \text{ lb}$$

If  $BC$  is increased to 2 ft,

$$P = 2,000 \text{ lb} \times \frac{2}{100} = 40 \text{ lb}$$

For any given slope the approximate value of  $P$  in pounds per ton is

$$P = 2,000 \text{ lb} \times \frac{\% \text{ slope}}{100} = 20 \text{ lb} \times \% \text{ slope}$$

**EXAMPLE.** Consider the effect of grade on the total tractive effort of a truck whose gross weight is 20 tons. The truck will be driven up a road whose slope is 5 per cent. The additional tractive effort resulting from the slope is

$$P = 20 \text{ tons} \times 20 \text{ lb per ton} \times 5\% = 2,000 \text{ lb}$$

Thus, the truck engine must continually deliver to the driving wheels 2,000 lb of rimpull to overcome the effect of the slope. If the truck is moving down the same slope, the effect of the grade will be to help the engine and truck, which is equivalent to adding 2,000 lb to the rimpull of the truck.

If a tractor is towing a load, the combined gross weights of the tractor and its towed load should be used in determining the effect of the grade.

Table 4-3 gives values for the effect of slope, expressed in pounds per gross ton of weight of the vehicle.

**The Effect of Grade in Locating a Borrow Pit.** Sometimes engineers and contractors do not give sufficient consideration to the grade or slope of the haul road in locating borrow pits. It is desirable, when possible, to locate a borrow pit at a higher elevation than the fill, in order that the slope down the haul road may help the loaded trucks or other hauling equipment by permitting them to carry larger loads or to travel at higher speeds. As the vehicles will be empty when returning up the haul road from the fill to the borrow pit, the effect of the grade will be considerably

less. This item is discussed in detail in Chap. 7, Trucks and Wagons (see pages 173 to 177).

**Coefficient of Traction.** The total energy of an engine in any unit of equipment designed primarily for pulling a load can be converted into tractive effort only if sufficient traction can be developed between the driving wheels or tracks and the haul surface. If there is not sufficient traction, the full power of the engine cannot be used. The wheels or tracks will slip on the surface. Thus, the coefficient of traction between rubber tires or crawler tracks and different haul surfaces is of importance to the operators of hauling units.

TABLE 4-3. THE EFFECT OF GRADE ON THE TRACTIVE EFFORT OF VEHICLES, IN POUNDS PER GROSS TON

Slope, %	Lb per ton of gross weight	Slope, %	Lb per ton of gross weight
1	20.0	12	238.4
2	40.0	13	257.8
3	60.0	14	277.4
4	80.0	15	296.6
5	100.0	20	392.3
6	119.8	25	485.2
7	139.8	30	574.7
8	149.2	35	660.6
9	179.2	40	742.8
10	199.0	45	820.8
11	218.0	50	894.4

The coefficient of traction may be defined as the factor by which the total load on a driving tire or track should be multiplied in order to determine the maximum possible tractive force between the tire or track and the surface just before slipping will occur. For example, the driving tires of a truck rest on a level haul road of dry clay. The total pressure between the tires and the road surface is 8,000 lb. In testing the tires for slippage by applying a driving force to the wheels, it is found that slippage will occur when the tractive force between the tires and the surface is 4,800 lb. The coefficient of traction is  $4,800 \div 8,000 = 0.60$ .

The coefficient of traction between rubber tires and road surfaces will vary with the type of tread on the tires and with the road surface. For crawler tracks it will vary with the design of the grouser and the road surface. These variations are such that exact values cannot be given. Table 4-4 gives approximate values for the coefficient of traction between rubber tires or crawler tracks and road surfaces which are sufficiently accurate for most estimating purposes.



TABLE 4-4. COEFFICIENTS OF TRACTION FOR VARIOUS ROAD SURFACES

Surface	Rubber tires	Crawler tracks
Dry, rough concrete.....	0.80-1.00	0.45
Dry clay loam.....	0.50-0.70	0.90
Wet clay loam.....	0.40-0.50	0.70
Wet sand and gravel.....	0.30-0.40	0.35
Loose, dry sand.....	0.20-0.30	0.30
Dry snow.....	0.20	0.15-0.35
Ice.....	0.10	0.10-0.25

**EXAMPLE.** Assume that a rubber-tired tractor has a total weight of 18,000 lb on the two driving tires. The maximum rimpull in low gear is 9,000 lb. If the tractor is operating in wet sand, with a coefficient of traction of 0.30, the maximum possible rimpull prior to slippage of the tires will be  $0.30 \times 18,000 \text{ lb} = 5,400 \text{ lb}$ . Regardless of the power of the engine, not more than 5,400 lb of tractive effort may be used because of the slippage of the wheels. If the same tractor is operating on dry clay, with a coefficient of traction of 0.60, the maximum possible rimpull prior to slippage of the tires will be  $0.60 \times 18,000 \text{ lb} = 10,800 \text{ lb}$ . For this surface the engine will not be able to cause the tires to slip. Thus, the full power of the engine may be used.

**The Effect of Altitude on the Performance of Internal-combustion Engines.** An internal-combustion engine operates by combining oxygen from the air with the fuel and then burning the mixture to convert latent energy into mechanical energy. The power of an engine is a measure of the rate at which it can produce energy from fuel. For each charge of fuel and air into a cylinder there must be a correct ratio between the quantity of fuel and air if the maximum efficiency and power are to be obtained from the engine. The ratio between the quantities should be that which will provide just enough oxygen to supply the requirements of the fuel for complete combustion. If the density of the air is reduced because of altitude, the quantity of oxygen in a given volume of air will be less than for the same volume of air at sea level. As each cylinder of an engine draws in a given volume of air prior to the firing stroke, there will be less oxygen in the cylinder if the density of the air is reduced. Since the ratio of the oxygen and fuel should remain constant, it will be necessary to reduce the quantity of fuel supplied to an engine at high altitudes. This is usually done by adjusting the carburetor. The effect on the engine is to reduce the power. A human being experiences the same effect when he engages in physical work at a high altitude. Although he breathes the same volume of air, he may not get enough oxygen to supply his requirements.

If the density of the air decreased uniformly with the altitude, it would be possible to express the loss in power of an engine, due to altitude by,

means of a simple formula with a high degree of accuracy. Actually this is not true.

For most practical purposes it is sufficiently accurate to assume that for four-cycle gasoline and diesel engines the loss in power due to altitude will be equal to approximately 3 per cent of the sea-level horsepower for each 1,000 ft above the first 1,000 ft. Thus, for a four-cycle engine with 100 belt hp at sea level, the power at 10,000 ft above sea level would be determined as follows:

$$\begin{array}{rcl} \text{Sea-level power} & & = 100 \text{ hp} \\ \text{Loss due to altitude, } \frac{0.03 \times 100 \times (10,000 - 1,000)}{1,000} & = & 27 \text{ hp} \\ \text{Effective power} & & = 73 \text{ hp} \end{array}$$

For the two-cycle engine, which is becoming increasingly more popular in the diesel field, the loss in power due to altitude is approximately 1 per cent of the sea-level horsepower for each 1,000 ft above the first 1,000 ft. This type engine has its air supplied, under a slight pressure, by a blower, whereas the four-cycle engine depends on the suction of the pistons for the supply of air. If the engine described in the previous paragraph is a two-cycle, the power at 10,000 ft above sea level would be determined as follows:

$$\begin{array}{rcl} \text{Sea-level power} & & = 100 \text{ hp} \\ \text{Loss due to altitude, } \frac{0.01 \times 100 \times (10,000 - 1,000)}{1,000} & = & 9 \text{ hp} \\ \text{Effective power} & & = 91 \text{ hp} \end{array}$$

The two previous problems indicate that, other factors being equal, a two-cycle engine will give better performance than a four-cycle engine at high altitudes.

The effect of the loss in power due to altitude may be eliminated by the installation of a supercharger. This is a mechanical unit which will increase the pressure of the air supplied to the engine, thus permitting sea-level performance at any altitude. If equipment is to be used at high altitudes for long periods of time, the increased performance probably will more than pay for the installed cost of a supercharger.

A contractor who has established production rates for his equipment at or near sea level will make a serious mistake if he uses those production rates in bidding a job to be constructed at a high altitude. He must install superchargers or apply a correction factor, as more fully explained hereafter under the subjects of trucks and tractors.

**The Effect of Temperature on the Performance of Internal-combustion Engines.** Many persons who have driven an automobile through a desert during a hot afternoon have noticed that the performance of the automobile seemed sluggish. If driving was continued into the night,



after the temperature had decreased appreciably, the performance of the engine seemed to improve noticeably. This experience was not imaginary. An internal-combustion engine will develop a higher horsepower at a low air temperature than at a high temperature. The effect of temperature on the performance of an internal-combustion engine has been determined from laboratory tests. The next article discusses the combined effect of pressure and temperature on the performance of internal-combustion engines.

**The Combined Effect of Pressure and Temperature on the Performance of Internal-combustion Engines.** When an internal-combustion engine is tested to determine its power, it is necessary to conduct the tests under standard conditions in order that the results may be significant. Standard conditions mean a temperature of 60 degrees Fahrenheit and average sea-level barometric pressure, equivalent to 29.92 inches of mercury (in. Hg). As the power of the engine usually is determined with a brake or a dynamometer, the result is expressed as the brake horsepower (bhp) of the engine.

If a test must be conducted under other than standard conditions, the corrected horsepower for standard conditions may be determined by using the formula

$$H_e = H_o \frac{P_o}{P_s} \sqrt{\frac{T_o}{T_s}} \quad (4-4)$$

where  $H_e$  = corrected bhp for standard conditions

$H_o$  = observed bhp, as determined from test

$P_s$  = standard barometric pressure, 29.92 in. Hg

$P_o$  = observed barometric pressure, in. Hg, at time of test

$T_o$  = absolute temperature, °F, equal to 460 + observed temperature

$T_s$  = absolute temperature for standard conditions, equal to 460 + 60 = 520°F

**EXAMPLE.** A gasoline engine was tested under the given conditions and was found to develop the indicated horsepower. It is desired to convert the results to bhp for standard conditions.

Observed hp = 86.43

Observed pressure = 29.52 in. Hg

Observed temperature = 42°F

Substituting these values in formula (4-4), we get

$$H_e = 86.43 \times \frac{29.92}{29.52} \sqrt{\frac{460 + 42}{520}} = 86.07 \text{ hp}$$

Thus, this engine should develop 86.07 bhp if tested under standard conditions.

Formula (4-4) may be used to determine the probable effective horsepower of a four-cycle engine at any temperature and altitude. From

Table 4-5 determine the probable barometric pressure for the given altitude. Estimate the probable temperature. Apply this information to formula (4-4), and solve for the effective horsepower.

TABLE 4-5. AVERAGE BAROMETRIC PRESSURES FOR VARIOUS ALTITUDES ABOVE SEA LEVEL, IN INCHES OF MERCURY

Altitude above sea level, ft	Barometric pressure, in. Hg
0	29.92
1,000	28.86
2,000	27.82
3,000	26.80
4,000	25.82
5,000	24.87
6,000	23.95
7,000	23.07
8,000	22.21
9,000	21.36
10,000	20.55

EXAMPLE. A tractor is operated by a four-cycle diesel engine. When tested under standard conditions, the engine developed 130 bhp. What is the probable horsepower at an altitude of 3,660 ft, where the average daily temperature is 72°F?

The information for use in formula (4-4) will be as follows:

$$H_s = 130$$

$$P_s = 29.92 \text{ in.}$$

$$P_a = 26.14 \text{ in. (from Table 4-5)}$$

$$T_s = 520^\circ\text{F}$$

$$T_a = 460 + 72 = 532^\circ\text{F}$$

Find  $H_a$ .

Rewriting formula (4-4) and substituting the given information, we get

$$\begin{aligned} H_a &= H_s \frac{P_s}{P_a} \sqrt{\frac{T_s}{T_a}} \\ &= 130 \times \frac{26.14}{29.92} \sqrt{\frac{520}{532}} = 112.7 \text{ hp} \end{aligned}$$

Thus, the probable bhp of the engine will be reduced to 112.7 as a result of the increased altitude and temperature.

Table 4-6 gives factors by which the horsepower of a four-cycle engine, as determined under standard conditions, may be multiplied to obtain the probable horsepower for various altitudes and temperatures. Owing to variations in the barometric pressure at any altitude as the result of changes in climatic conditions, the factors may vary slightly with climatic conditions.

The two-cycle diesel engine operates under different conditions from those which apply to a four-cycle engine. Therefore, the correction fac-



TABLE 4-6. CORRECTION FACTORS FOR DETERMINING THE EFFECTIVE HORSEPOWER OF FOUR-CYCLE ENGINES, FOR VARIOUS ALTITUDES AND TEMPERATURES

Altitude above sea level, ft	Temperature, °F								
	110	90	70	60	50	40	20	0	-20
0	0.954	0.971	0.991	1.000	1.008	1.018	1.039	1.062	1.085
1,000	0.920	0.937	0.955	0.964	0.974	0.984	1.003	1.025	1.048
2,000	0.887	0.904	0.921	0.930	0.938	0.948	0.968	0.988	1.010
3,000	0.855	0.872	0.888	0.896	0.905	0.914	0.933	0.952	0.974
4,000	0.825	0.840	0.856	0.865	0.873	0.882	0.899	0.918	0.938
5,000	0.795	0.809	0.825	0.833	0.842	0.849	0.867	0.885	0.904
6,000	0.767	0.781	0.795	0.803	0.811	0.820	0.836	0.853	0.872
7,000	0.738	0.752	0.767	0.775	0.782	0.790	0.806	0.823	0.840
8,000	0.712	0.725	0.739	0.746	0.754	0.762	0.776	0.793	0.811
9,000	0.686	0.699	0.713	0.720	0.727	0.734	0.748	0.764	0.782
10,000	0.682	0.675	0.687	0.639	0.707	0.707	0.722	0.737	0.753

tors given in Table 4-6 will not apply to two-cycle engines. If similar information is desired, it should be requested from the manufacturer.

**Drawbar Pull.** The available pull which a crawler tractor can exert on a load that is being towed is referred to as the drawbar pull of the tractor. The pull is expressed in pounds. From the total pulling effort of an engine there must be deducted the pull required to move the tractor over a level haul road before the drawbar pull can be determined. If a crawler tractor tows a load up a slope, its drawbar pull will be reduced by 20 lb for each ton of weight of the tractor for each 1 per cent slope.

The performance of crawler tractors, as reported in the specifications supplied by the manufacturer, is usually based on the Nebraska tests. In testing a tractor, to determine its maximum drawbar pull at each of the available speeds, the haul road is calculated to have a rolling resistance of 110 lb per ton. If a tractor is used on a haul road whose rolling resistance is higher or lower than 110 lb per ton, the drawbar pull will be reduced or increased, respectively, by an amount equal to the weight of the tractor in tons multiplied by the variation of the haul road from 110 lb per ton.

**EXAMPLE.** A tractor whose weight is 15 tons has a drawbar pull of 5,684 lb in sixth gear when operated on a level road having a rolling resistance of 110 lb per ton. If the tractor is operated on a level road having a rolling resistance of 180 lb per ton, the drawbar pull will be reduced by 15 tons  $\times$  (180 - 110) = 1,050 lb. Thus, the effective drawbar pull will be 5,684 - 1,050 = 4,634 lb.

The drawbar pull of a crawler tractor will vary indirectly with the

speed of each gear. It is highest in the first gear and lowest in the top gear. The specifications supplied by the manufacturer should give the maximum speed and drawbar pull for each of the several gears. The following is an example:

Gear	Speed, mph	Drawbar pull, lb
1st	1.72	28,019
2d	2.18	22,699
3d	2.76	17,265
4th	3.50	13,769
5th	4.36	10,074
6th	7.00	5,579

**Rimpull.** Rimpull is a term which is used to designate the tractive force between the rubber tires of driving wheels and the surface on which they travel. If the coefficient of traction is high enough to eliminate tire slippage, the maximum rimpull is a function of the power of the engine and the gear ratios between the engine and the driving wheels. If the driving wheels slip on the haul surface, the maximum effective rimpull will be equal to the total pressure between the tires and the surface multiplied by the coefficient of traction. Rimpull is expressed in pounds.

If the rimpull of a vehicle is not known, it may be determined from the formula

$$\text{Rimpull} = \frac{375 \times \text{hp} \times \text{efficiency}}{\text{speed, mph}} \quad (4-5)$$

The efficiency of most tractors and trucks will range from 80 to 85 per cent. For a rubber-tired tractor with a 140-hp engine and a maximum speed of 3.25 mph in first gear, the rimpull will be

$$\frac{375 \times 140 \times 0.85}{3.25} = 13,730 \text{ lb}$$

The maximum rimpull in all gear ranges for this tractor will be as follows:

Gear	Speed, mph	Rimpull, lb
1st	3.25	13,730
2d	7.10	6,285
3d	12.48	3,576
4th	21.54	2,072
5th	33.86	1,319

In computing the pull which a tractor can exert on a towed load, it is necessary to deduct from the rimpull of the tractor the tractive force



required to overcome the rolling resistance plus any grade resistance for the tractor. It will be noted that the rubber-tired tractor differs from the crawler tractor in this respect. For example, if a tractor whose maximum rimpull in the first gear is 13,730 lb weighs 12.4 tons and is operated up a haul road with a slope of 2 per cent and a rolling resistance of 100 lb per ton, the pull available for towing a load will be determined as follows:

Max rimpull		= 13,730 lb
Pull required to overcome grade, $12.4 \times 20 \times 2$	= 496 lb	
Pull required to overcome rolling resistance, $12.4 \times 100$	= 1,240 lb	
Total pull to be deducted		= 1,736 lb
Pull available for towing a load		= 11,994 lb

**Acceleration.** Acceleration is the increasing of the speed of a moving vehicle by the application of surplus power from the engine, that is, power which is not required to keep the vehicle moving at a uniform speed. The rate of acceleration depends on the weight of the vehicle and the surplus rimpull that is available for accelerating. Unless surplus rimpull is available, the speed of the vehicle cannot be increased.

Although it is impossible to analyze a hauling unit to determine the exact rates of acceleration for given conditions, it is possible to obtain results which are sufficiently accurate for most estimating purposes. Such an analysis is based on Newton's second law of motion. The basic law is expressed by the formula

$$F = \frac{W}{g} a \quad (4-6)$$

where  $F$  = an accelerating force, lb

$W$  = weight to be accelerated, lb

$g$  = acceleration of gravity, 32.2 ft per sec per sec

$a$  = acceleration of weight  $W$ , ft per sec per sec

Assume that a force of 10 lb is available to accelerate a weight of 1 ton, 2,000 lb. If this information is inserted in formula (4-6) and the formula is rewritten, we get

$$a = \frac{Fg}{W} = \frac{10 \times 32.2}{2,000} = 0.161 \text{ ft per sec per sec}$$

Expressed in words this means that for each second of elapsed time that the 10-lb force is applied the weight will undergo an increase in velocity of 0.161 fps. This is equivalent to an increase in speed of 0.11 mph per sec. In 1 min the speed will be increased by  $60 \times 0.11 = 6.6$  mph. If the force is increased from 10 to 20 lb, the rate of acceleration will be increased to 13.2 mph per min.

Table 4-7 gives the approximate rates of accelerating a 1-ton weight, in mph per minute, for various accelerating rimpulls. In the table the accelerating rimpull is expressed in pounds, and the weight remains constant at 1 ton.

TABLE 4-7. APPROXIMATE RATE OF ACCELERATING A 1-TON WEIGHT

Accelerating rimpull	Acceleration, mph per min
5	3.3
10	6.6
20	13.2
30	19.8
50	33.0
100	66.0
200	132.0
300	198.0

EXAMPLE. A practical application may be made to a loaded truck. Assume that the truck and its load weigh 40,850 lb and that the truck has a 125-hp engine. The maximum speed in first gear is 3.0 mph. For this speed the rimpull is obtained from formula (4-5), using an assumed efficiency of 81 per cent.

$$\text{Rimpull} = \frac{375 \times 125 \times 0.81}{3.0} = 12,620 \text{ lb}$$

The maximum speeds and rimpulls for various gears are as follows:

Gear	Maximum speed, mph	Rimpull, lb
1st	3.0	12,620
2d	5.2	7,275
3d	9.2	4,120
4th	16.8	2,250
5th	27.7	1,365

The haul road is level, with a rolling resistance of 60 lb per ton.

It is desired to determine the approximate total time required to bring the truck from a stationary position to the top speed in fifth gear. It is assumed that the rimpull in excess of that required to overcome rolling resistance will be available to accelerate the truck. Wind resistance will be neglected, which may be a factor in some cases.

The weight of the loaded truck is  $\frac{40,850}{2,000} = 20.425$  tons.

The rimpull required to overcome rolling resistance is  $20.425 \times 60 = 1,225$  lb. This rimpull must be deducted from the maximum rimpull of the truck in each gear in order to obtain the rimpull available for accelerating the truck.

The maximum rimpulls available for accelerating the truck are approximately those given below. In actual operation the available rimpulls will probably be less than the amounts given when a truck goes into any gear, because of the reduction in power of an engine at a reduced speed. If the truck is equipped with a torque converter



instead of just gears and a clutch, it is more probable that the indicated rimpulls will be available throughout the accelerating ranges.

Gear	Max accelerating rimpull, lb
1st	11,395
2d	6,050
3d	2,895
4th	1,025
5th	140

In actual operation of the truck the maximum accelerating rimpulls will probably not be available, especially in the lower gears, because of the reluctance of the driver to apply the full power of the engine, and also because of higher mechanical losses in the lower gears. Therefore, the first three values should be modified downward, and perhaps the other two should be reduced somewhat, according to the judgment and experience of the job planner.

The time required to bring the speed of the truck up to 3.0 mph in first gear would be determined as follows:

$$\text{Max accelerating rimpull per ton} = \frac{11,395}{20.425} = 557 \text{ lb}$$

If this rimpull is reduced to an effective value of 300 lb per ton, it will produce an acceleration of 198 mph in 1 min. The time required to produce an acceleration to 3.0 mph will be  $\frac{3.0}{198} = 0.015$  min. When the shift is made to second gear, the speed should be 3.0 mph. It is to be increased to 5.2 mph, for a gain of 2.2 mph. The time required to increase the speed from 3.0 to 5.2 mph would be determined as follows:

$$\text{Max accelerating rimpull per ton} = \frac{6,050}{20.425} = 296 \text{ lb}$$

If this rimpull is reduced to an effective value of 200 lb per ton, it will produce an acceleration of 132 mph in 1 min. The time required to produce an acceleration of

$$2.2 \text{ mph will be } \frac{2.2}{132} = 0.017 \text{ min.}$$

In a similar manner the time required to accelerate to top speed in each of the other three gears can be determined. The results are given in Table 4-8. The total time given in Table 4-8 must be increased by the total time required to shift gears. If time is started when the shift is made into first gear, there will be five shifts. It should be possible to make a shift in 4 sec without undue rushing.

The time given in Table 4-8 may be used as a guide, but it should be modified by the judgment of the job planner. For example, if the rolling resistance of the haul road is reduced to 50 lb per ton, the maximum accelerating rimpull in fifth gear will be increased to 16.8 lb per ton. If the effective rimpull is assumed to be 14 lb per ton, the acceleration will

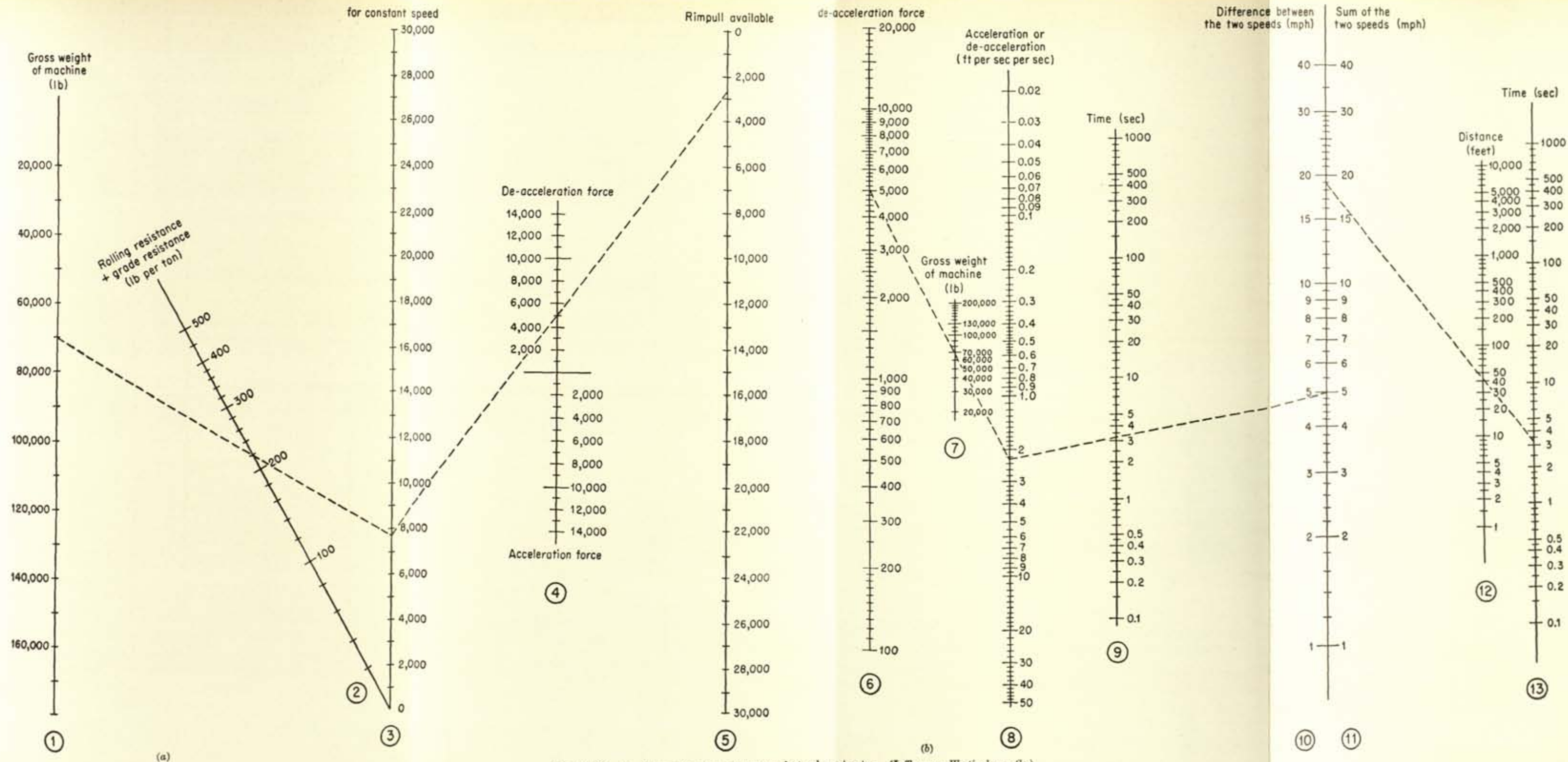






TABLE 4-8. APPROXIMATE TIME REQUIRED TO ACCELERATE A TRUCK

Gear	Top speed, mph	Required acceleration, mph	Accelerating rimpull, lb per ton		Acceleration, mph per min	Time to accelerate to top speed, min
			Max	Effective		
1st	3.0	3.0	557	300	198	0.015
2d	5.2	2.2	296	200	132	0.017
3d	9.2	4.0	141	100	66	0.061
4th	16.8	7.6	50	40	26.4	0.288
5th	27.7	10.9	7	6	4.0	2.725
Total time, no allowance for shifting gears.....						3.106
Add for 5 gear shifts @ 4 sec each.....						0.333
Total elapsed time.....						3.439

be increased to 9.25 mph in 1 min. The time required to increase the speed from 16.8 to 27.7 mph will be reduced from 2.725 min to

$$\frac{10.9}{9.25} = 1.18 \text{ min}$$

If the rolling resistance of the haul road is increased to 70 lb per ton, the truck will not be able to attain top speed in fifth gear. From this example it may be seen that the rolling resistance of a haul road is especially effective on the performance of a loaded vehicle in its top gear.

Figures 4-2a and b give charts which may be used to determine the acceleration or deceleration of a vehicle when the gross weight, rolling resistance, and available rimpull are known. The charts also may be used to determine the effect of grade on the performance of a vehicle by converting the effect of the grade into equivalent rolling resistance. Each +1 per cent of grade is equivalent to increasing the rolling resistance 20 lb per ton, and each -1 per cent grade is equivalent to reducing the rolling resistance 20 lb per ton. The combined effect of rolling resistance and grade is represented on line 2 of the chart in Fig. 4-2a.

**EXAMPLE.** A loaded vehicle whose total weight is 70,000 lb enters a +8 per cent grade, on a haul road whose rolling resistance is 60 lb per ton, at a speed of 12 mph. The available rimpull at 12 mph is 2,600 lb. Use the charts in Figs. 4-2a and b to determine the time and the distance traveled before the speed is reduced to 7 mph.

The combined effect of rolling resistance and grade will be

$$\begin{aligned} \text{Rolling resistance} &= 60 \text{ lb per ton} \\ \text{Grade, } 20 \times 8 &= 160 \text{ lb per ton} \\ \text{Combined effect} &= 220 \text{ lb per ton} \end{aligned}$$

The broken line on the two charts gives the solution to the problem.

A line from the gross weight on line 1 through the combined rolling resistance plus grade on line 2 indicates a value of approximately 7,700 lb on line 3.



A line from the required rimpull of 7,700 lb on line 3 to the available rimpull on line 5 indicates a decelerating force of 5,000 lb on line 4.

A line from the 5,000-lb decelerating force on line 6 through the gross weight of the vehicle to line 8 indicates a deceleration of approximately 2.2 ft per sec per sec.

A line from line 8 to line 10 indicates that it will require 3.2 sec to reduce the speed to 7 mph.

A line from line 11 to line 13 indicates that the vehicle will travel 45 ft while the speed is decreasing to 7 mph.

Any problem whose values fall within the limits of the charts can be solved in a similar manner.

### PROBLEMS

**4-1.** A truck whose bed measures 6 ft 3 in. wide and 10 ft 8 in. long, inside dimensions, has a struck capacity of 7.6 cu yd. When it is loaded, the earth fills the bed and extends above the sides on a slope of 2 ft horizontally to 1 ft vertically. Determine the volume of the load, expressed as loose, bank, and compacted measure, when the earth has the following weights per cubic foot:

Undisturbed, 96 lb

Loose, 78 lb

Compacted, 102 lb

**4-2.** A four-wheel tractor whose operating weight is 26,280 lb is pulled along a level haul road at a uniform speed by another tractor. The average tension in the tow cable is 1,160 lb. What is the rolling resistance of the haul road?

**4-3.** A four-wheel tractor whose operating weight is 26,280 lb is pulled along a haul road having a slope of +2 per cent at a uniform speed. The tension in the tow cable is 1,690 lb. What is the rolling resistance of the haul road?

**4-4.** Determine the slope down which a loaded truck will travel at a uniform speed, when coasting freely, if the rolling resistance of the haul road is 120 lb per ton. Will the slope be the same for an empty truck?

**4-5.** A four-wheel tractor whose operating weight is 31,200 lb has a weight distribution between the front and rear wheels of 40 and 60 per cent, respectively. Only the rear wheels are used for driving the unit. When this tractor is pulling a load on a level haul road whose rolling resistance is 80 lb per ton, what is the maximum possible drawbar pull, in pounds, if the coefficient of traction between the road surface and the tires is 0.60?

**4-6.** A two-wheel tractor whose weight on the driving wheels is 36,850 lb is used to pull a total load, including its own weight, of 64,720 lb on a haul road having a rolling resistance of 110 lb per ton of gross load. If the coefficient of traction between the driving tires and the road surface is 0.50, what is the steepest slope up which this tractor can operate at a uniform speed without slipping the driving wheels? Assume that the tractor will have sufficient rimpull to negotiate the slope.

**4-7.** A four-cycle gasoline engine was tested under the given conditions and was found to develop the indicated bhp. Determine the horsepower for standard conditions.

Observed hp, 64.62

Observed pressure, 29.16 in. Hg

Observed temperature, 88°F

**4-8.** A four-cycle diesel engine, with 120 bhp under standard conditions, will be operated at an altitude of 9,600 ft above sea level, where the average temperature will be 86°. Determine the horsepower of the engine at this location.

**4-9.** A crawler tractor whose weight is 18 tons has the indicated drawbar pull for each gear and speed when operating on a level haul road having a rolling resistance of 110 lb per ton. Prepare a table showing the probable drawbar pull for each speed when the tractor is operated on a level road with a rolling resistance of 160 lb per ton.

Gear	Speed, mph	Drawbar pull, lb
1st	1.64	33,910
2d	2.08	26,780
3d	2.84	19,600
4th	3.46	16,100
5th	4.24	13,124
6th	6.18	9,018

**4-10.** A wheel-type tractor, with a 160-hp engine, has a maximum speed in first gear of 3.45 mph. Determine the maximum rimpull of this tractor in each gear if the efficiency is 80 per cent.

Gear	Speed, mph
1st	3.45
2d	6.85
3d	12.16
4th	19.86
5th	29.54

**4-11.** If the tractor of Prob. 4-10 weighs 15.6 tons and is operated over a road with a slope of +3 per cent and a rolling resistance of 90 lb per ton, determine the maximum pull available for pulling a load in second gear.

**4-12.** A loaded vehicle whose total weight is 60,000 lb enters a +6 per cent grade, on a haul road whose rolling resistance is 100 lb per ton, at a speed of 15 mph. The available rimpull at 15 mph is 2,400 lb. Use the charts in Figs. 4-2a and b to determine the time and the distance traveled before the speed is reduced to 5 mph.



## CHAPTER 5

### TRACTORS AND RELATED EQUIPMENT

#### TRACTORS

**Types of Tractors.** Tractors are machines that convert engine energy into tractive energy. Their primary purpose is to pull or push loads, although, at times, they may be used for other purposes. They may be divided into two major types:

1. Crawler
2. Wheel
  - a. Two-wheel
  - b. Four-wheel

Although further divisions are possible, the division given above will be sufficient for considerations in this book. In selecting a tractor several factors should be considered, including the following:

1. The size required for a given job
2. The kind of job for which it will be used, bulldozing, pulling a scraper, pulling a wagon, ripping, etc.
3. The type of footing over which it will travel, high-tractive or low-tractive efficiency
4. The firmness of the haul road
5. The smoothness of the haul road
6. The slope of the haul road
7. The length of haul
8. The type of work to be done after this job is completed

**Crawler Tractors.** The crawler tractor is perhaps the most basic and versatile machine in the construction industry. It serves a multitude of purposes, such as a prime mover for pulling or pushing loads, a power unit for winches and hoists, and a moving mount for bulldozer blades, side booms, and front-end bucket loaders.

Crawler tractors usually are rated by size and power. The size, or weight, is important on many projects, because the maximum pull that a unit can provide will not exceed the product of the weight times the coefficient of traction for the particular road surface, regardless of the power supplied by the engine. Table 4-4 gives the coefficients of traction for various surfaces.

The pull developed at the drawbar is expressed in pounds or as drawbar horsepower. Several travel speeds are available, through the selection of the most suitable gear. As the speed is increased, through the selection of a higher gear, the drawbar pull will be reduced in approximately the same proportion. Thus, for a given unit whose engine is operated at rated power, traveling on a haul road with a uniform slope and rolling resistance, the product of the speed times the drawbar pull will remain approximately constant. Although the specifications for crawler tractors will vary between the several manufacturers, the maximum speeds are



FIG. 5-1. Crawler tractor pulling a self-loading scraper. (International Harvester Co.)

seldom in excess of 7 mph. This type tractor is used primarily where it is necessary to sacrifice high travel speed in order to obtain good traction and high drawbar pull.

Figure 5-1 illustrates a typical crawler tractor. Table 5-1 gives representative specifications for crawler tractors, corrected to standard conditions, sea-level elevation, and 60-deg temperature. The speeds are based on using standard gears, although optional gears may be available.

**Wheel Tractors.** Wheel tractors, equipped with pneumatic tires, have been used as prime movers since 1938. They are the result of efforts to obtain units with higher travel speeds than are possible with crawler tractors. Machines now available have maximum speeds in excess of 30 mph. They are capable of serving many purposes that are served by crawler tractors and are used in competition with crawler tractors on many projects.



TABLE 5-1. REPRESENTATIVE SPECIFICATIONS FOR CRAWLER TRACTORS

Approximate operating weight, lb.....	40,500	26,000	10,700
Belt hp.....	180	93	48.5
Drawbar hp.....	148.4	81	40.5
Ratio, lb per belt hp....	225	280	221
Gauge, center-to-center tracks, in.....	80	74	44
Length of tracks on ground, in.....	104.5	93.25	63.44
Width of track shoes, in.	22	20	13
Pressure under tracks, psi	8.8	7.0	6.5

	Performance								
	Speed		Draw-bar pull, lb	Speed		Draw-bar pull, lb	Speed		Draw-bar pull, lb
	Mph	Fpm		Mph	Fpm		Mph	Fpm	
Gear, forward:									
1st.....	1.6	141	33,714	1.4	123	21,700	1.5	132	9,900
2d.....	2.0	176	26,496	2.2	194	13,500	2.2	194	6,800
3d.....	2.4	211	21,873	3.2	282	9,020	3.0	264	4,750
4th.....	3.1	273	17,025	4.6	405	6,000	3.9	343	3,575
5th.....	4.0	352	12,468	6.0	528	4,270	5.3	466	2,400
6th.....	5.2	458	9,309						
7th.....	6.1	537	7,043						
8th.....	7.8	686	4,892						
Gear, reverse:									
1st.....	1.6	141	.....	1.6	141	.....	1.7	150	.....
2d.....	2.0	176	.....	2.6	229	.....			
3d.....	2.4	211	.....	3.8	334	.....			
4th.....	3.1	273	.....	5.4	475	.....			
5th.....	4.0	352							
6th.....	5.1	449							
7th.....	6.0	528							
8th.....	7.7	678							

The high travel speeds possible with wheel tractors give them an advantage on jobs requiring travel over considerable distances. However, the high travel speeds are obtained at the expense of pulling effort. As indicated in Table 4-4, the coefficient of traction for rubber tires on most haul surfaces is lower than for crawler tracks. Consequently, there is greater possibility of wheel slippage, which may reduce the maximum possible tractive effort. Because of the smaller contact areas between the tires

and the haul surfaces there is a tendency for wheel tractors to sink to greater depths into soft surfaces. As indicated in Table 4-2, this will increase the rolling resistance of wheel tractors compared with crawler tractors.

Wheel tractors usually are rated by size and power. The size, or weight, may determine the maximum possible pull, because this pull is limited to the product of the weight of the unit times the coefficient of traction for the particular road surface, regardless of the power supplied by the engine. Table 4-4 gives the coefficients of traction for various surfaces.

The traction developed by a wheel tractor is expressed in pounds, rimpull. This is a measure of the tractive effort which the engine is capable of delivering to the surface supporting the driving wheels. The net drawbar pull of a wheel tractor is obtained by deducting from the rimpull the pull required to overcome the rolling resistance of the unit when it is traveling on a level haul road. A unit is provided with a range of gears which permit the selection of the speed most suitable for the operating conditions. As the speed is increased through the selection of higher gears, the rimpull will be decreased in approximately the same proportion. Thus, for a given unit whose engine is operated at a rated power, the product of the speed times the rimpull will remain approximately constant.

**Types of Wheel Tractors.** There are two types of wheel tractors, the two- and the four-wheel. Each has certain advantages when compared with the other. The two-wheel type drives and steers with the same wheels. Because of the weight concentration on the driving wheels it may be able to develop more rimpull than a four-wheel unit having the same engine power. The front wheels of a four-wheel unit are used primarily for steering purposes.

Among the advantages claimed for each type, compared with the other, are the following:

#### Two-wheel type

1. Increased maneuverability
2. Increased traction for the driving axle
3. Decreased rolling resistance because of the elimination of the extra axle
4. Fewer tires to provide and maintain

#### Four-wheel type

1. Greater confidence of the operator in the machine because of better steering properties
2. Less tendency to bounce on rough haul roads
3. Probable greater actual speeds, because of (1) and (2) under Four-wheel type



4. Ability to operate as an independent unit when separated from the trailing unit

Figure 5-2 illustrates a two-wheel tractor, while Fig. 5-3 illustrates a four-wheel tractor, each pulling a self-loading scraper. Table 5-2 gives

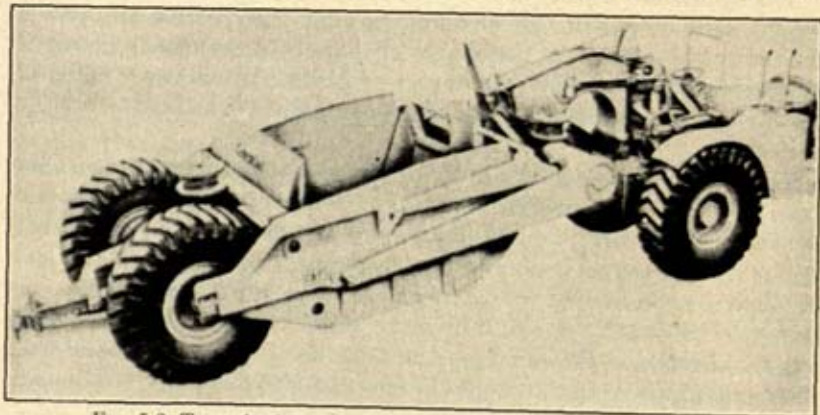


FIG. 5-2. Two-wheel tractor and scraper. (Caterpillar Tractor Co.)



FIG. 5-3. Four-wheel tractor and scraper. (Caterpillar Tractor Co.)

representative specifications for several models of wheel tractors, corrected to standard conditions, sea-level elevation and 60-deg temperatures. The specified speeds and rimpulls are based on using standard gear ratios, although optional ratios are available.

TABLE 5-2. REPRESENTATIVE SPECIFICATIONS FOR WHEEL TRACTORS

	Wheels			
	4	4	2	2
Approximate weight, lb:				
Front wheels...	12,100	7,910		
Drive wheels...	21,800	8,540	32,200	17,740
Engine hp.....	275	115	275	180
Ratio, lb per hp...	123	143	117	98
Tire sizes, in.:				
Front.....	14.00 X 24	12.00 X 20		
Drive.....	24.00 X 29	21.00 X 25	24.00 X 29	21.00 X 25

Gear	Performance							
	Speed, mph	Rim-pull, lb	Speed, mph	Rim-pull, lb	Speed, mph	Rim-pull, lb	Speed, mph	Rim-pull, lb
1st.....	2.88	25,000*	2.8	13,090	2.16	25,000*	3.41	15,850
2d.....	5.57	12,870	4.5	8,145	4.18	17,100	7.25	7,450
3d.....	9.54	7,520	7.2	5,090	7.15	10,050	12.63	4,280
4th.....	16.24	4,420	11.7	3,135	12.18	5,880	22.28	2,420
5th.....	26.60	2,700	18.8	1,950	20.00	3,580	35.03	1,540
Reverse.....	3.72	19,250	2.4	14,100	2.79	25,000*	4.35	12,440

\* These rimpulls are limited by the maximum traction resulting from the weights on the tires, when pulling a loaded scraper.

**Gradability.** Gradability is defined as the maximum slope, expressed as a per cent, up which a crawler or wheel-type prime mover may move at a uniform speed. The gradability may be determined for an empty or a loaded vehicle. Thus, the gradability of a tractor only will be greater than for a tractor that is pulling a loaded vehicle. Gradability may be specified for any desired gear.

The forward motion of a prime mover is limited by the following factors:

1. The power developed by the engine and available as drawbar pull or rimpull.
2. The rolling resistance of the haul road.
3. The gross weight of the prime mover and its load.
4. The grade to be negotiated. Adverse grade adds to the resistance, while favorable grade subtracts from the resistance.

The gradability of a crawler tractor is determined by subtracting from the available drawbar pull the total pull required to overcome the rolling



resistance on the unit and any load that it will pull. The surplus drawbar pull is then available to negotiate a grade. As the drawbar pull of a crawler tractor, taken from the manufacturer's specifications, is usually based on a rolling resistance of 110 lb per ton, any rolling resistance in excess of this amount should be applied to the weight of the tractor. The entire rolling resistance on the towed load should be used. In order to provide a reasonable factor of safety, not more than 85 per cent of the rated drawbar pull of a tractor should be used in determining the gradability of the unit.

**EXAMPLE.** Determine the gradability of a crawler tractor pulling a high-pressure rubber-tired self-loading scraper and its load. The following information is available:

Tractor horsepower, 180	
Weight of tractor, 40,500 lb or 20.25 tons	
Drawbar pull in 1st gear, 33,714 lb	
Available drawbar pull, $0.85 \times 33,714 =$	28,600 lb
Weight of loaded scraper, 78,960 lb or 39.48 tons	
Haul road, rutted, uneven earth	
Rolling resistance for tractor, 160 lb per ton	
Excess rolling resistance for tractor, 50 lb per ton	
Rolling resistance for scraper, 210 lb per ton	
The gradability is determined as follows:	
Rolling resistance of tractor, $20.25 \times 50 =$	1,012 lb
Rolling resistance of scraper, $39.48 \times 210 =$	8,291 lb
Combined rolling resistance	9,303 lb
Drawbar pull available to overcome grade will be	
Max available drawbar pull	28,600 lb
Required for rolling resistance	-9,303 lb
Pull available for grade	19,297 lb
The combined weight of tractor and loaded scraper will be	
Tractor, 20.25 tons	
Scraper, 39.48 tons	
Total 59.73 tons	
Pull required per ton per 1% grade, 20 lb	
Pull required per 1% grade for the total load, $20 \times 59.73 =$	1,195 lb
Max possible grade, $\frac{19,297}{1,195} =$	16%

For the tractor alone the maximum possible grade will be	
Max available drawbar pull	28,600 lb
Pull required for rolling resistance	-1,012 lb
Pull available for grade	27,588 lb
Pull required per 1% grade, $20 \times 20.25 =$	405 lb
Max possible grade, $\frac{27,588}{405} =$	68%, provided the tracks do not slip

The gradability of a wheel-type tractor or a truck can be determined in the same manner as for a crawler tractor. However, if the manufac-

turer's specifications furnish sufficient information, the gradability for any gear can be determined from the formula

$$K = \frac{972 \times T \times G}{R \times W} - \frac{N}{20} \quad (5-1)$$

where  $K$  = gradability, %

$T$  = rated engine torque, lb-ft

$G$  = total gear reduction for particular gear selected

$R$  = rolling radius, the radius of the loaded driving wheels, in., measured from center of axle to surface of ground

$W$  = gross weight of complete unit, lb

$N$  = rolling resistance, lb per ton

EXAMPLE. Determine the gradability of a wheel-tractor-pulled wagon and its load, when operating in third gear at sea level. The following information will apply:

Rated torque at 2,100 rpm, 750 lb-ft

Total gear reduction, 41.0

Rolling radius, loaded, 29.38 in.

Gross weight, 138,500 lb

Rolling resistance, 50 lb per ton

$$K = \frac{972 \times 750 \times 41.0}{29.38 \times 138,500} - \frac{50}{20} = 7.3 - 2.5 = 4.8\%$$

The value, 4.8 per cent, is obtained by using the full torque of the engine. If only 85 per cent of the torque is considered, as a safety precaution, the gradability will be

$$K = 7.3 \times 0.85 - 2.5 = 6.2 - 2.5 = 3.7\%$$

If the rolling resistance of the haul road is permitted to increase to 80 lb per ton, and 85 per cent of the torque is considered, the gradability will be

$$K = 6.2 - \frac{80}{20} = 6.2 - 4.0 = 2.2\%$$

Table 5-3 gives representative gradability for a wheel-tractor-pulled

TABLE 5-3. REPRESENTATIVE GRADABILITY OF A WHEEL TRACTOR AND WAGON

Gear	Speed, mph	Gear reduction	Gradability, %	
			Empty	Loaded
1st	3.1	116.9:1	22.0*	18.8
2d	5.2	70.3:1	22.0*	10.5
3d	9.0	41.0:1	15.3	5.3
4th	15.7	23.5:1	7.8	2.2
5th	24.7	14.9:1	4.3	0.6
Reverse	4.1	90.0:1	22.0*	14.0

\* These values are limited by the maximum traction between the tires and the haul road.



wagon, both empty and loaded, operating at sea level, based on using the full engine torque. In addition the following information is applicable:

Engine, 300 belt hp  
Rated torque at 2,100 rpm, 750 lb-ft  
Rolling radius, 29.38 in.  
Empty weight of tractor and wagon, 58,500 lb  
Gross weight with load, 138,500 lb  
Coefficient of traction, 0.6  
Rolling resistance, 40 lb per ton

### BULLDOZERS

**General Information.** The term bulldozer may be used in a broad sense to include both a bulldozer and an angledozer. These machines may be further divided, on the basis of their mountings, into crawler-tractor- or wheel-tractor mounted. Based on the method of raising and lowering the blade a bulldozer may be classified as cable- or hydraulic-controlled. Each type of equipment has a place in the construction industry. For some projects either type will be satisfactory, while for other projects one type will be superior.

Bulldozers are versatile machines on many construction projects, where they may be used from the start to the finish for such operations as:

1. Clearing land of timber and stumps
2. Opening up pilot roads through mountains and rocky terrain
3. Moving earth for haul distances up to approximately 300 ft
4. Helping load tractor-pulled scrapers
5. Spreading earth fills
6. Backfilling trenches
7. Clearing construction sites of debris
8. Maintaining haul roads
9. Clearing the floors of borrow and quarry pits

Bulldozers are mounted with the blades perpendicular to the direction of travel, while angledozers are mounted with the blades set at an angle with the direction of travel. The former push the earth forward, while the latter push it forward and to one side. Some blades may be adjusted to permit their use as bulldozers or angledozers. The size of a bulldozer is indicated by the length and height of the blade. Plates may be installed at the ends of a blade to reduce the spillage when a machine is used for moving earth.

**Cable Control versus Hydraulic Control.** There are numerous arguments about the merits of cable control versus hydraulic control of the raising and lowering of blades. Neither is superior to the other under all operating conditions.

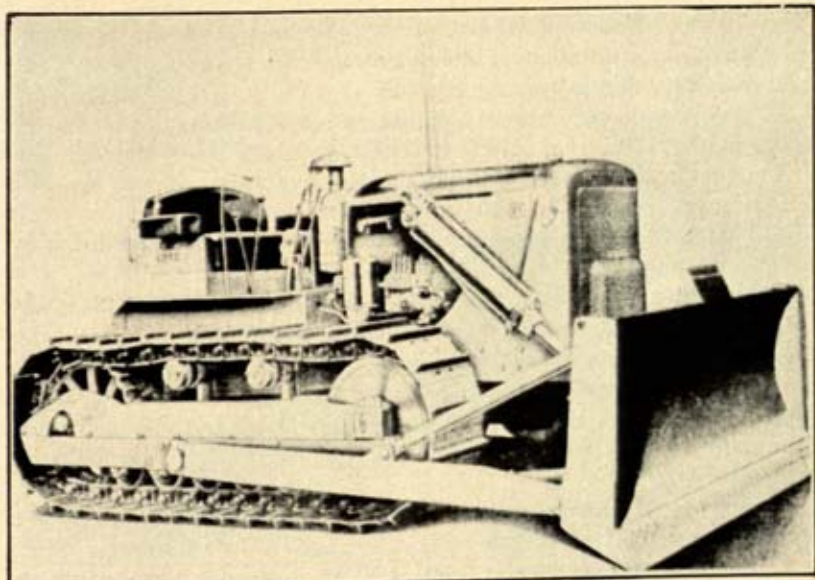


FIG. 5-4. Cable-controlled bulldozer. (Caterpillar Tractor Co.)

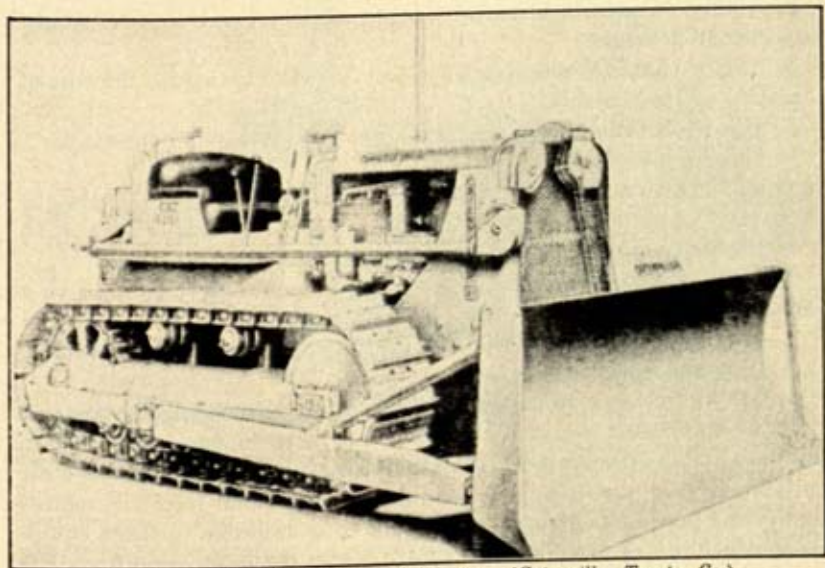


FIG. 5-5. Hydraulic-controlled bulldozer. (Caterpillar Tractor Co.)



Among the advantages claimed for the cable control are the following:

1. Simplicity of installation and operation
2. Simplicity of repairing the controls
3. Reduction in the danger of damaging a machine, as the blade can move up and ride over a rigid obstruction, such as a heavy boulder

Among the advantages claimed for the hydraulic control are the following:

1. Ability to produce a high down pressure on the blade, in addition to its weight, to force the blade into the ground

2. Ability to maintain a more precise setting of the position of the blade

**Crawler-mounted versus Wheel-mounted Bulldozers.** At one time bulldozers were mounted on crawler tractors only. However, with the development of wheel tractors, bulldozers have been mounted on them also. Each type of mounting has advantages under certain conditions. For some jobs the conditions are such that either type may be used satisfactorily.

Among the advantages claimed for the crawler-mounted bulldozer are the following:

1. Ability to deliver greater tractive effort, especially in operating on soft footing, such as loose or muddy soil
2. Ability to travel over muddy surfaces
3. Ability to operate in rocky formations, where rubber tires might be seriously damaged
4. Ability to travel over rough surfaces, which may reduce the cost of maintaining haul roads
5. Greater flotation because of the lower pressures under the tracks
6. Greater use versatility on jobs

Among the advantages claimed for wheel-mounted bulldozers are the following:

1. Higher travel speeds on the job or from one job to another
2. Elimination of hauling equipment to transport the bulldozer to a job
3. Greater output, especially when considerable traveling is necessary
4. Less operator fatigue
5. Ability to travel on paved highways without damaging the surfaces

If the equipment user has a job which is large enough to justify the purchase of special equipment, he should select the equipment that is most suitable for the particular job. However, since small jobs will seldom justify the purchase of special equipment, it is desirable to select equipment which can be used on other jobs. Under the latter conditions, the selection of versatile equipment will usually be a wise choice.

Figure 5-6 illustrates a crawler-mounted bulldozer opening up construction on a precarious mountain road-building job where sharp rocks

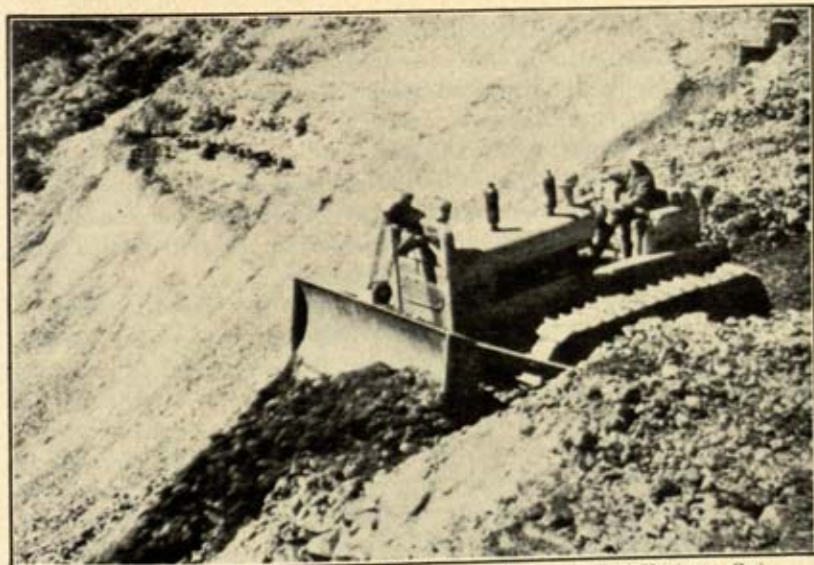


FIG. 5-6. Crawler-tractor-mounted bulldozer. (*International Harvester Co.*)

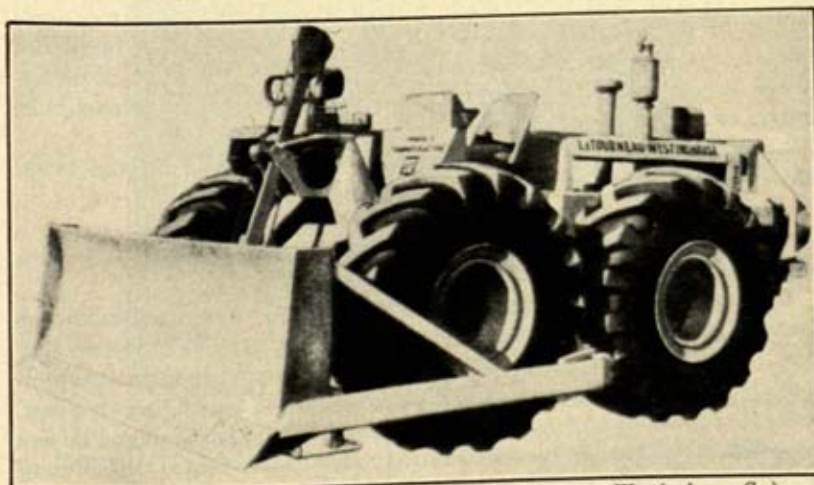


FIG. 5-7. Wheel-tractor-mounted bulldozer. (*LeTourneau-Westinghouse Co.*)

predominate. For work of this type the crawler-mounted bulldozer is usually superior to the wheel-mounted. Figure 5-7 illustrates a wheel-mounted bulldozer.

**Clearing Land with a Bulldozer.** The bulldozer is used extensively for clearing land. If the trees are not too large, the dozer blade is lowered



just below the surface of the ground, and as the tractor moves forward in the most suitable gear, the brush and timber are pushed ahead of it. A special dozer blade, equipped with teeth that project down, may be used to prevent the removal of topsoil.

If the trees are large, it may be possible to increase the leverage sufficiently by raising the blade to the full height to push them down. If this is not possible, it will be necessary to lower one edge of the blade and excavate deep enough around the trees to cut the horizontal roots. Then the trees can be pushed over with less difficulty.

Another method of land clearing, where it is not necessary to remove the underbrush in the first operation, is to fasten each end of a wire rope to a large tractor and pull it through the timber. The rope will pull up most of the trees by the roots and cut off others. Wire ropes, varying in size from  $1\frac{1}{4}$  to 2 in., have been used. If it is desirable to keep the rope off the ground, a hollow steel ball, approximately 8 ft in diameter, may be connected to the rope at its mid-point. If the tractors do not have sufficient tractive power to pull the rope through the timber, they may be anchored behind large trees and the rope pulled in with winches. In clearing the Hungry Horse Reservoir the contractor used two tractors, with up to 900 ft of  $1\frac{1}{4}$ -in. rope, to clear an average of 100 acres per 8-hr day. Trees up to 4 ft in diameter were felled. After the trees were pulled down, they were stacked with tractor-mounted timber rakes and burned. As the wire rope did not remove all the small trees and underbrush, it was necessary to complete the clearing with conventional bulldozers or other equipment.

Large V-shaped saw blades, with heavy teeth that tear through trees, have been mounted on the front of crawler tractors to cut trees up to 24 in. in diameter, at ground level. Circular saws, up to 60 in. in diameter, mounted on the front of tractors, have been used to cut trees at or slightly above ground level.

**Moving Earth with Bulldozers.** Under certain conditions bulldozers are satisfactory machines for moving earth for such jobs as excavating ponds for stock water, trench silos and highway cuts, stripping the topsoil from land or ore deposits, constructing low levees, backfilling trenches, spreading material on fills, etc. In general, haul distances should be less than 300 ft. Either a crawler-mounted or a wheel-mounted tractor may be used, a crawler-mounted machine having an advantage on short hauls with soft or muddy ground, and a wheel-mounted machine possibly having an advantage on longer hauls and firm ground.

The output of a bulldozer will vary with the conditions under which it operates. During the first passes over a given lane most of the initial earth will spill off the ends of the blade to form a windrow on each side of the lane. After these windrows have been built up to form a trench,

further end spillage will be reduced or eliminated, with a substantial increase in output. Steel plates on the end of a blade will reduce end spillage. On some jobs two bulldozers, working side by side, with adjacent ends of the blades in contact, have been used to increase the output as much as 50 per cent over the combined outputs of two machines working separately. If earth can be pushed downhill, the output of a machine will be increased substantially because of the advantage of the favorable grade and the ability to float larger quantities of earth ahead of the



FIG. 5-8. Clearing land with a wire rope and crawler tractors. (*The Constructor.*)

machine. Figure 5-9 illustrates two bulldozers operating under favorable conditions, namely, downhill dozing, slot excavation, side-by-side operation, and floating extra earth ahead of the machine.

**The Output of Bulldozers.** The blade of a bulldozer has a theoretical capacity which varies with the class of earth and the size of the blade. If the capacity of a blade is known, one can determine the approximate output of a machine by estimating the number of passes it will make in an hour.





FIG. 5-9. Operating two bulldozers side by side. (LeTourneau-Westinghouse Co.)

**EXAMPLE.** Estimate the approximate output of a bulldozer for the given conditions:

Material, sandy loam topsoil, weight 2,700 lb per cu yd bm

Swell, 25%

Haul distance, 100 ft, over level ground, with bulldozer operating in a slot

Crawler tractor, 72 drawbar hp

Moldboard size, 9 ft 6 in. long, 3 ft 0 in. high

Rated moldboard capacity, 3.6 cu yd loose volume

Net moldboard capacity,  $3.6 \div 1.25 = 2.9$  cu yd bm

Operating factor, 50-min hr

Probable round-trip time will be

Pushing, 100 ft @ 1.5 mph	= 0.758 min
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Returning, 100 ft @ 3.5 mph	= 0.324 min
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Fixed time, loading and shifting gears	= 0.320 min
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Total time	= 1.402 min
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Trips per hr,  $50 \div 1.402 = 35.7$

Output per hr, 35.7 trips @ 2.9 cu yd = 103.4 cu yd bm

This output is based on favorable operating conditions which permit a load equal to the maximum capacity of the dozer. For most projects the load will be less than the maximum possible capacity. For example, if the earth is ordinary soil, the load might be reduced to 2.0 cu yd bank measure. With other conditions remaining the same the output per hour will be

35.7 trips @ 2.0 cu yd = 71.4 cu yd bm

The approximate capacity of a bulldozer blade may be determined from the size of the load pushed by the blade. Actual measurements of representative loads will give better results than estimates. For example, if a blade 9 ft 6 in. long by 3 ft 0 in. high is used to push earth in a slot or trench whose height is about equal to that of the blade, it is possible to fill the blade to full length and height. Although the shape of the front slope of the earth will be irregular, assume that it is equivalent to a 2:1 slope. The size of the load will be 9 ft 6 in. long, 3 ft 0 in. high, and 6 ft 0 in. wide. The loose volume will be

$$\frac{9.5 \times 3 \times 6}{2 \times 27} = 3.2 \text{ cu yd}$$

For a swell of 25 per cent the net volume will be

$$3.2 \div 1.25 = 2.56 \text{ cu yd bm}$$

If the dozing is done without slots, the capacity of the blade will be reduced by approximately 25 per cent. Also, if the earth is so hard that a full load cannot be moved, the capacity must be reduced accordingly.

Table 5-4 gives approximate blade capacities and outputs in cubic yards bank measure for various sizes of blades and tractors. The information given in the table is based on pushing full loads in slots. It is assumed that the tractors will push the loads forward in first gear, then return for another load in reverse gear. It is assumed that the tractors will operate 50 min per hr. For other job conditions the outputs given in the table must be modified.

TABLE 5-4. REPRESENTATIVE BLADE CAPACITIES AND BULLDOZER OUTPUT, IN CUBIC YARDS BANK MEASURE

Blade size		Tractor, drawbar hp	Speed		Blade capacity, cu yd	Output, cu yd per hr			
Length	Height, in.		Forward, fpm	Reverse, fpm		Haul distance, ft			
						100	200	300	400
11 ft 3 in.	45½	130	150	326	4.8	184	105	74	57
10 ft 3 in.	45½	80	123	334	4.4	152	86	60	46
9 ft 6 in.	38	65	123	343	2.8	98	55	38	29
8 ft 2 in.	38	65	123	343	2.4	84	47	33	25
7 ft 2 in.	32½	43	150	167	1.5	47	26	18	14
5 ft 8 in.	27½	32	150	185	0.9	29	16	11	9
11 ft 2 in.	43	210*	141	712	4.2	178	103	73	56
11 ft 3 in.	36	122*	141	712	3.0	127	74	52	40

\* These values are bhp for wheel-mounted dozers.



### TRACTOR-PULLED SCRAPERS

**General Information.** Tractor-pulled scrapers have established an important position in the earth-moving field. As they are self-operating, to the extent that they can load, haul, and discharge material, they are not dependent on other equipment. If one of them experiences a temporary breakdown, it is not necessary to stop the job, as would be the case for a machine which is used exclusively for loading earth into hauling units, for if the loader breaks down, the entire job must stop until repairs can be made. The self-loading scrapers are available with capacities up to 30 cu yd or more.

These machines are the result of a compromise between the best loading and the best hauling machines, and, as must be expected of any composite machine, they are not superior to other equipment in both loading and hauling. Power shovels, draglines, and Euclid loaders usually will surpass them in loading only, while trucks may surpass them in hauling only, especially when long, well-maintained haul roads are used. However, their ability to load and haul earth gives them a definite advantage on many projects. The development of high-speed wheel-type tractors has increased the economic haul distance for this type equipment up to a mile or more on many projects.

The ability of these machines to deposit their loads in uniformly thick layers will facilitate the succeeding spreading operations. On the return trips to borrow pits the cutting blades of scrapers may be lowered enough to remove high spots, thereby assisting in maintaining the haul roads.

Earth frequently is found in stratified layers, which must be blended by mixing the materials from several layers. The limited depth of cut will not permit scrapers to mix the layers satisfactorily. For this reason shovels and trucks sometimes are used, even though scrapers will handle the earth more economically.

The advantages and disadvantages previously given for crawler compared with wheel tractors will, in most instances, apply to tractor-pulled scrapers.

**Crawler-tractor Scrapers.** For relatively short haul distances the crawler-type tractor, pulling a rubber-tired self-loading scraper, can move earth economically. The high drawbar pull in loading a scraper, combined with good traction, even on poor haul roads, gives the crawler tractor an advantage for short hauls. However, as the haul distance is increased, the low speed of a crawler tractor is a disadvantage compared with a wheel tractor.

Unless the loading operation is difficult, a crawler tractor can load a scraper without the aid of a bulldozer. However, if there are several scraper units on a job, the increased output resulting from using a bulldozer to help load the scrapers usually will justify the use of a bulldozer.

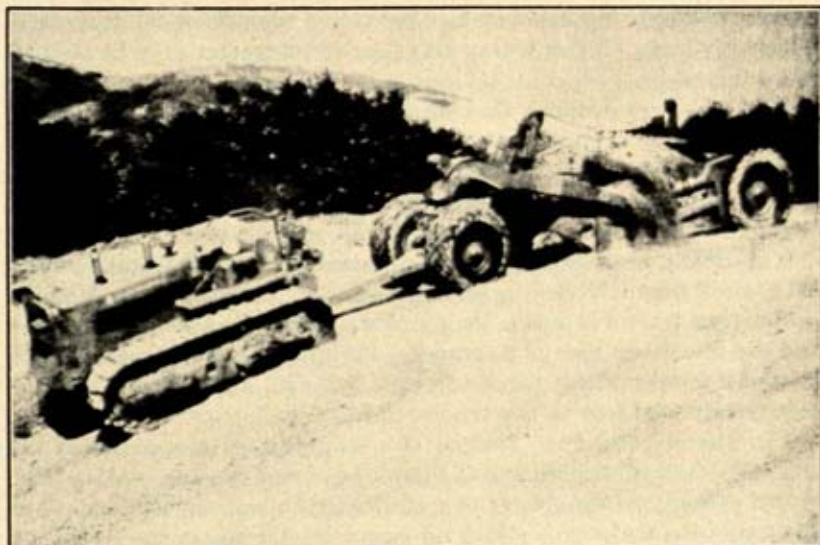


FIG. 5-10. Crawler tractor and self-loading scraper. (*International Harvester Co.*)

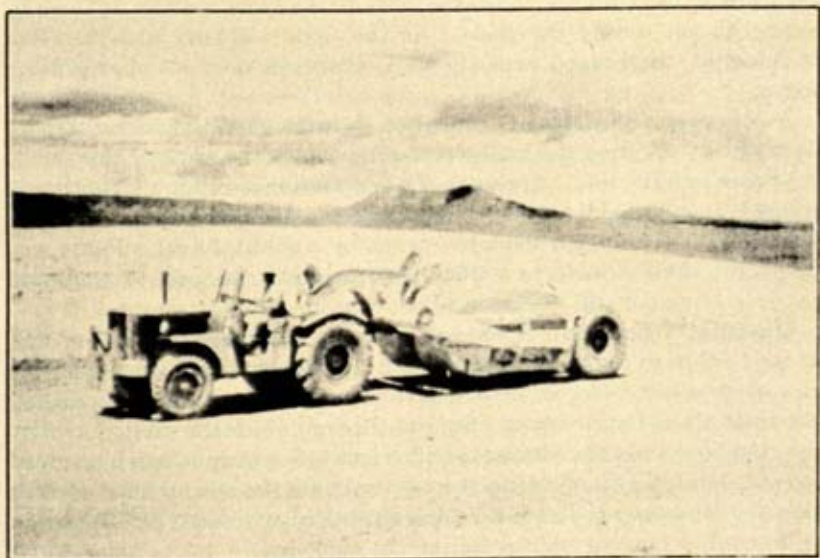


FIG. 5-11. Wheel tractor and self-loading scraper. (*Caterpillar Tractor Co.*)

**Wheel-tractor Scrapers.** For longer haul distances the higher speed of a wheel-type tractor-pulled self-loading scraper will permit it to move earth more economically than with a crawler-type tractor. Although the wheel-type tractor cannot deliver so great a tractive effort in loading a scraper, the higher travel speed, which may exceed 30 mph for some



models, will offset the disadvantage in loading when the haul distance is sufficiently long. Either a two- or a four-wheel tractor may be used to pull a scraper.

The break-even distance, the haul distance at which the cost of hauling with a crawler or a wheel tractor will be the same, may be determined by making an analysis of a given job. The analysis should consider the class of soil, the condition of the borrow pit, the condition, length, and slope of the haul road, the nature of the fill, and weather.

It is usually necessary to use a helper tractor, such as a bulldozer, to aid a wheel tractor while it is loading a scraper. One bulldozer can help load several tractor scrapers, the number depending on the loading time and the round-trip time of a scraper. For example, if it requires 1 min to load a scraper whose round-trip time is 5 min, a bulldozer should be able to help load four to five scraper units.

**The Size of Scrapers.** The size of a scraper may be specified as the struck, or heaped, capacity of the bowl, expressed in cubic yards. The struck capacity is the volume of material that a scraper will hold when the top of the material is struck off even with the top of the bowl. In specifying the heaped capacity of a scraper some manufacturers specify the slope of the material above the sides of the bowl, usually 1:1, while others do not specify the slope. As the slope will vary with the class of material, the heaped capacity of a scraper is only an approximate value.

The capacity of a scraper, expressed in cubic yards bank measure, is obtained by dividing the loose volume by 1 plus the swell of the earth, expressed as a fraction. Owing to the compacting effect on the earth in a scraper, as additional earth is forced into the bowl, the swell usually is less than for earth deposited into a truck by a power shovel. For example, if the swell of earth in a truck is 25 per cent, the swell of the same earth in a scraper will be about 20 per cent.

**Operating a Scraper.** A scraper is loaded by lowering the front end of the bowl until the cutting edge, which is attached to and extends across the width of the bowl, enters the ground and, at the same time, raising the front apron to provide an open slot through which the earth may flow into the bowl. As the scraper is pulled forward, a strip of earth is forced into the bowl. This operation is continued until the bowl is filled or until no more earth may be forced in. The cutting edge is raised and the apron is lowered to prevent spillage during the haul trip.

The dumping operation consists of lowering the cutting edge to the desired height above the fill, raising the apron, and forcing the earth out between the blade and the apron by means of a movable ejector mounted at the rear of the bowl.

Scrapers are available with either cable or hydraulic controls for operating the apron, ejector, and bowl positions.

**Factors Which Affect the Output of Tractor-pulled Scrapers.** The output of a tractor-pulled scraper will be affected by numerous factors which must be considered in estimating the probable production rate for a job. Among the factors are the following:

1. Size of tractor
2. Class of material
3. Size and condition of borrow pit or cut
4. Slope of loading zone
5. Extent of loosening material prior to loading
6. Use of a helper tractor during loading
7. Haul distance
8. Condition and slope of haul road
9. Altitude
10. Climatic conditions
11. Management conditions

The class of material will affect the ease or difficulty of loading a scraper, while the swell will affect the net load carried by a scraper.

If the borrow pit is large enough to permit easy maneuvering by the equipment, the time used in loading should be less than for a small, congested pit, where one machine is frequently in the way of another unit.

If the pit or cut is such that loading can be done while a machine is moving downhill, the favorable slope will have the effect of giving the tractor additional tractive effort. For example, consider a scraper with a heaped capacity of 18 cu yd, pulled by a crawler tractor having 130 drawbar hp. The combined empty weight of the two machines is about 31 tons. The combined loaded weight is about 54 tons. Assume that the machine is to load down a 5 per cent grade. At the start of loading the effect of the grade will be to add  $5 \times 20 \times 31 = 3,100$  lb of drawbar pull to the tractor. When the scraper is filled, the effect of the grade is to add  $5 \times 20 \times 54 = 5,400$  lb of drawbar pull, compared with loading on level ground. The latter figure, 5,400 lb, is equivalent to adding 28 hp of drawbar pull to the prime tractor when it is loading at 1.7 mph. This is equal to an increase of approximately 21 per cent in drawbar pull. On this basis, each 1 per cent of favorable slope will increase the equivalent drawbar pull by about 4 per cent. If earth is to be excavated from a hill and hauled to both sides of the hill, loading should always be done on the downslope.

If the earth is hard, it frequently will pay to loosen it with a tractor-pulled ripper. For some materials, such as hard clay and hardpan, this is the only way that a scraper can be loaded. If a contractor is paid \$0.20 per cubic yard for hauling earth, spending \$8.00 per hour for a tractor-pulled ripper that will increase the output 80 cu yd per hr is a good investment.



When a helper tractor will increase the value of the output in excess of its operating cost, it should be used.

**The Output of Tractor-pulled Scrapers.** The probable output of a tractor-pulled scraper may be determined by multiplying the average net volume per load by the number of loads per hour, considering an appropriate operating factor. This information may be difficult to obtain prior to starting a project. Consequently judgment must be applied to mathematical calculations.

While the manufacturer's specifications will give the struck and heaped capacity of a scraper, this information is useful only as a guide in esti-

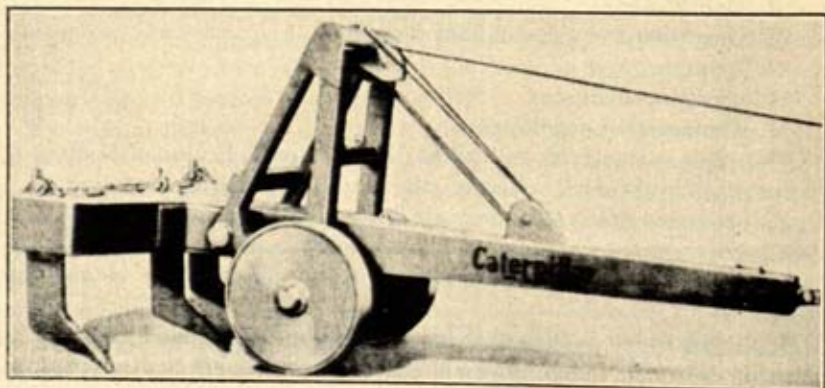


FIG. 5-12. Ripper. (Caterpillar Tractor Co.)

imating the actual capacity. The average load probably will exceed the struck capacity, but it seldom will equal the heaped capacity. A realistic value should be used in analyzing a job.

The number of trips per hour will depend on the average time required for a round trip and on the operating factor. The time required to make a trip will include:

1. Loading
2. Traveling to the fill
3. Dumping and turning
4. Returning to the pit
5. Getting in position to load

Items 1, 3, and 5 are sometimes combined into a single value, called fixed time, as they are reasonably constant for a given job. The variable time will include traveling between the pit and the fill.

The following example will illustrate a method of estimating the probable output of a scraper unit.

**EXAMPLE.** Determine the probable output of a scraper unit, in cubic yards bank measure, for the given conditions:

Crawler tractor, 80 drawbar hp  
 Weight, 24,600 lb  
 Tractor speeds and drawbar pulls are

Gear	Speed		Drawbar pull, lb
	Mph	Fpm	
1st	1.4	123	21,351
2d	2.2	194	13,454
3d	3.2	282	9,090
4th	4.6	405	5,994
5th	6.0	528	4,550

Scraper, 8.7 cu yd struck, 11 cu yd heaped capacity  
 Empty weight, 18,800 lb  
 Width of cut, 8 ft  
 Class of earth, sandy clay, bank weight, 2,700 lb per cu yd  
 Loading conditions, level pit, using push tractor  
 Haul road, level, rolling resistance for rubber tires, 150 lb per ton  
 Haul distance, 800 ft, one way  
 Fill depth per layer, 9 in. loose measure  
 For loading assume depth of cut to be 4 in.  
 Assume loose volume of scraper to be 10 cu yd  
 Assume earth swell to be 20%  
 Net volume of load,  $10 \div 1.2 = 8.3$  cu yd bm

Length of cut, 8 ft wide, 4 in. deep, required to fill scraper,  $\frac{8.3 \times 27}{8 \times 0.333} = 85$  ft

While emptying, travel distance =  $\frac{10 \times 27}{8 \times 0.75} = 45$  ft

Combined weight of scraper and load will be

Scraper = 18,800 lb	= 9.4 tons
Load, 8.3 cu yd $\times$ 2,700 lb = 22,400 lb	= 11.2 tons
Total weight	= 20.6 tons

Drawbar pull required for loaded scraper, 20.6 tons  $\times$  150 lb per ton = 3,090 lb

The maximum haul speed, loaded and empty, is 6.0 mph, or 528 fpm. However, the operator probably will limit the speed to 5 mph, or 440 fpm.

The total time for a round trip may be divided into fixed time and variable time. Fixed time includes loading, spreading, turning, and accelerating, because these are relatively constant for given equipment on a job.

For the given crawler-tractor unit the fixed time should be about as follows:

Loading, 85 ft $\div$ 123 fpm	= 0.690 min
Spreading, 45 ft $\div$ 123 fpm	= 0.366 min
Turning, 2 turns $\times$ 0.25 min each	= 0.500 min
Lost time, shifting and accelerating	= 0.694 min
Total fixed time	= 2.25 min

For a crawler-tractor unit the fixed time will vary from 2.25 to 3.0 min, depending on job conditions.



The probable round-trip time will be

Fixed time	= 2.25 min
Traveling, 1,600 ft ÷ 440 fpm	= 3.64 min
Total time	= 5.89 min
Assume a 45-min hr	
No. trips per hr, 45 ÷ 5.89	= 7.65
Output per hr, 7.65 trips × 8.3 cu yd	= 63.5 cu yd bm

If a 115-hp wheel-type tractor is used to pull the scraper, with all other conditions the same, the output may be determined in a similar manner.

The tractor speeds and rimpulls are

Gear	Speed		Rimpull, lb
	Mph	Fpm	
1st	2.6	229	14,100
2d	4.2	370	8,730
3d	6.8	598	5,390
4th	10.9	957	3,360
5th	17.6	1,547	2,080

Weight of tractor, 16,600 lb	= 8.30 tons
Weight of scraper, 17,300 lb	= 8.65 tons
Weight of tractor and scraper	= 16.95 tons
Weight of load, $\frac{8.3 \times 2,700}{2,000}$	= 11.20 tons
Gross weight	= 28.15 tons

Experience has indicated that for wheel-type tractor machines it requires about 20 lb of rimpull per ton to provide the desired acceleration.

The required rimpull will be

Rolling resistance, loaded, 28.15 tons × 150 lb	= 4,225 lb
Acceleration, loaded, 28.15 tons × 20 lb	= 563 lb
Total rimpull required, loaded	= 4,788 lb
Rolling resistance, empty, 16.95 tons × 150 lb	= 2,545 lb
Acceleration, empty, 16.95 tons × 20 lb	= 339 lb
Total rimpull required, empty	= 2,884 lb
Max speed, loaded, 6.8 mph	
Max speed, empty, 10.9 mph	

The fixed time for a wheel-type, tractor-pulled scraper should be about as follows:

Operation	Minimum, min	Maximum, min
Loading	1.0	1.5
Spreading	0.5	0.5
Turning, 2 turns @ 0.25 min	0.5	0.5
Shifting and accelerating	0.5	1.0
Total time	2.5	3.5

The probable round trip time will be

Fixed time	= 3.00 min
Traveling to fill, 800 ft ÷ 598 fpm	= 1.34 min
Returning to pit, 800 ft ÷ 957 fpm	= 0.83 min
Total time	= 5.17 min
Assume a 45-min hr	
No. trips per hr, 45 ÷ 5.17	= 8.7
Output per hr, 8.7 trips × 8.3 cu yd	= 72 cu yd bm

The cost of hauling earth with a tractor-pulled scraper should be determined by adding to the cost of a tractor-scraper unit a proportionate part of the cost of the bulldozer required to aid in loading the scraper. For the two types of machines previously analyzed it should be possible for a bulldozer to help load four scraper units. Thus, one-fourth of the operating cost of a bulldozer should be charged to each scraper unit. The cost of a unit does not include the wages paid to an operator.

The cost per cubic yard of earth, using each type of hauling unit, should be about as follows:

Cost of crawler tractor and scraper	= \$7.50 per hr
One-fourth the cost of a bulldozer	= 1.50 per hr
Total cost	= \$9.00 per hr
Hauling cost per cu yd, \$9.00 ÷ 63.5	= 0.142
Cost of wheel tractor and scraper	= \$8.00 per hr
One-fourth the cost of a bulldozer	= 1.50 per hr
Total cost	= \$9.50 per hr
Hauling cost per cu yd, \$9.50 ÷ 72	= 0.132

Table 5-5 gives comparative outputs in cubic yards per hour and costs of moving earth using crawler and wheel-type tractors to pull the scrapers.

TABLE 5-5. OUTPUT AND COST OF MOVING EARTH WITH  
TRACTOR-PULLED SCRAPERS

Haul distance, one way, ft	Crawler tractor, 84 hp		Wheel tractor, 115 hp	
	Output per hr, cu yd	Cost per cu yd	Output per hr, cu yd	Cost per cu yd
200	123	\$0.077	106	\$0.095
400	97	0.098	92	0.109
600	81	0.111	81	0.117
800	69	0.130	73	0.130
1,000	60	0.145	66	0.139
1,500	45	0.188	53	0.170
2,000	36	0.219	45	0.197
3,000	26	0.310	34	0.255
4,000	21	0.378	27	0.317
5,000	17	0.460	23	0.368



## 110 CONSTRUCTION PLANNING, EQUIPMENT, AND METHODS

The haul roads are level, with the rolling resistance equal to 150 lb per ton for rubber tires. The tractors and scrapers used are as follows:

- Crawler tractors, 84 drawbar hp
- Wheel tractors, 115 belt hp
- Scrapers, 8.7 cu yd struck, 11 cu yd heaped capacity, net load 8.3 cu yd bm
- Bulldozers to help load, 84 drawbar hp
- Cost of crawler tractor and scraper, \$7.50 per hr
- Cost of wheel tractor and scraper, \$8.00 per hr
- Cost of bulldozer, \$6.00 per hr
- Operating factor, 45 min per hr

EXAMPLE. The information given in the previous analysis and in Table 5-5 is based on a level haul road. However, many haul roads are not level and do not have a uniform grade for the full haul distance. Assume that earth is to be hauled 3,600 ft, one way, over the following haul road:

Distance, ft	Grade for haul, %
1,200	0
1,600	8, favorable
800	6, adverse

Class of material, common earth

Weight, bm, 2,400 per cu yd

Swell, 25%

Rolling resistance along haul road, 80 lb per ton

Coefficient of traction, 0.6

Tractor, 2-wheel, 186 belt hp

Gear	Speed, mph	Rimpull, lb
1st	3.41	17,400
2d	7.25	8,200
3d	12.63	4,700
4th	22.28	2,660
5th	35.03	1,695

Scraper, 2-wheel, capacity, 15 cu yd loose volume

Net volume =  $15 \div 1.25 = 12$  cu yd

Weight of tractor and scraper = 17.0 tons

Weight of load,  $\frac{12 \text{ cu yd} \times 2,400 \text{ lb}}{2,000} = 14.4$  tons

Gross weight = 31.4 tons

Driving tires carry 50% of weight, loaded or empty

The maximum traction on the driving wheels, loaded, is

$$31,400 \text{ lb} \times 0.60 = 18,840 \text{ lb}$$

Thus, the maximum rimpull at all speeds is available.

The maximum traction on the driving wheels, empty, is

$$17,000 \text{ lb} \times 0.60 = 10,200 \text{ lb}$$

This is the maximum available rimpull, regardless of the gear used.

The time for a round trip is determined as follows:

1. Fixed time, assumed to be 2.5 min
2. 1,200 ft of 0% grade, loaded
 

Rolling resistance, 31.4 tons $\times$ 80 lb	=	2,512 lb
Acceleration, 31.4 tons $\times$ 20 lb	=	628 lb
Total rimpull required	=	3,140 lb
Max speed, 3d gear, 12.63 mph		
Travel time, $\frac{1,200}{12.63 \times 88}$	=	1.08 min
3. 1,600 ft of -8% grade, loaded
 

Rolling resistance, 31.4 tons $\times$ 80 lb	=	2,512 lb
Acceleration, 31.4 tons $\times$ 20 lb	=	628 lb
Total	=	3,140 lb
Deduct effect of grade, 31.4 tons $\times$ 160	=	5,024 lb
Total rimpull required	=	-1,884 lb
Probable max speed due to operator, 30 mph		
Travel time, $\frac{1,600}{30 \times 88}$	=	0.61 min
4. 800 ft of +6% grade, loaded
 

Rolling resistance and acceleration	=	3,140 lb
Grade resistance, 31.4 tons $\times$ 120 lb	=	3,768 lb
Total rimpull required	=	6,908 lb
Max speed in 2d gear, 7.25 mph		
Travel time, $\frac{800}{7.25 \times 88}$	=	1.25 min
5. 800 ft of -6% grade, empty
 

Rolling resistance, 17 tons $\times$ 80 lb	=	1,360 lb
Acceleration, 17 tons $\times$ 20 lb	=	340 lb
Total	=	1,700 lb
Deduct effect of grade, 17 tons $\times$ 120 lb	=	2,040 lb
Total rimpull required	=	-440 lb
Probable max speed due to operator, 30 mph		
Travel time, $\frac{800}{30 \times 88}$	=	0.30 min
6. 1,600 ft of +8% grade, empty
 

Rolling resistance and acceleration	=	1,700 lb
Grade resistance, 17 tons $\times$ 160 lb	=	2,720 lb
Total rimpull required	=	4,420 lb
Max speed, 3d gear, 12.63 mph		
Travel time, $\frac{1,600}{12.63 \times 88}$	=	1.44 min
7. 1,200 ft of 0% grade, empty
 

Rolling resistance and acceleration	=	1,700 lb
Required rimpull	=	1,700 lb
Max speed, 5th gear, 30 mph		
Travel time, $\frac{1,200}{30 \times 88}$	=	0.46 min



The total time for a round trip will be

Fixed time	= 2.50 min
Travel, loaded	= 2.94 min
Travel, empty	= 2.20 min
Total	= 7.64 min
No. trips per 50-min hr,	$50 \div 7.64 = 6.55$
Output per hr,	$6.55 \text{ trips} \times 12 \text{ cu yd} = 78.5 \text{ cu yd bm}$

The method used in the previous project for analyzing a hauling problem may be applied to any project, provided the operating conditions are reasonably well known.

**The Effect of Grade on the Output of Tractor-pulled Scrapers.** Figure 5-13 illustrates the effect of grade on the output of and the cost of hauling earth with tractor-pulled scrapers. This information is given for a crawler and a wheel-type machine.

The job conditions are

- Haul distance, 800 ft, one way
- Rolling resistance, 100 lb per ton for rubber tires
- Coefficient of traction, 0.6
- Class of material, common earth, weight, 2,400 lb per cu yd bm

The crawler-tractor specifications are

- Tractor, 84 drawbar hp
- Weight, 24,600 lb
- Operating cost per hr, tractor and scraper, \$7.50

The wheel-tractor specifications are

- Tractor, 115 belt hp
- Weight, 16,600 lb
- Operating cost per hr, tractor and scraper, \$8.00

The scraper specifications are

- Capacity, 8.3 cu yd bm
- Empty weight, 17,300 lb
- Operating cost of bulldozer per hr, \$6.00

The figure shows that the cost of hauling earth with the wheel unit is lower than with the crawler unit for all grades, which does not correspond with the information given in Table 5-5. The reason for the difference is the reduced rolling resistance of the haul road used for Fig. 5-13. This illustrates the danger of comparing equipment without an engineering analysis.

**The Effect of Rolling Resistance on the Output of Tractor-pulled Scrapers.** A job condition which is sometimes not given adequate consideration is the effect of rolling resistance of a haul road on the output of a tractor-pulled scraper and the cost of hauling earth. Figure 5-14 illustrates this effect on a crawler and a wheel-type tractor-pulled unit. The information in the table is based on a level haul road, 800 ft long, one way. The tractors and scrapers are the same units that were used in the analysis for Fig. 5-13.

The effect of rolling resistance on the output for a given job may be

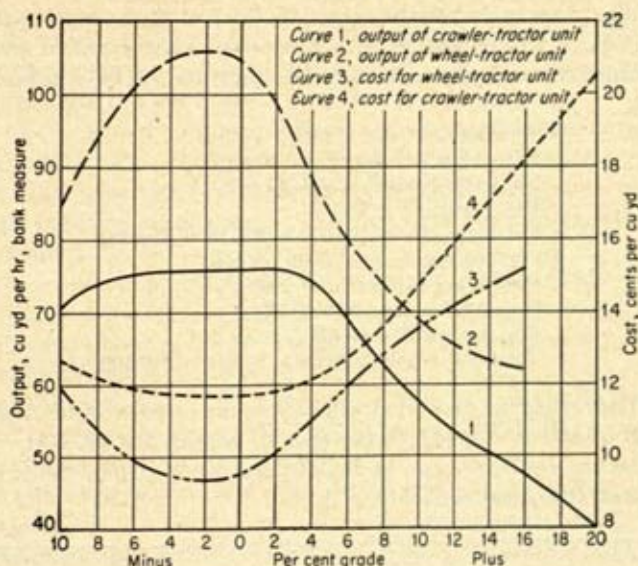


FIG. 5-13. The effect of grade on the output of tractor-pulled scrapers.

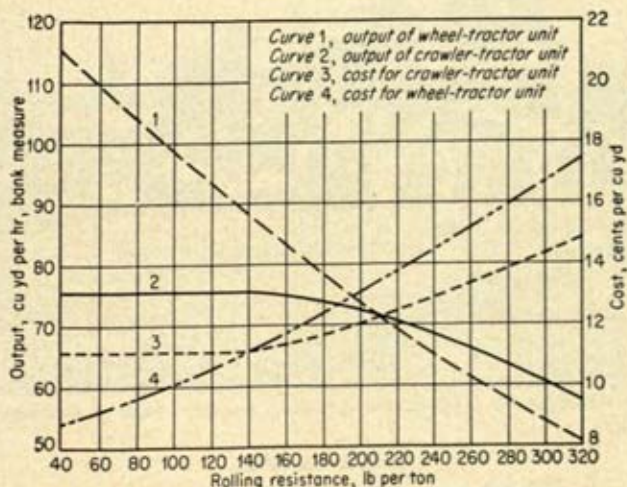


FIG. 5-14. The effect of rolling resistance on the output of tractor-pulled scrapers.

illustrated by assuming that 500,000 cu yd of sandy clay earth is to be hauled for a fill. The haul distance will be 800 ft, average one way, over a level road. It will be necessary to haul 650 cu yd per hr in order to satisfy the completion-date requirements. The initial rolling resistance is 160 lb per ton. The rolling resistance can be reduced to 60 lb per ton



by providing reasonable maintenance. Is the cost of the reduction justified if the earth will be hauled with wheel-type tractor-pulled scrapers? The information necessary to answer this question can be obtained from Fig. 5-14.

For rolling resistance of 160 lb per ton  
 Output per unit per hr = 83 cu yd  
 No. units required,  $650 \div 83 = 8$   
 Cost per cu yd = \$0.118  
 Total cost of job,  $500,000 \times \$0.118 = \$59,000$   
 For rolling resistance of 60 lb per ton  
 Output per unit per hr = 110 cu yd  
 No. units required,  $650 \div 110 = 6$   
 Cost per cu yd = \$0.093  
 Total cost of job,  $500,000 \times \$0.093 = \$46,500$

Thus, improving the haul road will reduce the number of hauling units from 8 to 6, and it will reduce the cost of hauling by \$12,500. If the reduced rolling resistance can be maintained for less than the saving in hauling cost, it is good economy to do so.



FIG. 5-15. Earth loader loading tractor-pulled wagons. (Euclid Division, General Motors Corp.)

#### EARTH LOADER

**General Information.** The term earth loader refers to a machine manufactured by the Euclid Road Machinery Company. The machine consists of a frame mounted on crawler tracks, a cutting blade, moldboard plow, conveyor chute, conveyor belt, and an engine to operate the belt. It is pulled by one or two crawler tractors, depending on the class of material loaded. The cutting blade has an over-all width of 9 ft 6½ in. The maximum cutting depth is 48 in. The best loading results usually are

obtained when cutting a cross section of approximately 4 sq ft. Where mixing of the layers of material is required, a special side-cutting blade may be installed to permit cuts up to 10 to 12 ft deep.

The loader is used on large projects to load large hauling units, such as trucks and tractor-pulled bottom-dump wagons. The machine works best in long, relatively level borrow pits or cuts, such as are frequently found on highway construction. The machine can make a 180° turn in approximately 37 ft.

As the loader is pulled along, the plow and cutting blade loosen the earth, which is forced through the throat onto the conveyor belt and carried to the upper end of the conveyor chute, where it is deposited into the hauling units.

**Output of a Loader.** The output of a loader will vary with the class of material, the length of cut, the size of hauling units, the power of the pulling tractor, and climatic conditions.

Although a loader will handle any material from soft loam through hard clay, the output will be reduced when it is loading harder materials.

Where conditions permit, borrow pits or cuts should be several hundred feet long in order to reduce the time lost in making turns at the ends of the runs.

The capacities of the hauling units should be large in order to reduce the frequency of stopping the loader while one unit moves out and another moves under the belt. Units having capacities of 15 cu yd or more are satisfactory. During actual loading a machine may move at a speed of 1.5 mph while cutting a section 4 ft square. This is equivalent to  $1.5 \times 5,280 \times 4 \div 27 = 1,170$  cu yd per hr. With an operating factor of only 50 per cent the output will be 585 cu yd per hr. The time required to load a 15 cu yd truck, with a net load of 12 cu yd, will be  $\frac{12 \times 60}{1,170} = 0.62$  min.

**The Cost of Operating a Loader.** The cost of operating a loader will vary with the class of material handled. The delivered cost of a machine is about \$38,000, depending on the distance from the factory. The hourly operating cost, under various conditions, should be about as follows:

Delivered cost	= \$38,000
Less cost of belt	= 2,500
Amount to be depreciated	= \$35,500
Life, 7 yr at 2,000 hr per yr	
Annual costs will be	
Depreciation, $\$35,500 \div 7$	= \$ 5,071
Investment, $\$38,000 \times 0.571 \times 0.10$	= 2,170
Total annual cost	= \$ 7,241
Cost per hr, $\$7,241 \div 2,000$	= \$ 3.62



The operating costs per hr will be

	Conditions		
	Adverse	Average	Favorable
Belt.....	\$ 5.00	\$ 2.50	\$ 1.67
Points, plow cutting blade.....	3.39	1.35	0.85
Repairs.....	1.63	1.10	0.87
Fuel.....	0.98	0.70	0.42
Oil, grease.....	0.20	0.20	0.20
Ownership.....	3.62	3.62	3.62
Total cost per hr.....	\$14.82	\$ 9.47	\$ 7.63
Add hourly cost of a pull tractor and operator.....	8.50	8.50	8.50
Total cost of loader and tractor.....	\$23.32	\$17.97	\$16.13
Loader production, cu yd per hr.....	300	500	700
Loading cost per cu yd.....	\$ 0.078	\$ 0.036	\$ 0.023

This machine not only gives low loading costs, but, because of the high rate of loading, the time spent by hauling units under the loader will be less than when they are loaded by most other types of excavating equipment. As a result, the production rate of hauling units will be higher, and the hauling cost will be lower.

### PLACING AND COMPACTING EARTH FILLS

**General Information.** When earth is placed in a fill, it is usually necessary to spread, wet, shape, and compact it in accordance with well-established engineering practices. The contractor should select balanced equipment in order that all operations may be synchronized as nearly as possible. Each operation should have enough equipment to keep it in balance with the other operations. An analysis of the project will indicate the number of units of each type of equipment needed. For example, if the volume of earth placed requires 10,000 gal of water per hour, enough water trucks should be provided to furnish this quantity of water. The job planner should start with the rate of delivery of earth to the fill, then, from this information, determine the number of units of each type required to keep the job going smoothly.

**Spreading the Earth for a Fill.** As the earth is placed for a fill, it is necessary to spread it in uniformly thick layers and to maintain a reasonably level surface. Although a bulldozer may be used for this operation, better performance, especially for the final spreading, can be obtained with a motor grader. Graders may be specified by the horsepower of the engine, the length of the blade, and the number of driving axles, single for one axle and tandem for two axles. Some graders are equipped with

adjustable scarifying teeth ahead of the blade to loosen the earth prior to blading it.

The blade may be set at any desired depth, and it may be rotated to any desired position, to push the earth straight ahead or to one side.

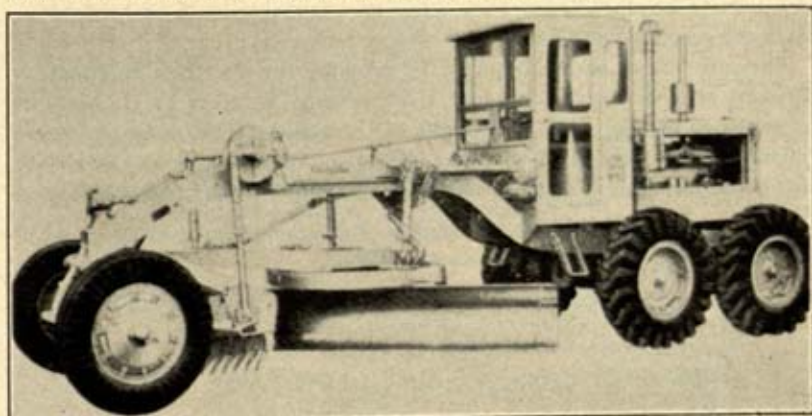


FIG. 5-16. Tandem-drive motor grader. (Caterpillar Tractor Co.)

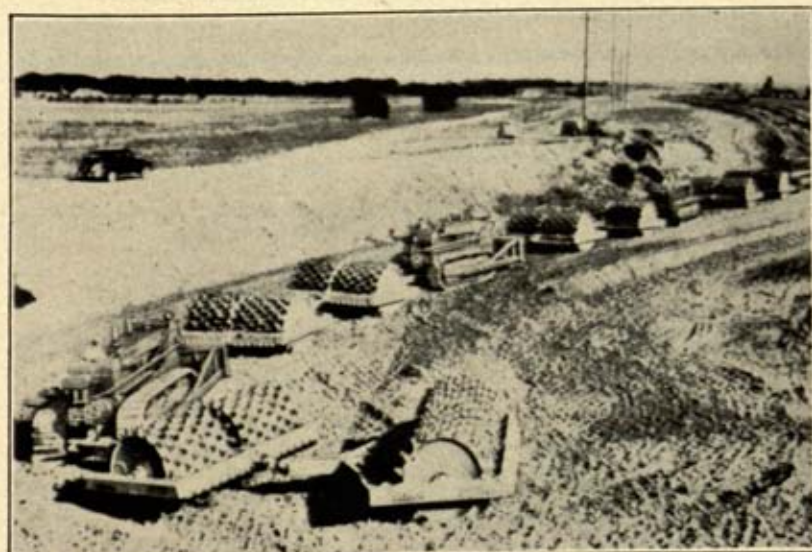


FIG. 5-17. Tractor-pulled sheep's-foot rollers. (International Harvester Co.)

Graders may have three to six forward gears, with speeds varying from approximately 2 to 20 mph. The lower speeds are used for grader operations, while the higher speeds are used for traveling.

**Compacting the Earth for a Fill.** In order to increase the density and



strength of earth for a fill, it is necessary to tamp, or compact, it. If the moisture content is too low, water should be added to facilitate compaction. Several types of equipment are available for compacting earth, which include sheep's-foot, grid, and smooth-wheel rollers and pneumatic tires. Compaction around obstructions may be obtained with hand-held pneumatic or gasoline-engine-operated compactors.

The number of passes required to produce the specified compaction will vary with the class of material, the thickness of the layers, the amount of moisture present, and the weight of the roller. The weight of most rollers can be varied to fit the job conditions. In compacting earth with sheep's-foot rollers, it may be necessary to make 6 to 15 passes per layer.

**EXAMPLE.** A project requires the placing of approximately 1,000,000 cu yd of earth for a dam. The job conditions are as follows:

Class of earth, sandy clay, weight, 2,400 lb per cu yd bm, swell 25%  
 Initial moisture content, 7%, by weight  
 Required moisture content, 12%, by weight  
 Average haul distance, 4,200 ft, at a 2% favorable grade  
 Rolling resistance of haul road, 50 lb per ton  
 Maximum thickness of layers deposited, 6 in. compacted measure  
 Estimated number of passes with sheep's-foot roller, 12 per layer  
 Average distance to water, 1 mile

The earth will be excavated with a Euclid loader, whose output is estimated to be 600 cu yd per hr bank measure. Hauling will be done with 15-cu-yd tractor-pulled bottom-dump wagons whose net load will be 12 cu yd bank measure.

All production rates will be based on a 50-min hr.

The analysis should be made as follows:

#### 1. Hauling the earth

Average speed, loaded, 22 mph

Average speed, empty, 26 mph

Round-trip time per unit

Fixed time, loading, spreading, turning, etc. = 2.50 min

Hauling,  $\frac{4,200}{22 \times 88}$  = 2.17 min

Returning,  $\frac{4,200}{26 \times 88}$  = 1.84 min

Total time = 6.51 min

No. trips per hr,  $50 \div 6.51 = 7.68$

Output per hr per unit,  $7.68 \text{ trips} \times 12 \text{ cu yd} = 92 \text{ cu yd bm}$

No. units required,  $600 \div 92 = 6.52$

Use 7 units, plus 1 unit for a stand-by

#### 2. Spreading the earth

The area covered per hour will be  $600 \times 27 \times 2 = 32,400 \text{ sq ft}$

Use a 100-hp motor grader with a 12-ft blade

Average speed, including turns, etc., 2.0 mph

Effective width of blade, 9 ft

Estimated number of passes per layer, 4

Area covered per hr, 1 pass,  $2.0 \times 5,280 \times 9 \times \frac{5}{8} = 79,300 \text{ sq ft}$

Area covered per hr, 4 passes,  $79,300 \div 4 = 19,825$  sq ft

No. units required,  $32,400 \div 19,825 = 2$

### 3. Wetting the fill

Weight of earth placed per hr,  $600 \text{ cu yd} \times 2,400 \text{ lb} = 1,440,000 \text{ lb}$

Water to be added by weight,  $12 - 7 = 5\%$

Weight of water required per hr,  $1,440,000 \times 0.05 = 72,000 \text{ lb}$

Quantity of water required per hr,  $72,000 \div 8.33 = 8,650 \text{ gal}$

Use water trucks with a capacity of 2,000 gal

The pump will deliver 500 gpm

The round-trip time per truck will be

Filling,  $2,000 \text{ gal} \div 500 \text{ gpm} = 4.00 \text{ min}$

Hauling water,  $1 \text{ mile} \times 60 \div 15 \text{ mph} = 4.00 \text{ min}$

Returning to pump,  $1 \text{ mile} \times 60 \div 20 \text{ mph} = 3.00 \text{ min}$

Sprinkling,  $2,000 \text{ gal} \div 300 \text{ gpm} = 6.67 \text{ min}$

Lost time per trip = 3.33 min

Total time = 21.00 min

Trips per 50-min hr,  $50 \div 21 = 2.38$

Quantity hauled per hr per truck,  $2.38 \text{ trips} \times 2,000 \text{ gal} = 4,760 \text{ gal}$

No. trucks required,  $8,650 \div 4,760 = 2$

### 4. Compacting the fill

Use sheep's-foot rollers, with 4-ft-wide drums, pulled with crawler tractors

Average speed, including turns, 3.2 mph

Area covered per hr per drum for 1 pass,  $4 \times 3.2 \times 5,280 \times \frac{5}{100} = 56,400 \text{ sq ft}$

Area covered per hr for 12 passes,  $56,400 \div 12 = 4,700 \text{ sq ft}$

No. drums required,  $32,400 \text{ sq ft} \div 4,700 \text{ sq ft} = 7$

Depending on the size of the tractors, these rollers may be pulled by two or three tractors

## PROBLEMS

5-1. A crawler tractor and a self-loading scraper, whose specifications are given below, will be used to haul earth.

Weight of tractor, 25,500 lb

Gear	Speed		Drawbar pull at sea level, lb
	Mph	Fpm	
1st	1.4	123	21,351
2d	2.2	194	13,431
3d	3.2	282	9,090
4th	4.6	405	5,994
5th	6.0	528	4,550

Capacity of scraper, 10.8 cu yd loose measure

Empty weight of scraper, 13,750 lb

The earth is sandy clay, weighing 100 lb per cu ft bank measure, with a swell of 25 per cent.

Job conditions are as follows:

Rolling resistance for tractor, 130 lb per ton

Rolling resistance for scraper, 90 lb per ton



Distance traveled to load scraper in low gear, 100 ft  
 Length of earth ramp out of pit, 200 ft, up a 5% grade  
 Distance from pit to dump, 600 ft, up a 3% grade  
 Distance from dump to pit, 600 ft, down a 3% grade  
 Length of ramp into pit, 150 ft, down a 6% grade  
 Total time spent at dump, 1 min  
 Working time, 45 min per hr

Determine the probable output of the tractor and scraper, in cubic yards per hour bank measure.

**5-2.** When a tractor loads a scraper moving down a slope, the effect of the slope is the same as increasing the drawbar pull of the tractor, whereas loading up a slope has the effect of decreasing the drawbar pull of the tractor. If the tractor and scraper of Prob. 5-1 load on a -4 per cent slope, determine the equivalent increase in drawbar pull resulting from the slope when the scraper is loaded. The loose earth will weigh 2,160 lb per cu yd.

**5-3.** Determine the output in cubic yards per hour bank measure for a self-loading scraper pulled by a crawler tractor for each of the given-haul distances, measured one way. The fixed time per cycle will be 2.5 min. The average travel speed between the borrow pit and the fill will be 4.5 mph. The heaped capacity of the scraper will be 13.6 cu yd loose measure. The soil will be ordinary earth.

The haul distances will be 300, 500, 800, and 1,000 ft.

If the cost of the tractor, scraper, and operator and a proportionate cost of the bulldozer that helps load the scraper are \$10.80 per hour, determine the cost per cubic yard bank measure for hauling earth each of the given distances.

**5-4.** Compare the cost of hauling earth using a crawler-tractor-pulled scraper and a four-wheel-tractor-pulled scraper for one-way haul distances of 400, 800, 1,000, 1,500, and 2,000 ft. Assume ordinary earth.

Use a crawler tractor with 90 drawbar hp and a scraper with a heaped capacity of 12 cu yd.

Use a two-wheel tractor with 180 belt hp and a scraper with a heaped capacity of 13 cu yd (see Table 5-2 for the speeds of this tractor).

Make any necessary assumptions regarding the fixed time, travel speed, operator's wages, operating factor, use of a bulldozer to help load, etc. Obtain the costs of equipment from Appendix A.

Prepare your results in tabular form, showing for each type of equipment the haul distance, output in cubic yards bank measure, and cost per cubic yard.

**5-5.** The fill for a dam, requiring 400,000 cu yd of earth bank measure, must be hauled an average distance of 1,500 ft. The earth is sandy clay. Two-wheel tractors, with 180-hp engines, will be used with scrapers having a heaped capacity of 13 cu yd. The fixed time per cycle will be  $2\frac{3}{4}$  min. Assume a 45-min hour.

Determine the cost per cubic yard, and for the entire project, based on hauling the earth over (a) a level road with a rolling resistance of 70 lb per ton; (b) a level road with a rolling resistance of 150 lb per ton.

See Table 5-2 for the speeds and rimpulls of the tractors and Appendix A for equipment costs. Bulldozers will be required to help the tractors load the scrapers. Use local wage rates for the operators.

**5-6.** For the example given on page 118, change the output of the loader to 450 cu yd per hr bank measure. Using other equipment and conditions given in the example, determine the number of units of each type of equipment required to haul, spread, wet, and compact the earth.

## CHAPTER 6

### EXCAVATING EQUIPMENT

#### POWER SHOVELS

**General Information.** Power shovels are used primarily to excavate earth and load it into trucks or tractor-pulled wagons or onto conveyor belts. They are capable of excavating all classes of earth, except solid rock, without prior loosening. They may be mounted on crawler tracks, in which case they are referred to as crawler-mounted. Such shovels

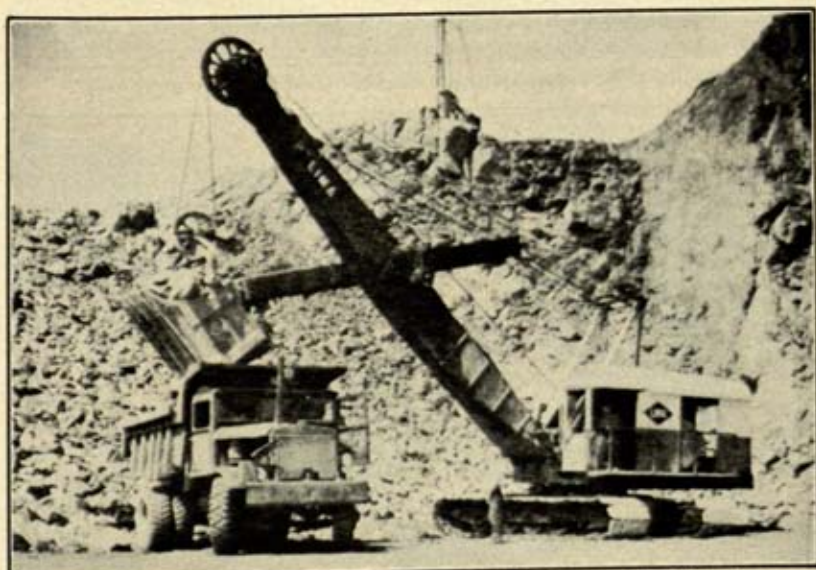


FIG. 6-1. Crawler-mounted power shovel. (*Lima Locomotive Works.*)

have very low travel speeds, but the wide treads give low soil pressures, which permit them to operate on soft ground. They may be mounted on rubber-tired wheels. Single-engine self-propelled units are powered and operated from the excavator cab. The non-self-propelled units mounted on the rear of trucks, which are referred to as truck-mounted, have separate engines for operating them. Rubber-tire-mounted shovels, which





FIG. 6-2. Wheel-mounted power shovel. (*Link-Belt Speeder Corp.*)



FIG. 6-3. Truck-mounted power shovel. (*The Thew Shovel Co.*)

have higher travel speeds than the crawler-mounted units, are useful for small jobs where considerable traveling is necessary and where the road surfaces and ground are firm. Figure 6-1 illustrates a crawler-mounted shovel, Fig. 6-2 a wheel-mounted shovel, and Fig. 6-3 a truck-mounted shovel.

**The Size of a Power Shovel.** The size of a power shovel is indicated by the size of the dipper, expressed in cubic yards. In measuring the size of the dipper the earth is struck even with the contour of the dipper. This is referred to as the struck volume, as distinguished from the heaped volume which a dipper may pick up in loose soil. Owing to the swelling of a soil when it is loosened, the bank-measure volume of a dipper will be less than the loose volume. It is possible that a dipper may be heaped sufficiently to give a bank-measure volume equal to the rated size of the dipper. However, this condition will not occur except for easy digging soils, under favorable conditions, and the assumption should not be made unless field tests indicate it to be correct. If a 2-cu-yd dipper, excavating a soil whose swell is 25 per cent, is able to fill the dipper to its struck volume only, the bank-measure volume will be  $2 \div 1.25 = 1.6$  cu yd.

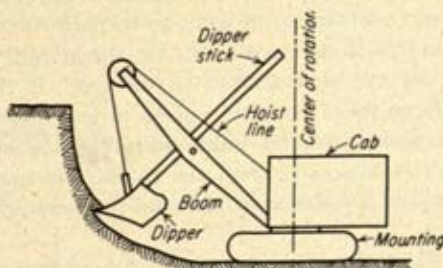


FIG. 6-4. Basic parts and operation of a power shovel. (Power Crane and Shovel Association.)

Power shovels are commonly available in the following sizes:  $\frac{3}{8}$ ,  $\frac{1}{2}$ ,  $\frac{3}{4}$ , 1,  $1\frac{1}{4}$ ,  $1\frac{1}{2}$ , 2, and  $2\frac{1}{2}$  cu yd, which are classified by the Power Crane and Shovel Association as commercial sizes. Larger sizes may be available, or they can be manufactured on special order.

**The Basic Parts and Operation of a Shovel.** The basic parts of a power shovel include the mounting, cab, boom, dipper stick, dipper, and hoist line. These parts are illustrated in Fig. 6-4.

With a shovel in the correct position, near the face of the earth to be excavated, the dipper is lowered to the floor of the pit, with the teeth pointing into the face. A crowding force is applied through the shipper shaft, and at the same time tension is applied to the hoisting line to pull the dipper up the face of the pit. If the depth of the face is just right, considering the type of soil and the size of the dipper, the dipper will be filled as it reaches the top of the face. If the depth of the face, referred to as the depth of cut, is too shallow, it will not be possible to fill the dipper completely without excessive crowding and hoisting tension, and possibly not at all. This subjects the equipment to excessive strains and reduces the output of the unit. If the depth of the face is greater than



is required to fill the dipper, when operating under favorable crowd and hoist, it will be necessary to reduce the depth of penetration of the dipper into the face if the full face is to be excavated or to start the excavation above the floor of the pit. The material left near the floor of the pit will be excavated after the upper portion of the face is removed.

**Optimum Depth of Cut.** The optimum depth of cut is that depth which produces the greatest output and at which the dipper comes up with a full load without undue crowding. The depth varies with the class of soil and the size of the dipper. Values of optimum depths for various classes of soils and sizes of dippers are given in Table 6-2.

**Selecting the Type and Size Power Shovel.** One of the problems which confronts the purchaser of a power shovel is the selection of the type and size. Several factors will affect the selection.

In selecting the type of shovel the prospective purchaser should consider the probable concentration of work to be performed. If there will be numerous small jobs in different locations, the mobility of the rubber-tire-mounted shovel will be a distinct advantage. If the work will be concentrated in large jobs, mobility will be of less importance and the crawler-mounted shovel will be more desirable. A crawler-mounted shovel usually is less expensive than the rubber-tire-mounted unit and can operate on ground surfaces which are not firm enough to support the latter type unit.

In selecting the size of a shovel, the two primary factors which should be considered are the cost per cubic yard of material excavated and the job conditions under which the shovel will operate.

In estimating the cost per cubic yard the following factors should be considered:

1. The size of the job, as a larger job may justify the higher cost of a large shovel.
2. The cost of transporting a large shovel will be higher than for a small one.
3. The depreciation rate for a large shovel may be higher than for a small one, especially if it is to be sold at the end of a job, owing to the probable greater difficulty of selling a large shovel.
4. The cost of down time for repairs for a large shovel may be considerably greater than for a small one, owing to increased delays in obtaining parts for a large shovel, especially if the parts must be manufactured to order.
5. The combined cost of drilling, blasting, and excavating rock for a large shovel may be less than for a small shovel, as a large machine will handle bigger rocks than a small one. This may permit a saving in the cost of drilling and blasting.

6. The cost of wages per cubic yard will be less for a large shovel than for a small one.

The following job conditions should be considered in selecting the size of a shovel:

1. High lifts to deposit earth from a basement or trench into trucks at natural ground level will require the long reach of a large shovel.

2. If blasted rock is to be excavated, the large-size dipper will handle bigger rocks.

3. If the material to be excavated is hard and tough, the dipper of the large shovel, which exerts higher digging pressures, will handle the material more easily.

4. If the time allotted for the completion of a project requires a high hourly output, a large shovel must be used.

5. The size of available hauling units should be considered in selecting the size of a shovel. If small hauling units must be used, the size of the

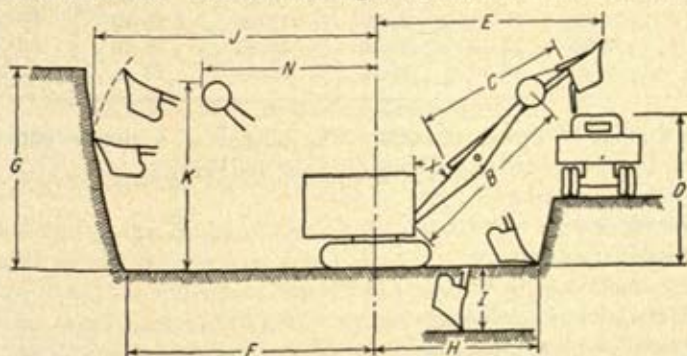


FIG. 6-5. Clearance diagram for a power shovel. (Power Crane and Shovel Association.)

shovel should be small, whereas, if large hauling units are available, a large shovel should be used.

6. The weight limitations imposed by most states for hauling on highways may restrict the size of a shovel if it is to be hauled over state highways. Also, the clearance of bridges and underpasses may restrict the size.

**Shovel Dimensions and Clearances.** In considering the size of a power shovel for a project it may be desirable to know the dimensions of the boom and the dipper stick and the maximum cutting height, digging radius, dumping radius, and dumping height. Figure 6-5 is a clearance diagram for a shovel. Table 6-1 gives representative dimensions and clearances for power shovels. The clearances are for a boom angle of  $45^\circ$ . For boom angles other than  $45^\circ$  the clearances will be more or less than the values given in Table 6-1. Manufacturer's specifications should be



consulted for exact values of the clearances. The maximum dumping height and radius are especially important when a shovel in a pit is loading trucks at natural ground level.

TABLE 6-1. DIMENSIONS AND CLEARANCES FOR POWER SHOVELS, FOR 45-DEG BOOM ANGLE

Size dipper, cu yd	Length std boom, ft	Length std handle, ft	Maximum cutting height, ft	Maximum digging radius, ft	Maximum dumping height, ft	Maximum dumping radius, ft
$\frac{3}{8}$	13-15	11-13	17-19	22	13-15	18-20
$\frac{1}{2}$	15-16	12-13	19-24	21-24	14-16	19-20
$\frac{3}{4}$	17-18	13-15	21-27	25-28	15-17	22-24
1	20	16	23-27	31-32	15-18	23-25
$1\frac{1}{4}$	21	16	23-27	31-32	16-19	24-27
$1\frac{1}{2}$	21-23	16-18	24-29	32-33	18-20	28-30
$1\frac{3}{4}$	22-24	16-18	26-30	32-33	18-20	28-30
2	22-25	17-19	26-30	33-36	19-20	30-33
$2\frac{1}{2}$	25-26	18-19	28-35	35-38	19-21	32-34

**The Output of Power Shovels.** The output of a power shovel is affected by numerous factors, including the following:

1. Class of material
2. Depth of cut
3. Angle of swing
4. Job conditions
5. Management conditions
6. Size of hauling units
7. Skill of the operator
8. Physical condition of the shovel

The output of a shovel should be expressed in cubic yards per hour based on bank-measure volume. The capacity of a dipper is based on its struck volume. In excavating some classes of materials it is possible for a dipper to pick up a heaping volume which may exceed the struck volume. In order to obtain the bank-measure volume of a dipper of earth, the average loose volume should be divided by 1 plus the swell, expressed as a fraction. For example, if a 2-cu-yd dipper, excavating material whose swell is 25 per cent, will handle an average loose volume of 2.25 cu yd, the bank-measure volume will be  $2.25 \div 1.25 = 1.8$  cu yd. If this shovel can make 2.5 cycles per min, which includes no allowance for lost time, the output will be  $2.5 \times 1.8 = 4.5$  cu yd per min, or 270 cu yd per hr. This is an ideal output, which will seldom, if ever, be experienced on a project. Table 6-2 gives the ideal outputs of power shovels, expressed in cubic yards bank measure, for various classes of materials,

based on digging at optimum depth with a 90° swing and no delays. In the table the upper figure is the optimum depth in feet, and the lower figure is the ideal output in cubic yards.

TABLE 6-2. IDEAL OUTPUTS OF POWER SHOVELS, IN CUBIC YARDS PER HOUR, BANK MEASURE\*

Class of material	Size shovel, cu yd								
	3/8	1/2	3/4	1	1 1/4	1 1/2	1 3/4	2	2 1/2
Moist loam or light sandy clay	3.8 85	4.6 115	5.3 165	6.0 205	6.5 250	7.0 285	7.4 320	7.8 355	8.4 405
Sand and gravel	3.8 80	4.6 110	5.3 155	6.0 200	6.5 230	7.0 270	7.4 300	7.8 330	8.4 390
Good common earth	4.5 70	5.7 95	6.8 135	7.8 175	8.5 210	9.2 240	9.7 270	10.2 300	11.2 350
Hard, tough clay	6.0 50	7.0 75	8.0 110	9.0 145	9.8 180	10.7 210	11.5 235	12.2 265	13.3 310
Well-blasted rock	40	60	95	125	155	180	205	230	275
Wet, sticky clay	6.0 25	7.0 40	8.0 70	9.0 95	9.8 120	10.7 145	11.5 165	12.2 185	13.3 230
Poorly blasted rock	15	25	50	75	95	115	140	160	195

\* Courtesy Power Crane and Shovel Association.

**The Effect of the Depth of Cut on the Output of a Power Shovel.** If the depth of the face from which a shovel is excavating material is too shallow, it will be difficult or impossible to fill the dipper in one pass up the face. The operator will have a choice of making more than one pass to fill the dipper, which will increase the time per cycle, or he may carry a partly filled dipper to the hauling unit each cycle. In either case the effect will be to reduce the output of the shovel.

If the depth of the face is greater than the minimum required to fill the dipper, with favorable crowding and hoisting forces, the operator may do one of three things. He may reduce the depth of penetration of the dipper into the face in order to fill the dipper in one full stroke. This will increase the time for a cycle. He may start digging above the base of the face, then remove the lower portion of the face later. Or he may run the dipper up the full height of the face and let the excess earth spill down to the bottom of the face, to be picked up later. The choice of any one of the procedures will result in some lost time, based on the time required to fill the dipper when it is digging at optimum depth. As indicated in Table 6-2, the optimum depth varies with the class of material and the size of the dipper.

The effect of the depth of cut on the output of a shovel is illustrated in Table 6-3. In the table the per cent of optimum depth of cut is obtained



by dividing the actual depth of cut by the optimum depth for the given material and dipper, then multiplying the result by 100. Thus, if the actual depth of cut is 6 ft and the optimum depth is 10 ft, the per cent of optimum depth of cut is  $\frac{6}{10} \times 100 = 60$ .

#### The Effect of the Angle of Swing on the Output of a Power Shovel.

The angle of swing of a power shovel is the horizontal angle, expressed in degrees, between the position of the dipper when it is excavating and the position when it is discharging the load. The total time in a cycle includes digging, swinging to the dumping position, dumping, and returning to the digging position. If the angle of swing is increased, the time for a cycle will be increased, while if the angle of swing is decreased, the time for a cycle will be decreased. The effect of the angle of swing on the output of a shovel is illustrated in Table 6-3. For example, if a shovel which is digging at optimum depth, has the angle of swing reduced from 90 to 60°, the output will be increased by 16 per cent.

The output of a shovel operating at 90° swing and optimum depth, which is obtained from Table 6-2, should be multiplied by the proper conversion factor from Table 6-3 in order to obtain the probable output for any given depth and angle of swing.

**EXAMPLE.** The use of the tables is illustrated by considering a 2-cu-yd shovel excavating common earth, with a depth of cut of 12 ft and an angle of swing of 60°.

The per cent of optimum depth is  $\frac{12}{10.2} \times 100 = 118$ .

By interpolating in Table 6-3 the factor is found to be 1.115. However, it is doubtful that values beyond two decimal places are significant. Therefore, 1.11 is sufficiently accurate for practical purposes. The probable output of the shovel will be  $300 \times 1.11 = 333$  cu yd per 60-min hour, provided there are no other factors which affect the output.

TABLE 6-3. CONVERSION FACTORS FOR DEPTH OF CUT AND ANGLE OF SWING FOR A POWER SHOVEL\*

Per cent of optimum depth	Angle of swing, deg						
	45	60	75	90	120	150	180
40	0.93	0.89	0.85	0.80	0.72	0.65	0.59
60	1.10	1.03	0.96	0.91	0.81	0.73	0.66
80	1.22	1.12	1.04	0.98	0.86	0.77	0.69
100	1.26	1.16	1.07	1.00	0.88	0.79	0.71
120	1.20	1.11	1.03	0.97	0.86	0.77	0.70
140	1.12	1.04	0.97	0.91	0.81	0.73	0.66
160	1.03	0.96	0.90	0.85	0.75	0.67	0.62

\* Courtesy Power Crane and Shovel Association.

Although the information given in Tables 6-2 and 6-3 is based on extensive field studies, the reader is cautioned against using it too literally

without adjusting it for conditions which will probably exist on a project. As explained later, additional factors must be applied to whatever extent they are necessary in the judgment of the project planner.

**The Effect of Job Conditions on the Output of a Power Shovel.** As every owner of a power shovel knows, no two excavating jobs are alike. There are certain conditions at every job over which the owner of the shovel has no control. These conditions must be considered in estimating the probable output of a shovel.

A shovel may operate in a large, open pit, with a firm, well-drained floor, where trucks can be spotted on either side of the shovel to eliminate lost time waiting for hauling units. The terrain of the natural ground may be uniformly level, so that the depth of cut will always be optimum. The haul road is not affected by climatic conditions, such as rains. The job is large enough to justify the selection of balanced hauling units. A project of this type might be classified as having excellent job conditions.

Another shovel may be used to excavate material for a highway cut through a hill. The depth of cut may vary from zero to considerably more than the optimum depth. The sides of the cut must be carefully sloped. The cut may be so narrow that a loaded truck must move out before an empty truck can back into loading position. As the truck must be spotted behind the shovel, the angle of swing will approximate 180°. The floor of the cut may be muddy, which will delay the movement of the trucks. Light rains may delay operations for several days. A project of this type might be classified as having poor job conditions.

In excavating a basement, which requires the trucks to travel up an earth ramp, a power shovel may be delayed considerably by ground water or rain, by the difficulty of getting hauling units in and out, and by the difficulty of excavating the corners.

Job conditions may be classified as excellent, good, fair, and poor. There is no uniform standard which may be used as a guide in classifying a job. Each job planner must use his own judgment and experience in deciding which condition best represents his job. Table 6-4 illustrates the effect of job conditions on the output of a power shovel.

**The Effect of Management Conditions on the Output of a Power Shovel.** The attitude of the owner of a shovel in establishing the conditions under which a shovel is operated will affect the output of the shovel. While the owner may not be able to improve job conditions, he may take several steps to improve management conditions, including the following:

1. Greasing and lubricating the shovel frequently
2. Checking the shovel parts that are subject to the greatest wear, and replacing worn parts while the shovel is not being operated, as at the end of a shift
3. Replacing badly worn wire rope between shifts



4. Replacing dull dipper teeth with sharp ones, as required
  5. Giving the shovel a major overhaul between jobs, if necessary
  6. Keeping at the job extra parts that are subject to the greatest wear
  7. Keeping the pit floor clean and smooth to permit better truck spotting and to reduce the angle of swing
  8. Providing adequate trucks of the correct size to eliminate lost time in loading and waiting for trucks
  9. Paying a bonus to the crew for production in excess of an agreed amount to encourage high production
  10. Providing a competent supervisor to keep the job running smoothly
- Management conditions may be classified as excellent, good, fair, and poor. Table 6-4 illustrates the effect of management conditions on the output of a power shovel.

TABLE 6-4. FACTORS FOR JOB AND MANAGEMENT CONDITIONS\*

Job conditions	Management conditions			
	Excellent	Good	Fair	Poor
Excellent.....	0.84	0.81	0.76	0.70
Good.....	0.78	0.75	0.71	0.65
Fair.....	0.72	0.69	0.65	0.60
Poor.....	0.63	0.61	0.57	0.52

\* Courtesy Power Crane and Shovel Association and Frank A. Nikirk.

**Examples Illustrating the Effect of the Various Factors on the Output of a Power Shovel.** It is doubtful that a job planner will be able to select exactly the correct factors to be used in estimating the output of a shovel. As a result, the actual output of a shovel may vary from the estimated output. Experience and good judgment are essential to the selection of the correct factors. If the output is found to fall below that estimated, it may be possible to increase it by modifying the operating conditions.

**EXAMPLE.** To illustrate the use of the information in Tables 6-2 to 6-4, consider a 1-cu-yd power shovel for excavating hard clay with a depth of cut of 7.5 ft. An analysis of the project indicates an average angle of swing of 75°, job conditions will be fair, and management will be good. Determine the probable output in cubic yards per hour bank measure.

From Table 6-2 the ideal output will be 145 cu yd per hr. The optimum depth is 9 ft.

$$\text{The per cent of optimum depth, } \frac{7.5}{9} \times 100 = 83.3$$

From Table 6-3 the depth-swing factor is 1.04

From Table 6-4 the job-management factor is 0.69

$$\text{The probable output per hr, } 145 \times 1.04 \times 0.69 = 104 \text{ cu yd}$$

**EXAMPLE.** This example illustrates the effect of the various conditions on the output of a shovel. Determine the probable output, in cubic yards per hour bank measure, for a 1-cu-yd shovel for each of the given conditions.

	Class of material				
	Moist loam	Common earth	Hard clay	Wet clay	Poorly blasted rock
Depth, ft.....	6.0	10.0	8.0	12.0	Varies
Angle of swing, deg.....	60	90	120	180	120
Job conditions.....	Good	Fair	Fair	Poor	Fair
Management conditions.....	Good	Good	Fair	Poor	Good
Ideal output, cu yd per hr.....	200	175	145	95	75
Optimum depth, ft.....	6.0	7.8	9.0	9.0	
Per cent optimum depth.....	100	128	89	133	
Depth-swing factor.....	1.16	0.94	0.87	0.67	
Job-management factor.....	0.75	0.69	0.65	0.52	0.69
Probable output, cu yd per hr....	174	114	82	33	52

**Methods of Increasing the Output of a Power Shovel.** A problem which frequently confronts a consultant on the selection and operation of excavating equipment is to analyze a project which is not being operated satisfactorily in order to recommend corrective steps to increase the output and reduce the cost of handling the material.

**EXAMPLE.** On one such project, where the cost was exceeding the estimate, an analysis was made to determine methods of reducing the cost of excavating and hauling the earth. The material was common earth. The analysis of the operations revealed the following information:

Size of power shovel,  $1\frac{1}{2}$  cu yd  
 Depth of cut, 12 ft  
 Angle of swing,  $120^\circ$   
 Size of trucks, 6 cu yd bm  
 Round-trip time for a truck, 19 min  
 No. trucks, 8

The time spent by the shovel in cleaning up the floor of the pit, moving, and under-going repairs reduced the actual excavating time to about 30 min per hr.

The floor of the pit was rough, muddy, and heavily rutted because of inadequate drainage, which reduced the efficiency of the hauling units.

The output averaged 108 cu yd per hr.

The direct cost of excavating and hauling the earth was determined as follows:

Shovel, operator, and oiler	= \$14.75 per hr
Truck and drivers, 8 @ \$5.40	= 43.20 per hr
Direct overhead and supervision	= 5.60 per hr
Total cost	= \$63.55 per hr
Cost per cu yd, $\$63.55 \div 108$	= 0.59



The analysis indicated that the output could be increased by taking the following steps:

1. Use a small bulldozer to keep the floor of the pit clean and well drained.
2. Reduce the depth of cut to the optimum.
3. Reduce the angle of swing to 75° by improving the floor of the pit.
4. Improve the job conditions to good by proper maintenance of the pit and haul roads and by excavating at optimum depth.
5. Improve the management conditions to good by properly servicing the equipment at the end of the shifts and by paying a bonus of \$0.04 per cu yd, to be divided among the workers, for all production in excess of 120 cu yd per hr.
6. Reduce the round-trip time of the trucks to 15 min by improving the haul road and the pit floor.
7. Provide additional trucks to haul the increased output of the shovel.

If the recommended steps are taken, the probable output of the shovel will be as follows:

Est. actual excavating time, 45 min per hr  
 Ideal output, 240 cu yd per hr  
 Depth-swing factor, 1.07  
 Job-management factor, 0.75  
 Probable output,  $240 \times 1.07 \times 0.75 = 193$  cu yd per hr

The number of trucks required to haul the earth will be as follows:

Assume trucks operate 50 min per hr  
 No. trips per hr per truck,  $\frac{60}{15} = 3.33$   
 Volume hauled per hr per truck,  $3.33 \times 6 = 20$  cu yd  
 No. trucks needed,  $193 \div 20 = 9.6$   
 \*Use 10 trucks

The revised direct cost of excavating and hauling the earth will be as follows:

Shovel, operator, and oiler	= \$14.75 per hr
Trucks and drivers, 10 @ \$5.40	= 54.00 per hr
Direct overhead and supervision	= 5.60 per hr
Cost of bulldozer and operator	= 4.20 per hr
Cost of bonus, 73 cu yd @ \$0.04	= 2.92 per hr
Total cost	= \$81.47 per hr
Cost per cu yd, $\$81.47 \div 193$	= 0.42
Net saving in cost per cu yd	= 0.17

On a project requiring the handling of 100,000 cu yd of earth the direct cost would be reduced from \$59,000 to \$42,000, which is a net saving of \$17,000. This saving is sufficiently large to demonstrate the financial effect of applying intelligent engineering in the selection of equipment and in analyzing an operation. The failure to apply engineering analysis to the operation of a project is one reason why a contractor may complete a project with a loss, while another will complete a similar project with a profit.

**Motion and Time Study Applied to a Power Shovel.** A motion and time study is a method of analyzing the actual performance of equipment on a project. It is made to determine the time required for each operation or factor that affects the output of the equipment. The engineer who makes the motion and time study will need a stop watch and a clip

board with suitable note paper. Each study should be given a title for identification purposes. The study should be sufficiently comprehensive to give information that is representative of actual conditions over a period of time. For example, a study that is made for a period of 1 hr, when equipment is operating under favorable conditions, or when it is operating under adverse conditions, will not give representative results for the entire project. The study must be continued long enough to permit all factors which affect the output of the equipment to produce their correct effects. Such a study may be of considerable value in analyzing the operations of a project as a means of improving the output or reducing the cost of production.

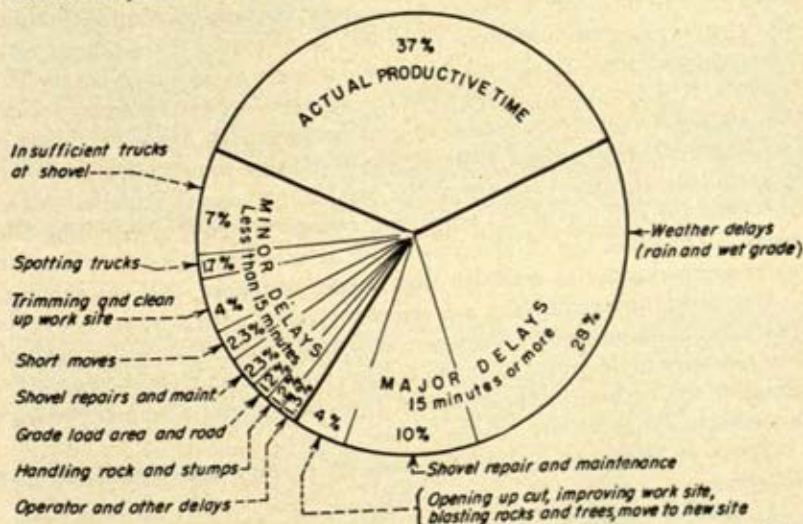


FIG. 6-6. Representative power-shovel productive time. (Power Crane and Shovel Association.)

Figure 6-6 illustrates the results of a number of motion and time studies made by the Highway Research Board of the National Research Council. The studies involved 16 power shovels on 10 highway projects during the period 1947 to 1949. The information given on the time chart is the average for the 10 projects. The actual productive time, amounting to 37 per cent of the total, seems very low and is not representative of conditions that will exist on a heavy engineering project, which is not subject to some of the delays shown in the chart. Operations in a dry climate or in a rock quarry would not be likely to be subject to such severe weather delays. Thus, this chart should not be used as a guide in estimating the probable output of a power shovel. It is simply indicative of the information that may be obtained from a motion and time study.



## DRAGLINES

**General Information.** Draglines are used to excavate earth and load it into hauling units, such as trucks or tractor-pulled wagons, or to deposit it in levees, dams, and spoil banks near the pits from which it is excavated. In general, a power shovel up to a capacity of  $2\frac{1}{2}$  cu yd can be converted into a dragline by replacing the boom of the shovel with a crane boom and substituting a dragline bucket for the shovel dipper.

For some projects either a power shovel or a dragline may be used to excavate materials, but for others the dragline will have a distinct advantage compared with a shovel. A dragline usually does not have to go into a pit or hole in order to excavate. It may operate on natural ground while excavating material from a pit with its bucket. This will be very advantageous when earth is removed from a ditch, canal, or pit containing water. If the earth is hauled with trucks, they do not have to go into the pit and contend with mud. If the earth can be deposited along a canal or ditch or near a pit, it frequently is possible to use a dragline with a boom long enough to dispose of the earth in one operation, eliminating the need for hauling units, which will reduce the cost of handling the earth. Draglines are excellent units for excavating trenches when the sides are permitted to establish their angles of repose, without shoring.

One disadvantage in using a dragline compared with a power shovel is the reduced output of the dragline. A comparison of the ideal output of various sizes of draglines with the output of power shovels shows that a dragline will excavate approximately 75 to 80 per cent as much earth as a shovel of the same size.

**Types of Draglines.** Draglines may be divided into four types, as follows:

1. Crawler-mounted (see Fig. 6-7)
2. Wheel mounted, self-propelled (see Fig. 6-8)
3. Truck-mounted (see Fig. 6-9)
4. Walking (see Fig. 6-17)

Crawler-mounted draglines can operate on surfaces which are too soft for wheel- or truck-mounted equipment, but their travel speeds are so slow, frequently less than 1 mph, that it may be necessary to use auxiliary hauling equipment to transport them from one job to another, especially if the distance is great. Wheel- and truck-mounted units may have travel speeds in excess of 30 mph.

Walking draglines will be discussed in later articles.

**The Size of a Dragline.** The size of a dragline is indicated by the size of the bucket, expressed in cubic yards, which, in general, is the same size as the dipper of the power shovel into which it may be converted. However, most draglines may handle more than one size bucket, depending on

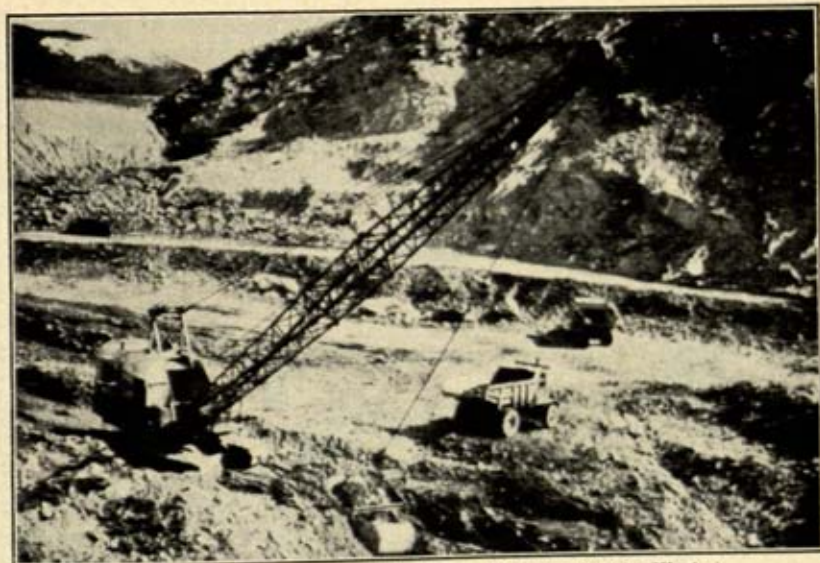


FIG. 6-7. Crawler-mounted dragline. (*Lima Locomotive Works.*)



FIG. 6-8. Wheel-mounted dragline. (*The Thew Shovel Co.*)



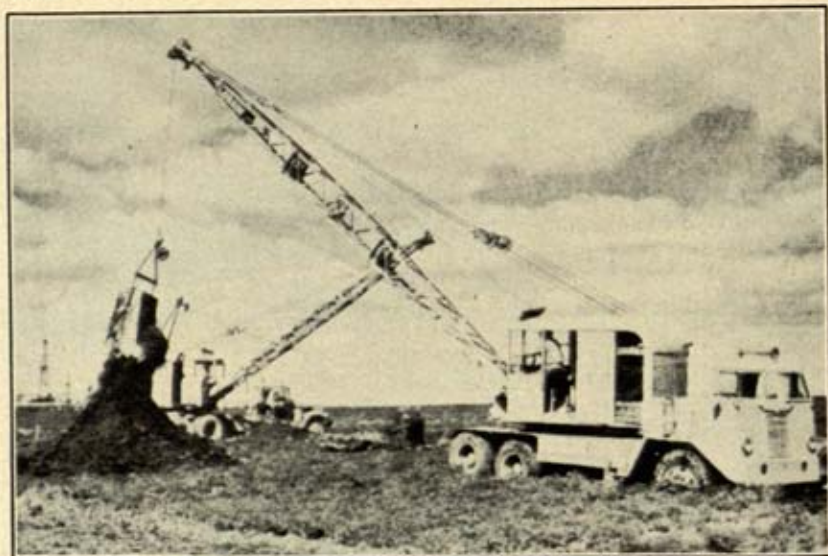


FIG. 6-9. Truck-mounted dragline. (*The Thew Shovel Co.*)

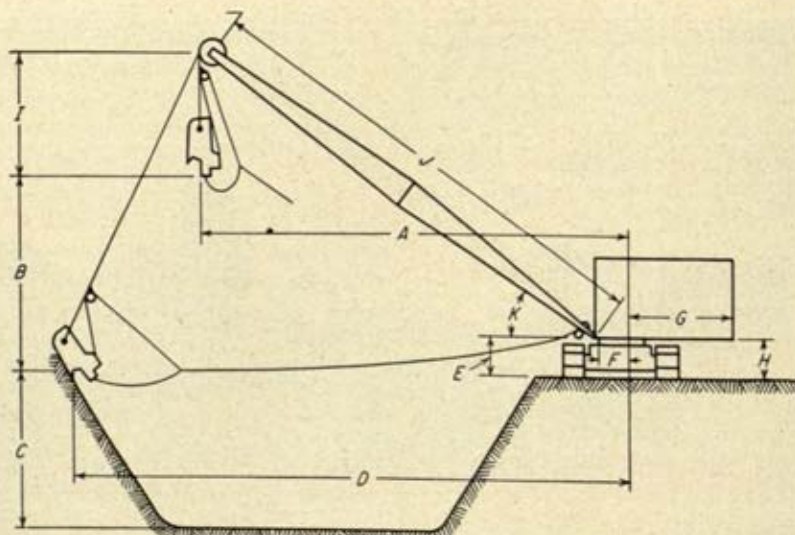


FIG. 6-10. Dragline-range diagram. (*Power Crane and Shovel Association.*)

the length of the boom and the class of material excavated. As the maximum lifting capacity of a dragline is limited by the force which will tilt the machine over, it is necessary to reduce the size of the bucket when a long boom is used or when the material has a high specific gravity. In practice the combined weight of the bucket and its load should produce a tilting force not greater than 75 per cent of the force required to tilt the machine over. A longer boom, with a smaller bucket, should be used when it is necessary to increase the digging reach or the dumping radius.

If the material is difficult to excavate, the use of a smaller bucket, which will reduce the digging resistance, may permit an increase in the output of a dragline.

Typical working ranges, for a dragline that will handle buckets varying in sizes from  $1\frac{1}{4}$  to  $2\frac{1}{2}$  cu yd, are given in Table 6-5 (see Fig. 6-10 for the dimensions given in the table).

TABLE 6-5. TYPICAL WORKING RANGES FOR A DRAGLINE WITH MAXIMUM COUNTERWEIGHTS

<i>J</i> , boom length, 50 ft:						
Capacity, lb*.....	12,000	12,000	12,000	12,000	12,000	12,000
<i>K</i> , boom angle, deg. ....	20	25	30	35	40	45
<i>A</i> , dumping radius, ft. ...	55	50	50	45	45	40
<i>B</i> , dumping height, ft. ...	10	14	18	22	24	27
<i>C</i> , max digging depth, ft	40	36	32	28	24	20
<i>J</i> , boom length, 60 ft:						
Capacity, lb*.....	10,500	11,000	11,800	12,000	12,000	12,000
<i>K</i> , boom angle, deg. ....	20	25	30	35	40	45
<i>A</i> , dumping radius, ft. ...	65	60	55	55	52	50
<i>B</i> , dumping height, ft. ...	13	18	22	26	31	35
<i>C</i> , max digging depth, ft	40	36	32	28	24	20
<i>J</i> , boom length, 70 ft:						
Capacity, lb*.....	8,000	8,500	9,200	10,000	11,000	11,800
<i>K</i> , boom angle, deg. ....	20	25	30	35	40	45
<i>A</i> , dumping radius, ft. ...	75	73	70	65	60	55
<i>B</i> , dumping height, ft. ...	18	23	28	32	37	42
<i>C</i> , max digging depth, ft	40	36	32	28	24	20
<i>J</i> , boom length, 80 ft:						
Capacity, lb*.....	6,000	6,700	7,200	7,900	8,600	9,800
<i>K</i> , boom angle, deg. ....	20	25	30	35	40	45
<i>A</i> , dumping radius, ft. ...	86	81	79	75	70	65
<i>B</i> , dumping height, ft. ...	22	27	33	39	42	47
<i>C</i> , max digging depth, ft	40	36	32	28	24	20
<i>D</i> , digging reach.....	Depends on working conditions and operator's skill with bucket)					

\* Combined weight of bucket and material must not exceed capacity.

**The Basic Parts and Operation of a Dragline.** The basic parts of a dragline are illustrated in Fig. 6-11.



Excavating is started by swinging the empty bucket to the digging position, at the same time slacking off the drag and the hoist cables. Separate drums on the basic unit are available for each of these cables so that they may be coordinated into a smooth operation. Excavating is accomplished by pulling the bucket toward the machine while regulating the digging depth by means of the tension maintained in the hoist cable. When the bucket is filled, the operator takes in on the hoist line while playing out the drag cable. The bucket is so constructed that it will not dump its contents until it is desired. Hoisting, swinging, and dumping of the loaded bucket follow in that order; then the cycle is repeated. Dumping is accomplished by releasing the drag cable. An experienced operator can cast the excavated material beyond the end of the boom.

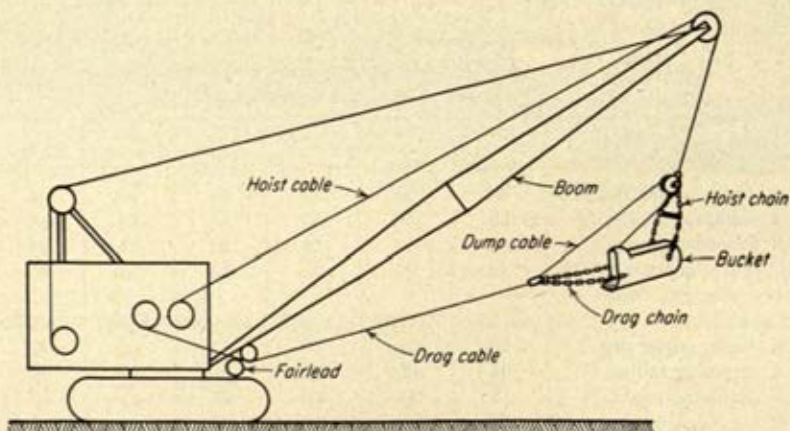


FIG. 6-11. Basic parts of a dragline. (Power Crane and Shovel Association.)

Since it is more difficult to control the accuracy in dumping from a dragline as compared with a power shovel, it is desirable to use larger hauling units for dragline loading in order to reduce the spillage. A size ratio equal to at least five to six times the capacity of the bucket is recommended.

Figure 6-12 shows the dragline digging zones. The work should be planned to permit most of the digging to be done in the zones which permit the best digging, with the poor digging zone used as little as possible.

**Optimum Depth of Cut.** A dragline will produce its greatest output if the job is planned to permit the earth to be excavated at the optimum depth where possible. Table 6-6 gives the optimum depth of cut for various sizes of buckets and classes of materials, using short-boom draglines.

**The Output of Draglines.** The output of a dragline will vary with the following factors:

1. Class of material
2. Depth of cut
3. Angle of swing
4. Size and type of bucket
5. Length of boom
6. Job conditions

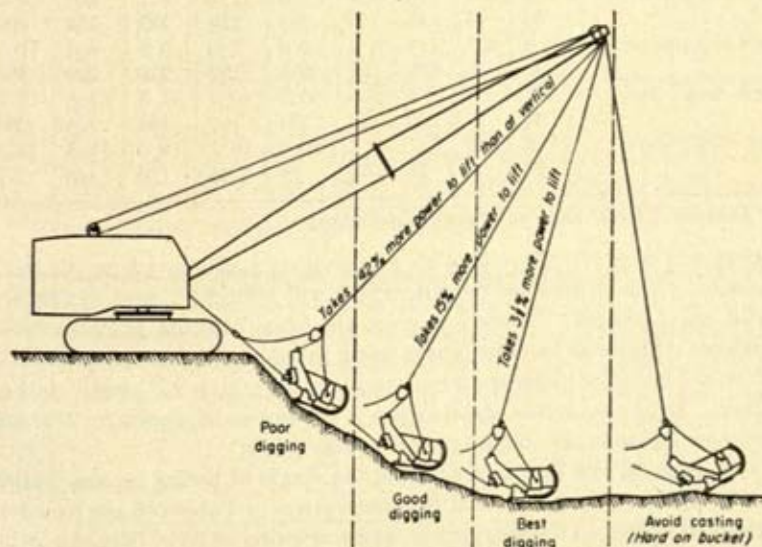


FIG. 6-12. Dragline digging zones. (Power Crane and Shovel Association.)

7. Management conditions
8. Method of disposal, casting or loading trucks
9. Size of hauling units, if used
10. Skill of the operator
11. Physical condition of the machine

The output of a dragline should be expressed in cubic yards per hour bank measure. This quantity may be obtained from field observations, or it may be estimated by multiplying the average loose volume per bucket by the number of cycles per hour and dividing by 1 plus the swell factor for the earth, expressed as a fraction. For example, if a 2-cu-yd bucket, excavating material whose swell is 25 per cent, will handle an average loose volume of 2.4 cu yd, the bank-measure volume will be  $2.4 \div 1.25 = 1.92$  cu yd. If the dragline can make 2 cycles per min, the



digging clay, soft shale, or loose gravel. Heavy-duty buckets are used for mine stripping, handling blasted rock, and excavating hardpan and highly abrasive materials. Buckets are sometimes perforated to permit excess water to drain from the loads. Figure 6-13 shows a medium-duty dragline bucket. Figure 6-14 shows a 1-cu-yd dragline bucket dumping its load on a spoil bank.

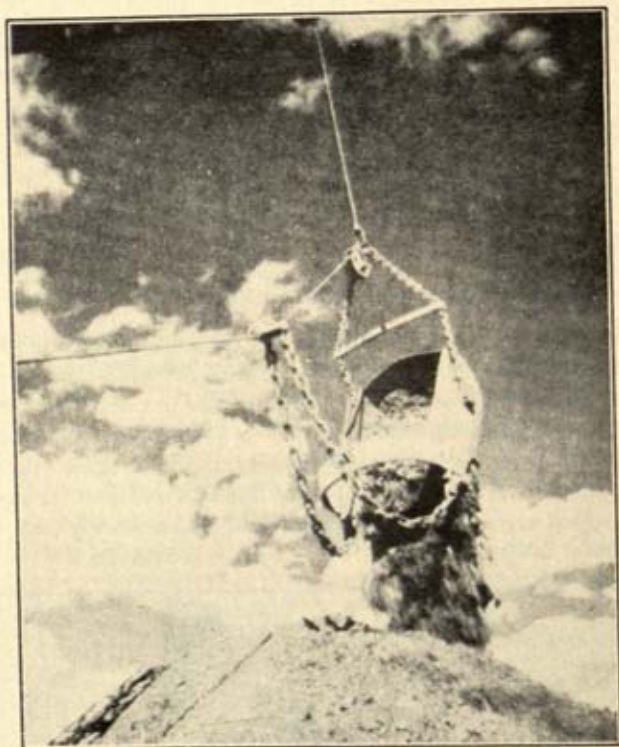


FIG. 6-14. Dragline bucket dumping its load. (Page Engineering Co.)

Table 6-8 gives representative capacities, weights, and dimensions for dragline buckets.

The normal size of a dragline bucket is based on its struck capacity, which is expressed more accurately in cubic feet. In selecting the most suitable size bucket for use with a given dragline, it is desirable to know the weight of the loosened material to be handled, expressed in pounds per cubic foot. While it is desirable to use the largest size bucket possible in the interest of increasing the output, care should be exercised to see that the combined weight of the load and the bucket does not exceed the safe load recommended for the dragline.

TABLE 6-8. REPRESENTATIVE CAPACITIES, WEIGHTS, AND DIMENSIONS OF DRAGLINE BUCKETS

Size, cu yd	Struck capacity, cu ft	Weight of bucket, lb			Dimensions, in.		
		Light- duty	Medium- duty	Heavy- duty	Length	Width	Height
$\frac{3}{8}$	11	760	880	.....	35	28	20
$\frac{1}{2}$	17	1,275	1,460	2,100	40	36	23
$\frac{3}{4}$	24	1,640	1,850	2,875	45	41	25
1	32	2,220	2,945	3,700	48	45	27
$1\frac{1}{4}$	39	2,410	3,300	4,260	49	45	31
$1\frac{1}{2}$	47	3,010	3,750	4,525	53	48	32
$1\frac{3}{4}$	53	3,375	4,030	4,800	54	48	36
2	60	3,925	4,825	5,400	54	51	38
$2\frac{1}{4}$	67	4,100	5,350	6,250	56	53	39
$2\frac{1}{2}$	74	4,310	5,675	6,540	61	53	40
$2\frac{3}{4}$	90	4,950	6,225	7,390	63	55	41
3	90	5,560	6,660	7,920	65	55	43

EXAMPLE. The importance of this analysis is illustrated by referring to the information given in Table 6-5. Assume that the material to be handled has a loose weight of 90 lb per cu ft. The use of a 2-cu-yd medium-duty bucket will be considered. If the dragline is to be operated with an 80-ft boom at a 40° angle, the maximum safe total load will be 8,600 lb. The approximate weight of the bucket and its load will be

Bucket, from Table 6-8	= 4,825 lb
Earth, 60 cu ft @ 90 lb per cu ft	= 5,400 lb
Combined weight	= 10,225 lb
Max safe load	= 8,600 lb

As this weight will exceed the safe load on the dragline, it will be necessary to use a smaller bucket. Try a  $1\frac{1}{2}$ -cu-yd bucket, whose combined weight will be

Bucket	= 3,750 lb
Earth, 47 cu ft @ 90 lb per cu ft	= 4,230 lb
Combined weight	= 7,980 lb

If a  $1\frac{1}{2}$ -cu-yd bucket is used, it may be filled to heaping capacity, without exceeding the safe load of the dragline.

If a 70-ft boom, whose maximum safe load is 11,000 lb, will provide sufficient working range for excavating and disposing of the earth, a 2-cu-yd bucket may be used and filled to heaping capacity. The reduced cycle time, in using the 70-ft boom, will probably offset the increased time required to fill the 2-cu-yd bucket. The ratio of the output resulting from the use of a 70-ft boom and a 2-cu-yd bucket, compared with a  $1\frac{1}{2}$ -cu-yd bucket, should be approximately as follows:

$$\begin{aligned}\text{Output ratio, } \frac{60 \text{ cu ft}}{47 \text{ cu ft}} \times 100 &= 127\% \\ \text{Increase in output} &= 27\%\end{aligned}$$



This example illustrates the importance of analyzing a job prior to selecting the size excavator to be used. The haphazard selection of equipment can result in a substantial increase in the cost of handling earth.

### The Effect of the Class of Material on the Cost of Excavating Earth.

Figure 6-15 illustrates the effect which the class of material has on the cost per cubic yard bank measure in excavating with draglines. The hourly cost of a machine includes fixed-machine, variable-machine, and

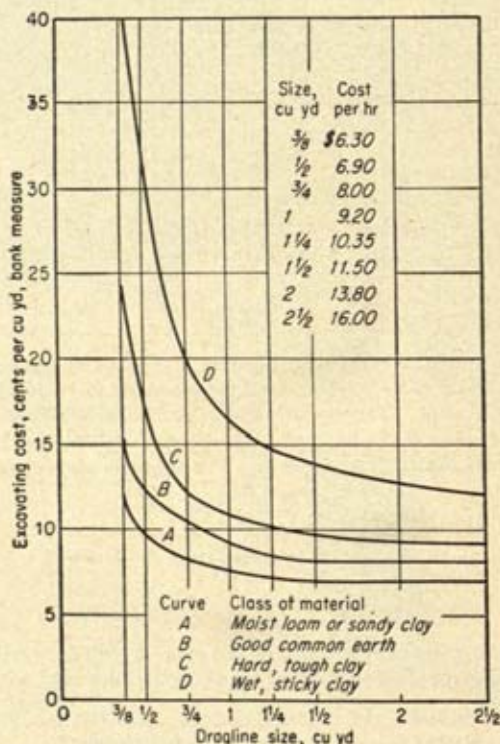


FIG. 6-15. The effect of the class of material and the size of the bucket on the cost of excavating earth with a dragline.

labor costs. Each machine is assumed to operate 2,000 hr per year at 75 per cent efficiency. Thus, the probable hourly output of any given size machine is obtained by multiplying the ideal output, as given in Table 6-6, by 75 per cent. For example, the cost of excavating good common earth using a 1-cu-yd machine is determined as follows:

Operating cost per hr	= \$9.20
Ideal output per hr	= 135 cu yd
Probable output, $0.75 \times 135$	= 101 cu yd
Cost per cu yd, $\$9.20 \div 101$	= \$0.091

**Walking Draglines.** Walking draglines have several advantages compared with crawler types, which include large sizes, long booms, with corresponding large working ranges, simple structural design, simplified maneuverability, and low soil pressure under the bases. Figure 6-16 illustrates a walking dragline stripping overburden from a coal vein to permit open-pit mining with a power shovel.

Because of their exceptionally long reach these machines are particularly suited to the construction of levees and canals, stripping of overburden, and the removal of materials from rivers. Excavation and dis-

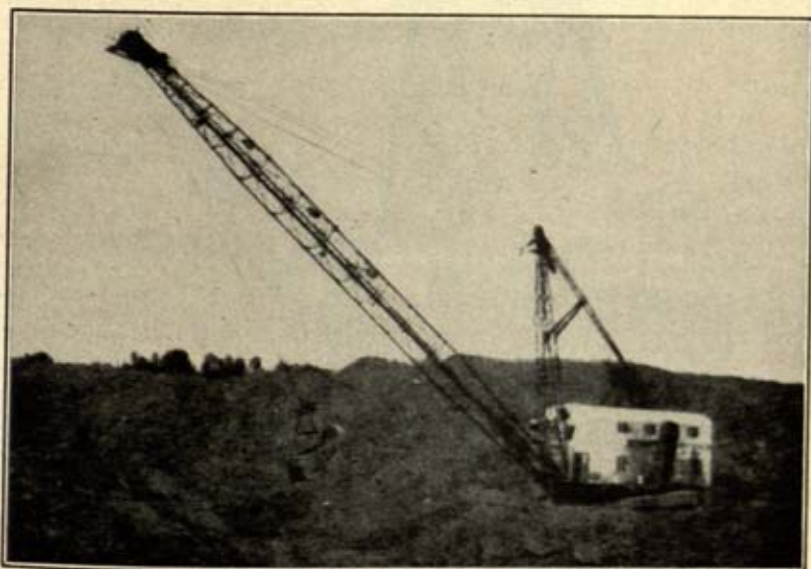


FIG. 6-16. Walking dragline with 135-ft boom and 6-cu-yd bucket. (*Page Engineering Co.*)

posal of the material frequently can be accomplished in one operation, thus eliminating the need of hauling equipment. Boom lengths in excess of 200 ft are sometimes used. The soil pressure under the base of a machine will vary from 4 to 10 psi, which gives it good stability when operating on soft materials.

The walking operation of the machine is relatively simple. When it is desired to move to a new location, the boom is rotated to a position opposite the direction of movement. The two shoes, one on each side of the revolving frame, are lowered by an eccentric cam until most of the weight of the machine is supported by the shoes, and the base is lifted off the ground. As the cam continues to rotate, the machine is dragged backward several feet; then the operation is repeated until the machine



reaches its new location. While the machine is in operation, the shoes are suspended above the ground, as illustrated in Fig. 6-17.

Table 6-9 gives representative working ranges, dimensions, and other information for a walking dragline. The capacities of the buckets are based on handling materials weighing 3,000 lb per cu yd (see Fig. 6-10 for definitions of the terms used in Table 6-9).

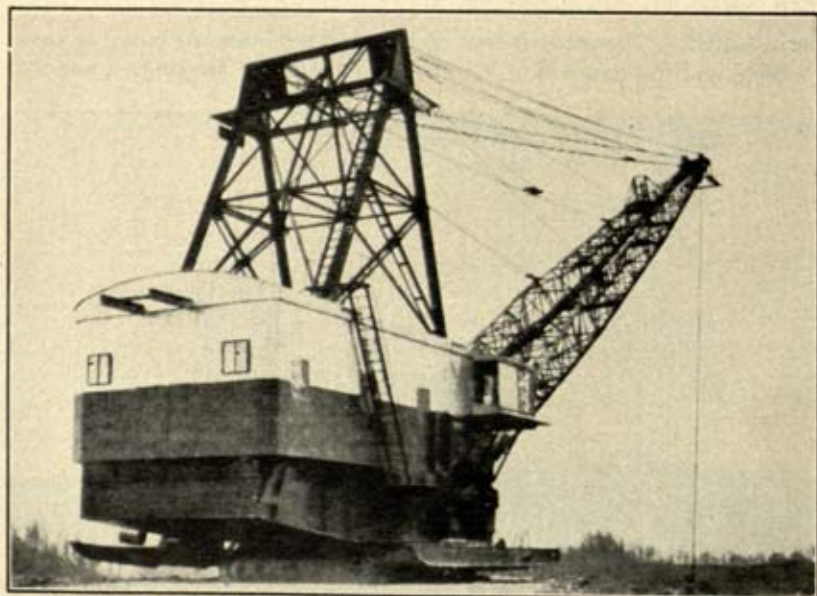


FIG. 6-17. Walking dragline with shoes lifted for walking. (Bucyrus-Erie Co.)

TABLE 6-9. REPRESENTATIVE WORKING RANGES, DIMENSIONS, AND OTHER INFORMATION FOR A WALKING DRAGLINE

Length of boom, ft.....	140			160			180		
Max allowable load, lb.....	36,500			30,000			26,500		
Size bucket, cu yd.....	7			6			5		
Angle of boom, deg	25	30	35	25	30	35	25	30	35
Dumping height, ft	51	61	71	60	73	84	70	84	97
Dumping reach, ft	143	136	129	160	153	146	178	170	162
Digging reach, under boom point, ft	147	140	133	164	157	150	182	174	166
Digging depth, favorable conditions, ft	96	85	74	108	95	82	125	110	95

**EXAMPLE.** This example will illustrate a method of analyzing a project to determine the size dragline required. Select a crawler-mounted dragline to excavate 234,000 cu yd bank measure of common earth in digging a canal. The dimensions of the canal will be:

Bottom width, 20 ft  
 Top width, 44 ft  
 Depth, 12 ft  
 Side slopes, 1:1

The excavated earth will be cast into a levee along one side of the canal, with a berm of at least 20 ft between the toe of the levee and the nearest edge of the canal. The cross section area of the canal will be  $\frac{20 + 44}{2} \times 12 = 384$  sq ft. If the earth swells 25 per cent when it is loosened, the cross-section area of the levee will be

$$384 \times 1.25 = 480 \text{ sq ft.}$$

The dimensions will be:

Height, 12 ft  
 Base width, 64 ft  
 Crest width, 16 ft  
 Side slope, 2:1

The total width from the outside of the levee to the outside of the canal will be:

Width of levee = 64 ft  
 Width of berm = 20 ft  
 Width of canal = 44 ft  
 Total = 128 ft

It will require a dragline with a boom length of 70 ft to furnish the necessary digging and dumping reaches, which will permit adequate dumping height and digging depth, with a boom angle of 30°.

The project must be completed in 1 year. Assume that weather conditions, holidays, and other major losses in time will reduce the operating time to 44 weeks of 40 hr each, or a total of 1,760 working hours. The required output per working hour will be 133 cu yd. It should be possible to operate with a 150° maximum angle of swing. The management factor should be approximately 0.80.

Required output divided by job-management factor,  $133 \div 0.80 = 167$  cu yd per hr  
 Assume a depth-swing factor of 0.81

Required ideal output,  $167 \div 0.81 = 206$  cu yd per hr

Reference to Table 6-6 indicates a 1¾-cu-yd medium-duty bucket. The combined weight of the bucket and load will be:

Weight of load, 53 cu ft @ 80 lb per cu ft = 4,240 lb  
 Weight of bucket = 4,030 lb  
 Total weight = 8,270 lb  
 Max safe load, from Table 6-5 = 9,200 lb

The equipment selected should be checked to verify whether it will produce the required output.

Ideal output, 210 cu yd per hr

Per cent of optimum depth,  $\frac{12.0}{9.5} \times 100 = 126$

Depth-swing factor, 0.82

Job-management factor, 0.80

Probable output,  $210 \times 0.82 \times 0.80 = 138$  cu yd per hr



Thus the equipment should produce the required output, with a slight surplus capacity.

### CLAMSHELLS

**General Information.** Clamshells are used primarily for handling loose materials such as sand, gravel, crushed stone, coal, etc., and for removing materials from inside cofferdams, pier foundations, sewer man-holes, sheet-lined trenches, etc. They are especially suited to vertically lifting materials from one location to another, as in charging hoppers and overhead bins. The limits of vertical movement may be relatively large when they are used with long crane booms.

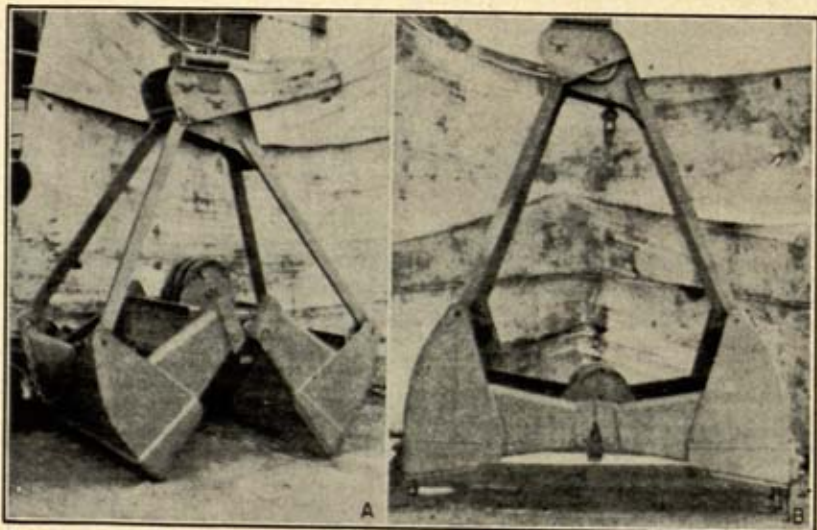


FIG. 6-18. (A) Wide rehandling clamshell bucket. (B) Heavy-duty clamshell bucket. (The C. S. Johnson Co.)

**Clamshell Buckets.** Clamshell buckets are available in various sizes, and in heavy-duty types for digging, medium-weight for general-purpose uses, and lightweight types for rehandling light materials. Manufacturers supply buckets either with teeth that can be removed easily or without teeth. Teeth are used in digging the harder types of materials but are not required when a bucket is used for rehandling purposes. Figure 6-18 illustrates a rehandling and a heavy-duty digging bucket.

The capacity of a clamshell bucket usually is given in cubic yards. A more accurate capacity is given as water-level, plate-line, or heaped-measure, generally expressed in cubic feet. The water-level capacity is the capacity of the bucket if it were hung level and filled with water. The plate-line capacity indicates the capacity of the bucket following a line

along the tops of the clams. The heaped capacity is the capacity of the bucket when it is filled to the maximum angle of repose for the given material. In specifying the heaped capacity the angle of repose usually is assumed to be  $45^{\circ}$ . The deck area indicates the number of square feet covered by the bucket when it is fully open. Table 6-10 gives representative specifications for medium-weight general-purpose-type buckets furnished by one manufacturer.

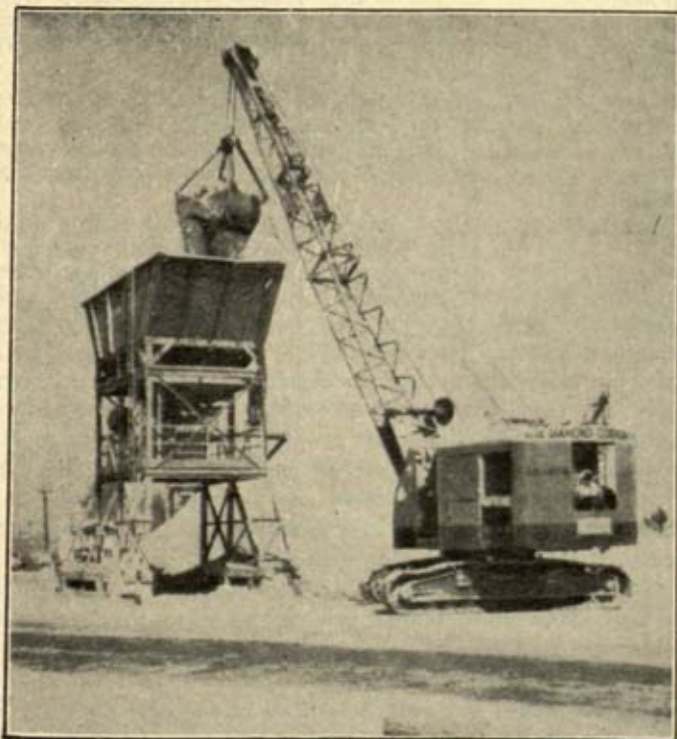


Fig. 6-19. Feeding a batching plant with a clamshell. (*Link-Belt Speeder Corp.*)

**Safe Lifting Capacities of Cranes.** For a clamshell bucket that is handled by a mobile crane, such as a crawler-mounted one, the maximum size bucket will be limited by the lifting capacity of the crane. The rated lifting capacity of a crawler-mounted crane is based on 75 per cent of actual tipping load. When a crawler unit is used primarily as a crane, such as for a dragline, clamshell, or general hoisting, it is equipped with a larger counterweight and longer and wider crawler spacings than for a standard shovel, in order to increase the resistance to tipping.

Figure 6-20 shows typical crane lifting capacities for various shovel



TABLE 6-10. REPRESENTATIVE SPECIFICATIONS FOR MEDIUM-WEIGHT GENERAL-PURPOSE-TYPE CLAMSHELL BUCKETS

	Size, cu yd								
	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{3}{4}$	1	$1\frac{1}{4}$	$1\frac{1}{2}$	$1\frac{3}{4}$	2	$2\frac{1}{2}$
Capacity, cu ft:									
Water-level.....	8.0	11.8	15.6	23.2	27.6	33.0	38.0	47.0	52.0
Plate-line.....	11.0	15.6	21.9	32.2	37.6	43.7	51.5	60.0	75.4
Heaped.....	13.0	18.8	27.7	37.4	45.8	55.0	64.8	74.0	90.2
Weights, lb:									
Bucket only.....	1,662	2,120	2,920	3,870	4,400	5,310	5,440	6,000	7,775
Counterweights.....	230	300	400	400	400	500	500	600	600
Teeth.....	180	180	180	180	180	190	266	300	390
Complete.....	2,072	2,600	3,500	4,450	4,980	6,000	6,206	6,900	8,765
Dimensions:									
Deck area, sq ft.....	13.7	16.0	21.8	24.0	29.0	33.4	36.6	40.0	44.6
Width.....	2'6"	2'6"	3'0"	3'0"	3'5"	3'9"	4'0"	4'3"	4'6"
Length, open.....	5'5"	6'5"	7'3"	7'10"	8'5"	9'0"	9'2"	9'4"	9'11"
Length, closed.....	4'9"	5'7"	6'3"	6'9"	7'1"	7'6"	7'11"	8'0"	9'3"
Height, open.....	7'1"	7'10"	9'1"	9'9"	10'3"	10'9"	10'9"	11'6"	13'0"
Height, closed.....	5'9"	6'4"	7'4"	7'10"	8'3"	8'9"	8'9"	9'3"	10'4"

ratings. The lifting capacities of units made by different manufacturers may vary slightly from the values given in the table. The most commonly used tonnage rating of a crane is based on its safe lifting capacity at a 12-ft radius. Thus, a 1-cu-yd shovel-size unit will safely lift a  $17\frac{1}{2}$ -ton load when the load is suspended 12 ft from the center line of rotation of the unit. Figure 6-20 indicates that the same unit will lift approximately 3 tons at a radius of 40 ft, based on 75 per cent of the tipping load. The figure shows that for a 1-cu-yd unit the recommended maximum safe load, regardless of the distance from the center line of rotation, for a single hoist line is approximately 11,000 lb and that the maximum load for a two-part hoist line is approximately 22,000 lb. These loads are based on using less than the full engine power for hoisting purposes, which will leave surplus engine power for other purposes, such as accelerating and swinging the load.

**Working Ranges of Cranes.** Figure 6-21 shows graphically the height of the boom point above the ground and the radius or reach from the center of the crane for various sizes of cranes and boom lengths and for safe operating angles of boom elevation. All the combinations of boom lengths usually available for crane service are plotted. The lengths of the boom-jib extensions generally available also are plotted on the chart. For example, a  $\frac{3}{4}$ -cu-yd crawler-crane basic boom is shown to be 35 ft long, with extensions available in 5-ft increments up to 60 ft. Truck cranes in the  $\frac{3}{4}$ -cu-yd size have boom lengths of 30 to 90 ft.

If a lift must be made to a net height of 30 ft, plus 10 ft for slings, for a total height to boom point of 40 ft, with a reach of 30 ft, the chart indicates a boom approximately 43 ft long. This will require a standard 45-ft boom, working at an angle of approximately  $55^\circ$  above horizontal.

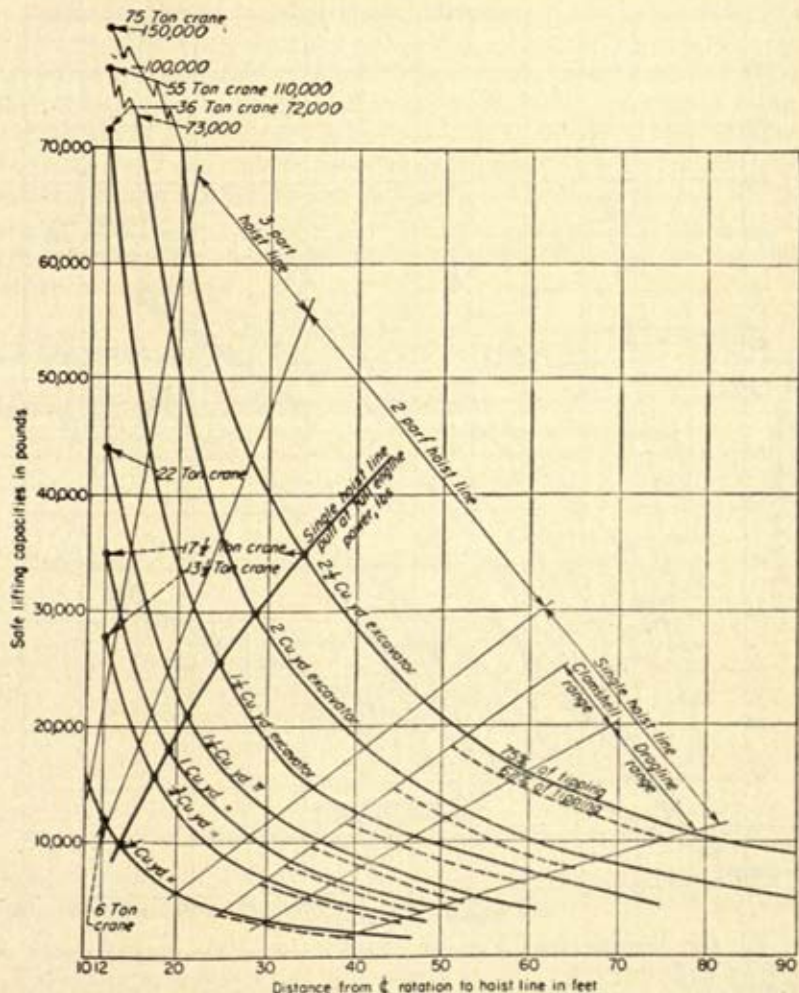


Fig. 6-20. Safe lifting capacities of cranes. (Power Crane and Shovel Association.)

The chart shows the ranges of boom angles customarily used for dragline, clamshell, and lift-crane service. Boom lengths should be kept as short as possible in order to obtain the maximum lifting capacity, to use less power in swinging the unit, and to reduce the time required for a swing cycle.



**EXAMPLE.** Assume you wish to select the largest size clamshell bucket that can be handled with a 1-cu-yd crawler-mounted unit for general-purpose use. The job will require a net lift of 25 ft, with a maximum reach of 30 ft. The material will weigh 90 lb per cu ft loose measure. From Fig. 6-20 the maximum safe lifting capacity of a 1-cu-yd unit is approximately 9,500 lb. If the bucket will operate at plate-line

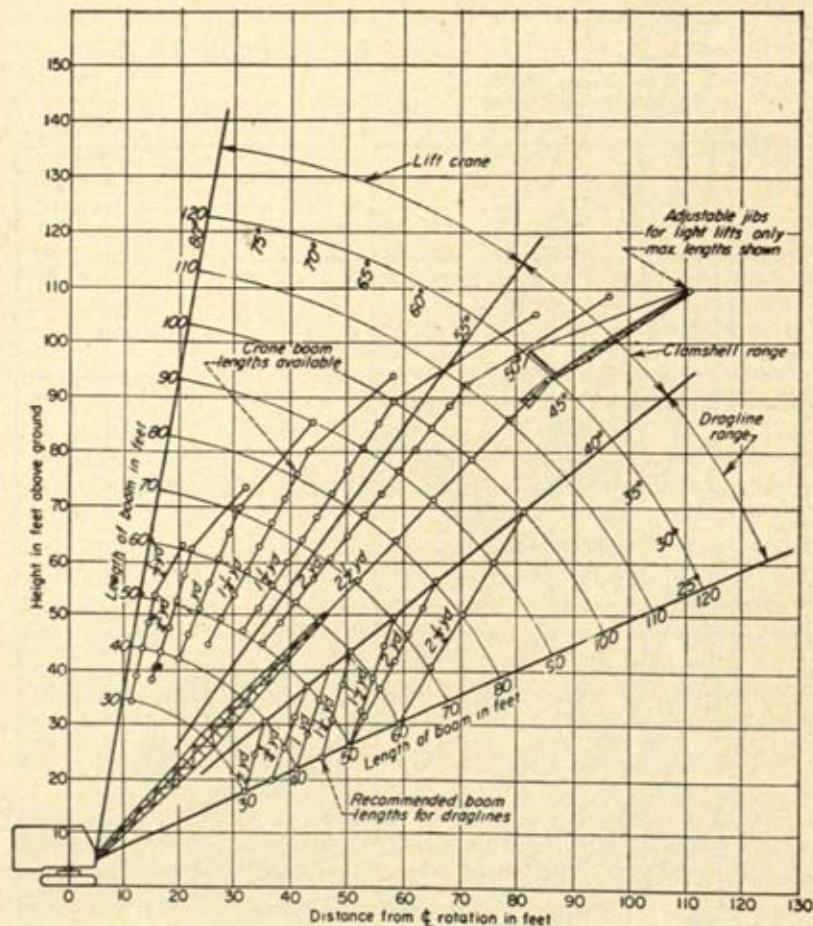


Fig. 6-21. Working ranges of cranes. (Power Crane and Shovel Association.)

capacity, Table 6-10 indicates a  $1\frac{1}{4}$ -cu-yd bucket. The probable combined weight will be:

Load, 37.6 cu ft @ 90 lb per cu ft	= 3,384 lb
Bucket, with teeth and counterweights	= 4,980 lb
Combined weight	= 8,364 lb

If this bucket should be loaded to heaped capacity, the combined weight will be approximately 9,100 lb, which will be safe.

The open height of the bucket is 10 ft 3 in. If 5 ft is added for the length of the hoist line, the total required height of the boom point above the ground will be about 40 ft. Reference to Fig. 6-21 indicates that it will be necessary to provide a boom length of 43 ft. Use a standard 45-ft boom, operating at a boom angle of 55°. For this load a single hoist line is satisfactory.

**Production Rates for Clamshells.** Because of the variable factors which affect the operations of a clamshell it is difficult to give production rates that are dependable. These factors include the difficulty of loading the bucket, the size load obtainable, the height of lift, the angle of swing, the method of disposing of the load, and the experience of the operator. For example, if the material must be discharged into a hopper, the time required to spot the bucket over the hopper and discharge the load will be greater than when the material is discharged onto a large spoil bank. The following example will illustrate a method of estimating the probable output of a clamshell.

**EXAMPLE.** A  $1\frac{1}{2}$ -cu-yd rehandling-type bucket, whose empty weight is 4,300 lb, will be used to transfer sand from a stock pile into a hopper, 25 ft above the ground. The angle of swing will average 90°. The average loose capacity of the bucket will be 48 cu ft.

The specifications for the crane unit give the following information:

Speed of hoist line, 153 fpm  
Swing speed, 4 rpm

The time per cycle should be about as follows:

Loading bucket	= 6 sec
Lifting and swinging load, 25 ft @ 153 fpm	= 10 sec <sup>1</sup>
Dumping load	= 6 sec
Swinging back to stock pile	= 4 sec
Lost time, accelerating, etc.	= 4 sec
Total time	= 30 sec

$$\text{Max no. cycles per hr, } \frac{60 \times 60}{30} = 120$$

$$\text{Max volume per hr, } \frac{120 \times 48}{27} = 213 \text{ cu yd}$$

If the unit operates 45 min per hr, the probable output will be  $\frac{213 \times 45}{60} = 159 \text{ cu yd}$  per hr loose volume.

If the same equipment is used with a general-purpose bucket to dredge muck and sand from a sheet-piling cofferdam partly filled with water, requiring a total vertical lift of 40 ft, and to discharge it into a barge, the production rate previously determined will not apply. It will be necessary to lift the bucket above the top of the dam prior to starting the swing, which will increase the time cycle. Because of the nature

<sup>1</sup> A skilled operator should lift and swing simultaneously. If this is not possible, additional time should be allowed for swinging the load.



of the material the load will probably be limited to the water-filled capacity of the bucket, which is 33 cu ft. The time per cycle should be about as follows:

Loading bucket	= 8 sec
Lifting load, 40 ft @ 153 fpm	= 16 sec
Swinging, 90° @ 4 rpm	= 4 sec
Dumping load	= 4 sec
Swinging back	= 4 sec
Lowering bucket, 40 ft @ 350 fpm	= 7 sec
Lost time, accelerating, etc	= 10 sec
Total time	= 53 sec

$$\text{Max no. cycles per hr, } \frac{60 \times 60}{53} = 68$$

$$\text{Max volume per hr, } \frac{68 \times 33}{27} = 83 \text{ cu yd}$$

If the unit operates 45 min per hr, the probable output will be  $\frac{83 \times 45}{60} = 62 \text{ cu yd}$  per hr loose volume.

### HOES

**General Information.** The term hoe applies to an excavating machine of the power-shovel group. It is referred to by several names, such as hoe, backhoe, back shovel, and pull shovel. As illustrated in Fig. 6-22, a power shovel is converted into a hoe by installing a dipper stick and a dipper at the end of the shovel boom. A hoe frequently is equipped with a goose neck boom to increase the digging depth of the machine.

Hoes are used primarily to excavate below the natural surface of the ground on which the machine rests. They are adapted to excavating trenches, pits for basements, and general grading work, which requires precise control of depths. Because of their rigidity they are superior to draglines in operating on close-range work and dumping into trucks. Because of the direct pull on the dipper, hoes may exert greater tooth pressures than power shovels.

In some respects hoes are superior to wheel- or ladder-type trenching machines, especially in digging utility trenches whose banks are permitted to establish natural slopes and for which trench shoring will not be used. Hoes can remove the earth as it caves in to establish natural slopes, whereas trenching machines cannot do this easily. The reduction in construction costs resulting from the elimination of shoring may be a significant item.

**The Basic Parts and Operation of a Hoe.** The basic parts of a hoe are illustrated in Fig. 6-22. The machine is placed in operation by setting the boom at the desired angle and pulling in on the hoist cable, while releasing the drag cable, to move the dipper out to the desired position. The free end of the boom is lowered by releasing the tension in the hoist

cable until the dipper teeth engage the material to be dug. As the drag cable is pulled in, the dipper is filled. The dipper is lifted by raising the boom, then swinging to the dumping position, which may be over a spoil bank or a truck.

**Working Ranges of Hoes.** Figure 6-23 illustrates the terms which are commonly used to identify the dimensions and working ranges of hoes.

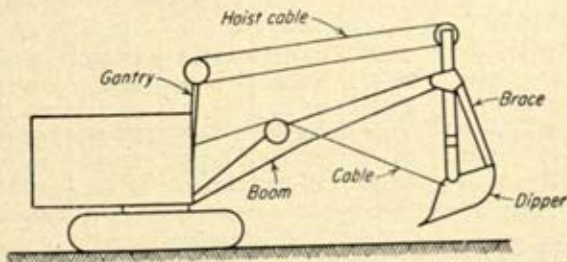


FIG. 6-22. Basic parts of a hoe. (Power Crane and Shovel Association.)

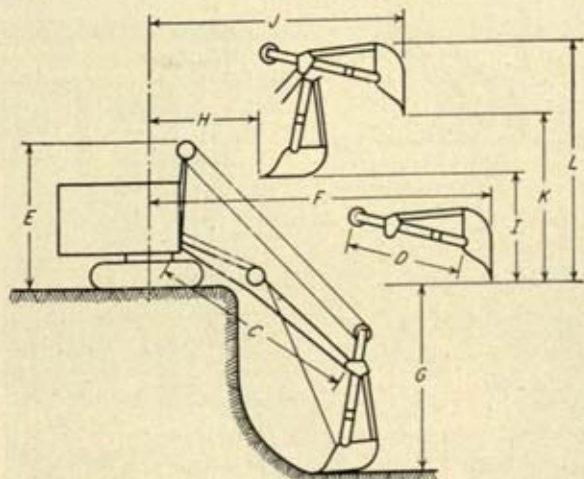


FIG. 6-23. Hoe-clearance diagram. (Power Crane and Shovel Association.)

Table 6-11 gives representative dimensions and clearances for hoes. Dippers are available in various widths to suit the needs of the owner.

**Output of Hoes.** When a hoe is used to dig at moderate depths, the output may approach the output of a power shovel of comparable size digging in the same class of material. However, as the depth is increased, the output of a hoe will decrease considerably. The most effective digging action occurs when the dipper stick is at right angles to the boom. The greatest output will be obtained if digging is done near the machine,



TABLE 6-11. REPRESENTATIVE DIMENSIONS AND CLEARANCES FOR HOES

Size dipper, cu yd	Length of boom, ft	Length of stick, ft	Max digging radius, ft	Max digging depth, ft	Radius, ft, at		Clearance, ft, under dipper	
					Begin- ning of dump	End of dump	Begin- ning of dump	End of dump
$\frac{3}{8}$	14-15	6-8	23-25	11-12	8-10	17-18	9-10	15-17
$\frac{1}{2}$	16-17	6-8	26-27	15-18	8-10	19-22	9-11	15-18
$\frac{3}{4}$	16-20	7-9	28-33	17-22	8-13	20-27	10-12	16-22
1	18-21	8-10	30-34	20-23	9-11	22-26	12-14	18-21
$1\frac{1}{2}$	22-26	9-11	36-42	25-28	13-15	28-32	14-16	27-30
$1\frac{3}{4}$	24-27	9-12	38-43	26-29	14-16	29-33	15-17	28-31



FIG. 6-24. Hoe used to dig a trench. (Link-Belt Speeder Corp.)

because of the reduced cycle time, and because the material rolls back into the dipper better when the dipper is pulled upward near the machine.

## TRENCHING MACHINES

**General Information.** The term trenching machines, as used in this book, applies to the wheel- and ladder-type machines shown in Figs. 6-25 and 6-26, respectively. These machines are satisfactory for digging utility trenches for water-, gas-, and oil-pipe lines, telephone cables, drainage ditches, and sewers where the job and soil conditions are such that they may be used. They provide relatively fast digging, with positive controls of depths and widths of trenches, which reduce expensive hand finishing to a minimum. They are capable of digging any type soil

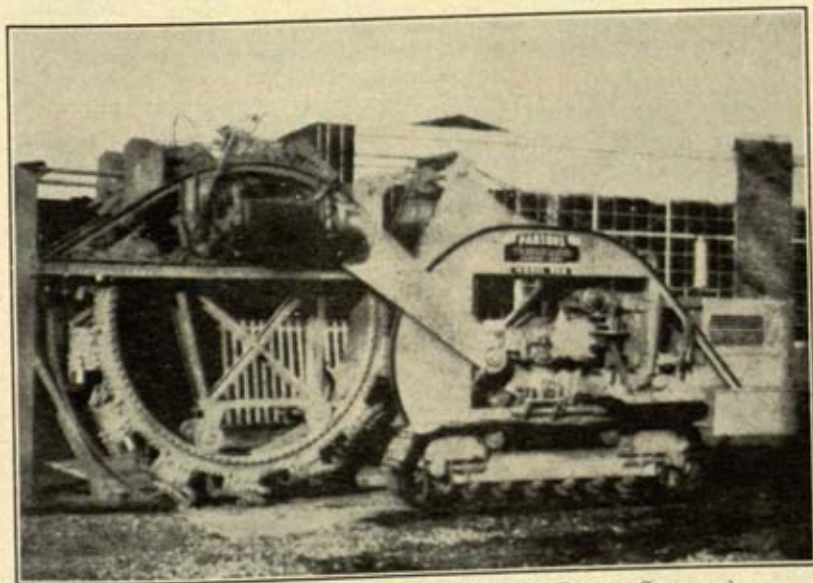


FIG. 6-25. Wheel-type trenching machine. (*The Parsons Company.*)

except rock. They are available in various sizes for digging trenches of varying depths and widths. They are usually crawler-mounted to increase their stability and to distribute the weight over a greater area.

**Wheel-type Trenching Machines.** Figure 6-25 illustrates a wheel-type trenching machine. These machines are available with maximum cutting depths exceeding 8 ft, with trench widths varying from 12 in. or less to approximately 60 in. Many of them are available with 25 or more digging speeds to permit the selection of the most suitable speed for any job condition.

The excavating part of the machine consists of a power-driven wheel, on which are mounted a number of removable buckets, equipped with cutter teeth. Buckets are available in varying widths, to which there



may be attached side cutters when it is necessary to increase the width of a trench. The machine is operated by lowering the rotating wheel to the desired depth, while the unit moves forward slowly. The earth is picked up by the buckets and deposited onto an endless belt conveyor, which can be adjusted to discharge the earth on either side of the trench.

Table 6-12 gives representative specifications for wheel-type trenching machines. As these specifications do not necessarily include all machines that are available, a prospective purchaser should consult the manufacturer's specifications for the particular machine under consideration. The various trench widths for a given machine are obtained by using different bucket widths and installing side cutters.

TABLE 6-12. REPRESENTATIVE SPECIFICATIONS FOR WHEEL-TYPE TRENCHING MACHINES

Max trench depth, ft.	Trench width, in.	Engine power, hp	Approximate weight, lb	Wheel speed, fpm	Travel speed, mph	Digging speed, fpm
5.5	15-18-21	55	15,000	36-266	0.5-2.7	0.2-10
6.0	20-23-26					
	16-18-20	67	16,500	153-410	0.16-4.6	2.8-57.5
	20-22-24					
	24-26-28					
	28-30					
8.5	38-40	110	62,000	243	1.9	1.3-35.0
	40-51					

Wheel-type machines are especially suited to excavating trenches for water-, gas-, and oil-pipe lines, buried telephone cables, and pipe drains which are placed in relatively shallow trenches. They also may be used to excavate trenches for sewer pipes up to the maximum digging depths.

**Ladder-type Trenching Machines.** Figure 6-26 illustrates a ladder-type trenching machine. By installing extensions to the ladders or booms, and by adding more buckets and chain links, it is possible to dig trenches in excess of 30 ft deep with the large machines. Trench widths in excess of 12 ft may be dug. Most of these machines have booms whose lengths may be varied, thereby permitting a single machine to be used on trenches varying considerably in depth. This eliminates the need of owning a different machine for each depth range. A machine may have 30 or more digging speeds to suit the needs of any given job.

The excavating part of the machine consists of two endless chains, which travel along the boom, to which there are attached cutter buckets equipped with teeth. In addition, shaft-mounted side cutters may be

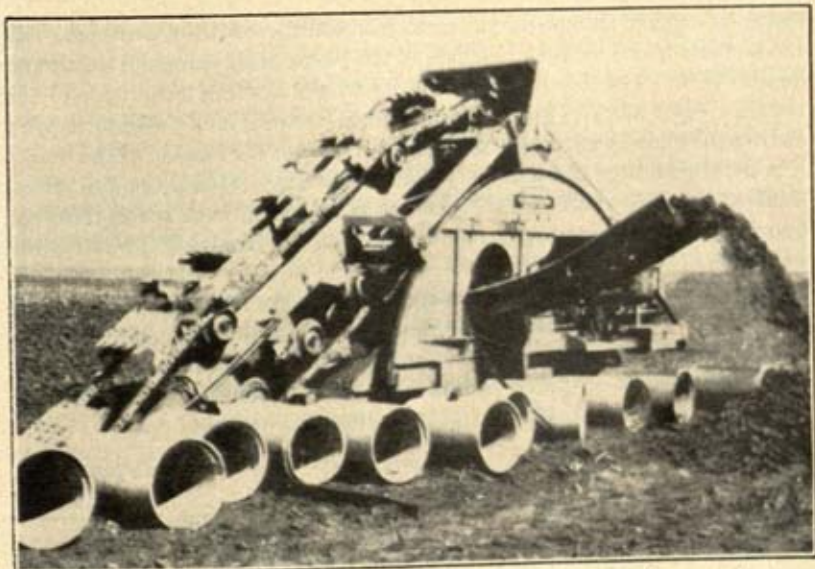


FIG. 6-26. Ladder-type trenching machine. (*The Parsons Company.*)

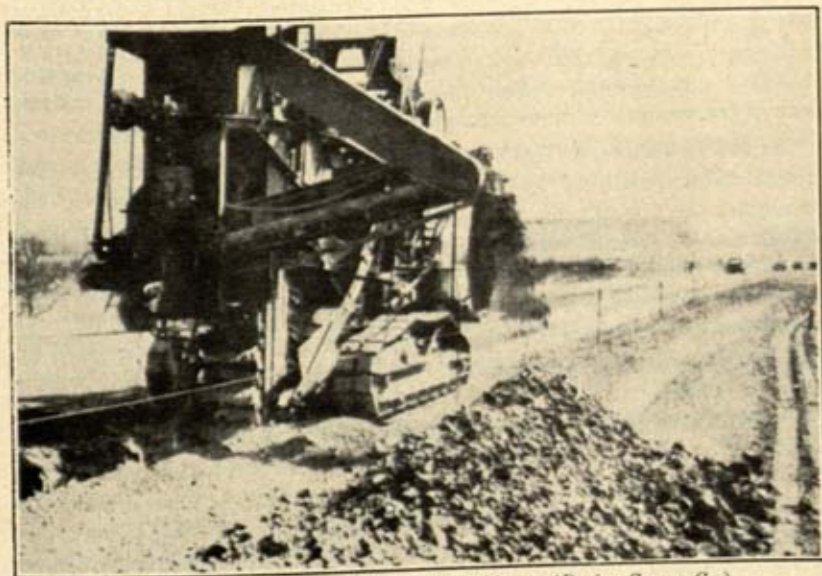


FIG. 6-27. Vertical-boom trenching machine. (*Barber-Greene Co.*)



installed on each side of the boom to increase the width of a trench. As the buckets travel up the underside of the boom, they bring out earth and deposit it on a belt conveyor, which discharges it along either side of the trench. As a machine moves over uneven ground, it is possible to vary the depth of cut by adjusting the position, but not the length, of the boom.

A modification of the ladder-type machine is one with a vertical boom, illustrated in Fig. 6-27. This machine is available with seven different boom sizes, which permit trench depths varying from 4 ft to 8 ft 3 in., with trench widths varying from 14 to 24 in.

Table 6-13 gives representative specifications for ladder-type trenching machines. The prospective purchaser of a machine should check the manufacturer's specifications for the particular machine under consideration. The various trench widths for a given machine are obtained by using different bucket widths and installing side cutters.

TABLE 6-13. REPRESENTATIVE SPECIFICATIONS FOR LADDER-TYPE TRENCHING MACHINES

Max trench depth, ft	Trench width, in.	Engine power, hp	Approximate weight, lb	Bucket speed, fpm	Travel speed, mph	Digging speed, fpm
4	6-8	47	6,600	245-538	0.7-3.4	2.2-21.8
8.5	16-36	55	23,000	96-225	1.4-3.2	0.5-13.8
12.5	16-42	74	29,500	135-542	1.4-3.2	0.3-9.7
15	18-54	90	40,000	103-168	1.7	0.7-15.5

As can be seen from Table 6-13, ladder-type trenching machines have considerable flexibility in trench depths and widths. However, the machines are not suitable for excavating trenches in rock or where large quantities of ground water, combined with unstable soil, prevent the walls of a trench from remaining in place. If the soil, such as loose sand or mud, tends to flow into the trench, it may be desirable to adopt some other method of excavating the trench. Usually, the trench is lined on both sides with sheet piling, lumber or steel, prior to excavating with a clamshell bucket.

**Selecting the Most Suitable Equipment for Excavating Trenches.** The choice of equipment to be used in excavating a trench will depend on the job conditions, the depth and width of the trench, the class of soil, the extent to which ground water is present, the width of the right of way for the disposal of excavated earth, and the type of equipment already owned by a contractor.

If a relatively shallow and narrow trench is to be excavated in firm soil, the wheel-type trenching machine is probably the most suitable. How-

ever, if the soil is rock, which requires blasting, the most suitable excavator will be a hoe, or a less desirable substitute could be a dragline. If the soil is an unstable, water-saturated material, it may be necessary to use a dragline, hoe, or clamshell and let the walls establish a stable slope. If it is necessary to install solid sheeting to hold the walls in place, neither a hoe nor a dragline will work satisfactorily. A clamshell, which can excavate between the trench braces that hold the sheeting in place, probably will be the best equipment for the job.

Consider the selection of a machine to excavate a trench 24 ft deep and 10 ft wide in soil which is sufficiently firm to require only shoring to hold the walls in place. A trench of this size can be excavated with a ladder-type machine provided the length and height of the conveyor belt are adequate to dispose of the earth along one side of the trench. The cross-section area of the trench will be 240 sq ft. If the loose earth has a 30 per cent swell, the cross-section area of the spoil pile will be

$$240 \times 1.3 = 312 \text{ sq ft}$$

If the excavated earth will repose with 1:1 side slopes, the pile will have a height of 17.6 ft and a base width of 35.2 ft. If a minimum of 4 ft of clearance is required along the side of the trench, the end of the conveyor must have a height clearance of 17.6 ft and a length of approximately 27 ft, measured from the center of the trench. The casting effect on the earth, as it leaves the end of the conveyor belt, may permit the use of a shorter conveyor. Unless the machine under consideration satisfies these clearances, it is probable that difficulties will be experienced in disposing of the earth. A dragline would have no difficulty in disposing of the excavated earth. Also, a dragline can be used to backfill the trench if more suitable equipment is not available.

**Production Rates of Trenching Machines.** Many factors will influence the production rates of trenching machines. These include the class of soil, depth and width of the trench, extent of shoring required, topography, climatic conditions, extent of vegetation such as trees, stumps, and roots, physical obstructions such as buried pipes, sidewalks, paved streets, buildings, etc., and the speed with which the pipe can be placed in the trench. Any factors that may affect the progress on a project should be considered in estimating the probable digging speed of a trenching machine.

In laying oil and gas pipes through open level country, with no physical obstructions to interfere with the progress, it is possible to install in excess of 6,000 ft of pipe in an 8-hr day. This is equivalent to approximately 800 ft per hr, which is not excessive for a wheel-type machine. However, if a trench must be excavated into rock over a rough terrain covered with



heavy timber, it may not be possible to excavate more than a few hundred feet per day.

If a trench is dug for the installation of sewer pipe, under favorable conditions, it is possible that the machine could dig 300 ft of trench per hour. However, an experienced pipe-laying crew may not be able to lay more than 25 joints of small-diameter pipe, 3 ft long, in an hour. Thus the speed of the machine will be limited to about 75 ft per hr regardless of its ability to dig more trench. In estimating the probable rate of digging a trench, an appropriate operating factor must be applied to the speed at which the machine could dig if there were no interruptions.

**EXAMPLE.** Estimate the probable average production rate, in feet per hour, in excavating a trench 36 in. wide, with a maximum depth of 12 ft, in hard, tough clay. The trench will be dug for the installation of a 21-in.-diameter sewer pipe, which can be laid at a rate of approximately 30 ft per hr. An examination of the site along the trench reveals that there are obstructions which will reduce the digging speed to approximately 60 per cent of the theoretically possible speed. This will require the application of an operating factor of 0.6 to the speed of the machine.

An examination of Table 6-13 indicates a ladder-type machine with a maximum digging depth of 12.5 ft. Considering the class of soil and the depth and width of the trench, the maximum possible digging speed should be about 1 fpm, or 60 ft per hr. The application of the operating factor will reduce the average speed to 36 ft per hr. However, since only 30 ft of pipe can be laid per hour, this will be the controlling speed.

The probable cost per linear foot of trench, for excavating only, should be as follows:

Trenching machines, from Appendix A	= \$ 5.40 per hr
Operator	= 2.00 per hr
Helpers, 3 men @ \$1.25 per hr	= 3.75 per hr
Foreman, $\frac{1}{2}$ time charged to excavating	= 1.50 per hr
Total cost	= \$12.65 per hr
Cost per lin ft, $\$12.65 \div 30$ ft per hr	= 0.422

### PROBLEMS

**6-1.** Select the minimum size power shovel that will excavate 60,000 cu yd of ordinary earth in 130 days of 8 hr each.

The depth of excavation will be 10 ft, and the angle of swing will be  $120^\circ$ . The job and management factors will be good. Assume that 20 per cent of the time will be lost owing to weather.

**6-2.** The fill for an airfield requires 480,000 cu yd of earth, bank measure. A power shovel will be used to excavate and load the earth, which is tough clay.

The earth will be obtained from a large hill, with sufficient operating room to permit trucks to back in on each side of the shovel. The angle of swing will be  $90^\circ$ . Any desired depth can be dug. The job conditions will be excellent, and the management conditions will be good.

If the project must be completed in 320 days of 8 hr each, determine the minimum size shovel required. Assume that 25 per cent of the time will be lost because of weather.

**6-3.** For each of the given conditions determine the probable output of a 1-cu-yd power shovel, in cubic yards per hour bank measure.

	Class of earth				
	Moist loam	Moist loam	Common earth	Hard clay	Wet clay
Depth of dig, ft. ....	10	4	11	10	7
Angle of swing, deg. ....	90	150	75	60	120
Job conditions. ....	Good	Fair	Good	Fair	Poor
Management conditions. ....	Excellent	Good	Good	Fair	Poor

**6-4.** A project involves excavating and hauling 64,500 cu yd of good common earth. The following conditions apply to the project as it is now being operated.

Power shovel used,  $\frac{3}{4}$  cu yd  
 Cost of operating shovel, \$9.20 per hr  
 Depth of cut, 8.6 ft  
 Angle of swing, 120°  
 Job conditions, good  
 Management conditions, fair  
 Size trucks used to haul earth,  $3\frac{1}{2}$  cu yd  
 No. trucks used, 5  
 Cost of each truck and driver, \$3.75 per hr

Lost time waiting for trucks and due to other delays reduces the output of the shovel to 70 per cent of the possible output for the given conditions.

After analyzing the project, you believe that the output can be increased substantially if the following changes are made:

- Reduce the depth of dig to the optimum for the shovel.
- Use a small bulldozer to keep the floor of the pit clean so the trucks can be spotted closer to the shovel to reduce the angle of swing to 75°. The cost of the bulldozer and operator will be \$4.00 per hour.
- Pay a bonus of \$0.02 per cubic yard for all production in excess of the original amount to increase the management conditions to good.
- Add extra trucks to take care of the increased output of the shovel.

Determine whether the changes are financially justified. What will be the reduction in the excavating cost per cubic yard? What will be the amount of the total net saving on the project?

**6-5.** Select a dragline to excavate a drainage ditch, trapezoidal in cross section, 20 ft wide at the bottom, 40 ft wide at the top, 10 ft deep, and  $2\frac{1}{2}$  miles long, in tough clay. The excavated earth will be cast into a spoil bank along one side of the ditch, with a 20-ft berm left between the edge of the ditch and the toe of the spoil bank. The slope of the earth in the spoil bank will be 1:1 on each side.

The job must be completed in 10 weeks. A week will be 6 days of 22 hr each. Assume that 10 per cent of the time will be lost because of weather. Job and management conditions will be good. Assume a suitable angle of swing. Check in Table 6-5 to be sure that the size selected has sufficient working range for the project.

**6-6.** What is the largest-capacity medium-duty dragline bucket that can be used with a dragline having a 70-ft boom when the boom is operating at an angle of 35°? The earth to be excavated will weigh 80 lb per cu ft loose volume.

**6-7.** A sewer trench, 7 ft wide and 15 ft deep, is to be excavated in tough clay containing no excess ground water. The sides of the trench will stand safely if shores are



installed at approximately 8-ft intervals along the trench. There is sufficient space along the trench to deposit the excavated earth.

What type equipment would you use to excavate this trench? Give your reasons for the selection.

**6-8.** A sewer trench is to be excavated in moist sand to a depth of 12 ft. The bottom of the trench must be 5 ft wide. Tests indicate that the soil will be stable if the sides of the trench have a slope of 1 ft horizontally to 3 ft vertically. The excavated earth may be deposited along the trench.

What type equipment would you select to excavate this trench? Give your reasons for the selection.

## CHAPTER 7

### TRUCKS AND WAGONS

**Trucks.** In handling earth, aggregate, rock, ore, coal, and other materials, trucks serve one purpose. They are hauling units which, because of their high speeds when operating on suitable roads, have high capacities and provide relatively low hauling costs. They provide a high degree of flexibility, as the number in service may be increased or decreased easily to permit modifications in the total hauling capacity of a fleet. Most trucks may be operated over any haul road for which the surface is sufficiently firm and smooth and on which the grades are not excessively steep. Some units now in use are designated as off-highway trucks because their sizes and total loads are larger than are permitted on highways. These trucks are used for hauling materials on large projects, where the sizes and costs are justified.

Trucks may be classified according to a great many factors, including the following:

1. Size and type of engine, gasoline, diesel, butane, propane
2. Number of gears
3. Kind of drive, two-wheel, four-wheel, six-wheel, etc.
4. Number of wheels and axles and arrangement of driving wheels
5. Method of dumping the load, rear dump, side dump
6. Class of material hauled, earth, rock, coal, ore, etc.
7. Capacity, in tons or cubic yards
8. Method of dumping the load for rear dumps, hydraulic or cable

If trucks are to be purchased for general material hauling, the purchaser should select units that are adaptable to the purposes for which they will be used. However, if trucks are to be used on a given project for a given purpose, the purchaser should select trucks that most nearly fit the requirements of the project.

**Rear-dump Trucks.** Rear-dump trucks are suitable for use in hauling many types of materials. The shape of the body, such as the extent of sharp angles, corners, and the contour of the rear, through which the materials must flow during dumping will affect the ease or difficulty of dumping. The bodies of trucks that will be used to haul wet clay and similar materials should be free of sharp angles and corners. Dry sand and gravel will flow easily from almost any shape of body. If quarry rock is to be hauled, bodies should be shallow with sloping sideboards.



Figure 7-1 shows a 22-ton single-axle dual-wheel rear-dump truck dumping its load. The body of this truck is approximately 15 ft 3 in. long, 8 ft 4 in. wide, and 3 ft 6 in. deep, inside dimensions. The truck capacity is 14.8 cu yd. It is equipped with 14.00 by 24, 20-ply front tires and 18.00 by 24, 24-ply rear tires.

**Bottom-dump Wagons.** If units are to be used to haul materials, such as sand, gravel, reasonably dry earth, coal, etc., which flow easily, the use of bottom-dump wagons will reduce the time required to unload the units. Such units are particularly suitable for use where the materials



FIG. 7-1. Hydraulically operated rear-dump truck discharging its load. (*Euclid Division, General Motors Corp.*)

are distributed in layers on a fill or are discharged through grizzlies into hoppers. When discharging the loads onto fills, the wagons can dump their loads while moving. When discharging through grizzlies, they will need to stop for only a few seconds. The rapid rate of discharging the load gives these wagons a time advantage over rear-dump trucks.

As the doors through which these units discharge their loads have limited openings, difficulties may be experienced in discharging such materials as wet, sticky clay, especially if they are in large lumps.

These wagons are satisfactory hauling units on projects such as earthen dams, levees, highways, and airports, where large quantities of materials are to be transported and haul roads can be kept in reasonably good condition. They may be loaded by power shovels, draglines, or portable belt

loaders. They have top speeds of approximately 30 mph, with hauling capacities in excess of 25 cu yd.

Figure 7-2 shows a 13-cu-yd, struck-capacity, wagon receiving its load from a belt loader, while Fig. 7-3 shows the same unit discharging its load on a fill.

**Capacities of Trucks and Wagons.** There are at least three methods of expressing the capacities of trucks and wagons: by the load which it will carry, expressed in tons; by its struck volume; and by its heaped volume, the latter two expressed in cubic yards.



FIG. 7-2. Tractor-pulled bottom-dump wagons loaded by earth loader. (Euclid Division, General Motors Corp.)

The struck capacity of a truck is the volume of material which it will haul when it is filled to the top of the sides, with no material above the sides. The heaped capacity is the volume of material which it will haul when the load is heaped above the sides. The capacity should be expressed in cubic yards. While the struck capacity remains fixed for any given unit, the heaped capacity will vary with the height to which the material may extend above the sides and with the length and width of the body. Wet earth or sandy clay may be hauled with a slope of 1:1, while dry sand or gravel may not permit a slope greater than 3:1. In order to determine the probable heaped capacity of a unit, it is necessary to know the struck capacity, the length and width of the body, and the slope at which the material will remain stable while the unit is moving. Smooth haul roads will permit a larger heaped capacity than rough haul roads.



Because of variations in the heaping capacities of units it may be better to compare them on the basis of their struck capacities. In any event the capacities should be determined or compared in a realistic manner.

The weight capacity may limit the volume of the load when a unit is used to haul heavy material, such as iron ore. However, when the specific gravity of the material is such that the safe load is not exceeded, a unit may be filled to its heaped capacity.

In some instances it is possible to add sideboards to increase the depth of the body of a truck or wagon, thereby permitting it to haul a larger

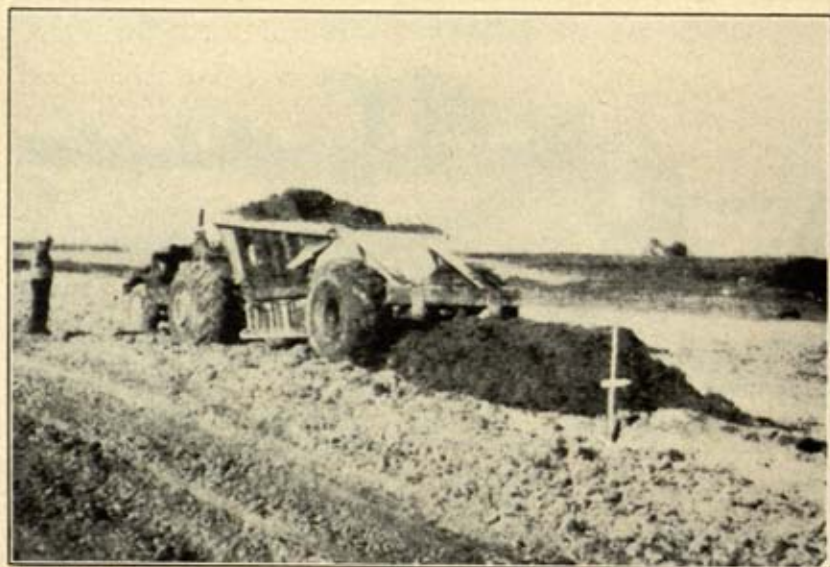


FIG. 7-3. Bottom-dump wagon discharging its load on a fill. (Euclid Division, General Motors Corp.)

load. This practice probably will increase the hourly cost of operating a unit, because of higher fuel consumption, reduced tire life, more frequent failures of parts, such as axles, gears, brakes, and clutches, and higher maintenance costs. However, if the value of the extra material hauled is greater than the total increase in the cost of operating a vehicle, the overloading is justified. The contractor on the Bonny Dam in Colorado added sideboards to his 13- and 25-cu-yd, struck capacity, dump trucks to increase the heaped capacities to 24 and 32 cu yd, respectively. In considering hauling larger volumes of materials, the maximum safe loads on the tires should be checked to prevent excessive overloading, which might result in considerable lost time due to tire failures.

**Balancing the Capacities of Hauling Units with the Size of the Exca-**

**vator.** In loading with power shovels, draglines, or belt loaders, it is desirable to use units whose capacities balance the output of the excavator. If this is not done, operating difficulties will develop and the combined cost of excavating and hauling material may be higher than when balanced units are used. For example, when an excavator is used to load earth into trucks, the size of the trucks may introduce several factors which will affect the production rate and the cost of handling earth.

1. Advantages of small compared with large trucks:
  - a. They are more flexible in maneuvering, which may be an advantage on short hauls.
  - b. They may have higher speeds.
  - c. There is less loss in production when one truck in a fleet breaks down.
  - d. It is easier to balance the number of trucks with the output of the excavator, which will reduce the time lost by the trucks or the excavator.
2. Disadvantages of small compared with large trucks:
  - a. It is more difficult for the excavator to load owing to small target for depositing earth.
  - b. More total time is lost in spotting the trucks because of the larger number required.
  - c. More drivers are required to haul a given output of material.
  - d. The greater number of trucks required increases the danger of bunching up at the pit, along the haul road, or at the dump.
  - e. The greater number of trucks required may increase the total investment in hauling equipment, with more expensive maintenance and repairs and more parts to stock.
3. Advantages of large compared with small trucks:
  - a. Fewer trucks are required, which may reduce the total investment in hauling units and the cost of maintenance and repairs.
  - b. Fewer drivers are required.
  - c. The smaller number of trucks facilitates synchronizing the equipment and reduces the danger of bunching up by the trucks. This is especially true for long hauls.
  - d. They give a larger target for the excavator during loading.
  - e. They reduce the frequency of spotting trucks under the excavator.
  - f. There are fewer trucks to maintain and repair and fewer parts to stock.
  - g. The engines ordinarily use cheaper fuels.
4. Disadvantages of large compared with small trucks:
  - a. The cost of truck time at loading is greater, especially with small excavators.



- b. The heavier loads may cause more damage to the haul roads, thus increasing the cost of maintaining haul roads.
- c. It is more difficult to balance the number of trucks with the output of the excavator.
- d. Repair parts may be more difficult to obtain.
- e. The largest sizes may not be permitted to haul on highways.

A rule-of-thumb practice which frequently is used in selecting the size of trucks is to use trucks with a minimum capacity of four to five times the capacity of the excavator bucket or dipper, when loading with a drag-line or shovel. The dependability of this practice is discussed in the following analyses.

**EXAMPLE.** Consider a  $\frac{3}{4}$ -cu-yd shovel excavating good common earth, with a  $90^\circ$  swing, with no delays waiting for hauling units, and with a 21-sec cycle time. If the dipper and the trucks are operated at their heaped capacities, the swelling effect of the earth should permit each to carry its rated or struck capacity, expressed in cubic yards bank measure. Assume that the number of dippers required to fill a truck will equal the capacity of the truck divided by the size of the dipper, both expressed in cubic yards. The sizes of the trucks considered are based on the struck capacities. Assume that the time for a travel cycle, excluding the time for loading, will be the same for the several sizes of trucks considered. If this is not true, an appropriate travel cycle should be determined for each truck. The time for a travel cycle, which includes traveling to the dump, dumping, and returning to the shovel, will be 6 min.

If 3-cu-yd trucks are used, it will require 4 dippers to fill a truck. With a shovel cycle of 21 sec it will be necessary to provide a new truck every 84 sec, or 1.4 min. The minimum round-trip cycle for a truck will be 7.4 min. The minimum number of trucks required to keep the shovel busy will be the round-trip time divided by the loading time =  $7.4 \div 1.4 = 5.3$ . Thus it will be necessary to use 6 trucks to keep the shovel busy or else permit the shovel to idle between trucks. Since the time required to load 6 trucks will be  $6 \times 1.4 = 8.4$  min, the lost time per truck cycle will be  $8.4 - 7.4 = 1$  min per truck. This will produce an operating factor of

$$\frac{7.4}{8.4} \times 100 = 88 \text{ per cent for the trucks}$$

If 6-cu-yd trucks are used, it will require 8 dippers to fill a truck. The time required to load a truck will be 168 sec, or 2.8 min. The minimum round-trip cycle for a truck will be 8.8 min. The minimum number of trucks required to keep the shovel busy will be  $8.8 \div 2.8 = 3.15$ . For this condition it probably will be cheaper to provide 3 trucks and let the shovel idle a short time between trucks. The time required to load 3 trucks will be  $3 \times 2.8 = 8.4$  min. Thus the shovel will lose  $8.8 - 8.4 = 0.4$  min in loading 3 trucks. The time lost will be  $\frac{0.4}{8.8} \times 100 = 4.5$  per cent, which is not serious.

If 4 trucks are used, the time required to load them will be  $4 \times 2.8 = 11.2$  min. As this will increase the round-trip cycle of each truck from 8.8 to 11.2 min, the lost time per truck cycle will be 2.4 min per truck. This will result in a loss of

$$\frac{2.4}{11.2} \times 100 = 21.4 \text{ per cent of the truck time}$$

which is equivalent to an operating factor of 78.6 per cent for the trucks.

If 15-cu-yd trucks are used, it will require 20 dippers to fill a truck. The time required to load a truck will be 420 sec, or 7 minutes. The minimum round-trip cycle for a truck will be 13 min. The minimum number of trucks required to keep the shovel busy will be  $13 \div 7 = 1.85$ . Use 2 trucks. Since the time required to load 2 trucks will be  $2 \times 7 = 14$  min, the lost time per truck cycle will be  $14 - 13 = 1$  min per truck. This will produce an operating factor of  $1\frac{3}{4} \times 100 = 93$  per cent for the trucks.

**The Effect of the Size of Trucks on the Cost of Hauling Earth.** A comparison of the cost of hauling earth with each of several sizes of trucks based on the previous analysis is illustrated in Table 7-1. The information given in the table is obtained as illustrated in the following example.

**EXAMPLE.** Assume that the shovel operates at 80 per cent efficiency while it is excavating, with no lost time waiting for trucks.

No. cycles per min,  $60 \div 21 = 2.86$

No. cycles per hr,  $60 \times 2.86 = 171.6$

Ideal output per hr,  $171.6 \times \frac{3}{4} = 128$  cu yd

Output at 80% efficiency,  $0.8 \times 128 = 102$  cu yd per hr

Travel cycle time for each truck, 6 min

If 6-cu-yd trucks are used, the ideal number will be 3.15, as previously determined.

If 3 trucks are used, the output will be  $\frac{3.0}{3.15} \times 102 = 97$  cu yd per hr.

The cost per hr for a truck and a driver, \$4.90

The total cost per hr for trucks,  $3 \times \$4.90 = \$14.70$

The truck cost while loading,  $\frac{2.8 \times \$4.90}{60} = \$0.23$

The truck cost per cu yd of earth loaded,  $\frac{\$0.23}{6} = \$0.038$

The hauling cost per cu yd equals the total truck cost per hr divided by the output per hr,  $\$14.70 \div 97 = \$0.152$

TABLE 7-1. COMPARISON OF THE COSTS OF HAULING COMMON EARTH WITH VARIOUS SIZES OF TRUCKS, USING A  $\frac{3}{4}$ -CU-YD POWER SHOVEL FOR EXCAVATING

Size truck, cu yd	No. trucks	Output per hr, cu yd	Loading time, min	Truck cost per hr		Truck cost at loading		Hauling cost per cu yd
				Per truck	Total	Per truck	Per cu yd	
3	5	96	1.4	\$ 3.75	\$18.75	\$0.09	\$0.030	\$0.195
3	6	102	1.4	3.75	22.50	0.09	0.030	0.221
6	3	97	2.8	4.90	14.70	0.23	0.038	0.152
6	4	102	2.8	4.90	19.60	0.23	0.038	0.192
10	2	89	4.6	7.05	14.10	0.54	0.054	0.159
10	3	102	4.6	7.05	21.15	0.54	0.054	0.207
15	2	102	7.0	10.80	21.60	1.26	0.084	0.212
20	2	102	9.3	15.20	30.40	2.36	0.118	0.299



The information given in Table 7-1 indicates that, for the given power shovel and project, the lowest hauling cost will be obtained if three 6-cu-yd trucks are used. For other sizes of power shovels and truck travel cycles the comparative costs given in the table will not necessarily hold true. If the travel cycle for the larger trucks is greater than for the 3-cu-yd trucks, namely, 6 min, the actual time should be used in preparing information similar to that given in the table.

If the size of the excavator is increased, the time lost by the larger trucks at loading will be reduced, which will reduce the hauling cost per cubic yard. One disadvantage in using large trucks, for which costs are paid by the hour, is that the cost of the trucks while they are being loaded will be higher than for smaller trucks. This results from two factors, the longer time required to load and the higher hourly cost of the larger trucks. Since it is desirable to have a truck under the excavator at all times, the total hourly truck cost while loading 15-cu-yd trucks will be \$10.80, compared with \$4.90 for 6-cu-yd trucks, regardless of the size of the excavator. Unless this higher cost for the larger trucks can be recovered by more economical performance during the travel cycle, the use of the larger trucks will not be justified.

**EXAMPLE.** The higher cost of using trucks that are too large for the excavator is illustrated by considering the use of a 15-cu-yd truck to haul earth. The earth will be loaded by 2 men, each of whom will load 1 cu yd per hr. The total time required to load the truck will be  $15 \div 2 = 7.5$  hr. The cost of lost time by the truck while it is being loaded will be  $7.5 \times \$10.80 = \$81.00$  if the cost of the driver is included in the truck cost. Excluding the driver, the truck cost will be  $7.5 \times \$9.30 = \$69.75$ . While this is an exaggerated example, it does illustrate the effect which loading time has on the cost of hauling earth with trucks.

**The Effect of the Size of the Excavator on the Cost of Excavating and Hauling Earth.** If the size of the excavator is increased, while the size of trucks remains constant, the resulting increase in the output of the shovel will reduce the time required to load a truck. This will reduce the truck cost per cubic yard during loading. The effect which the size of a power shovel has on the truck cost at loading and the hauling cost is illustrated in Table 7-2. The material will be good common earth, the depth of cut will be optimum, and the angle of swing will be  $90^\circ$ . The operating factor for the shovel will be 80 per cent, with no lost time waiting for trucks. Trucks with a heaped capacity of 15 cu yd bank measure will be used to haul the earth. The travel cycle for the trucks will be 8 min. The cost per hour for a truck and driver will be \$10.80.

Sample calculations, using a 1 cu yd shovel, are as follows:

Ideal output of the shovel is 175 cu yd per hr

Output at 80% efficiency,  $0.8 \times 175 = 140$  cu yd per hr

Time to load a truck,  $\frac{15 \times 60}{140} = 6.4$  min

Round-trip time per truck, with no delays waiting for the shovel,  $6.4 + 8 = 14.4$  min  
 No. trucks needed,  $14.4 \div 6.4 = 2.25$

Output using 2 trucks,  $\frac{2.0 \times 140}{2.25} = 125$  cu yd per hr

Output using 3 trucks, 140 cu yd per hr

Cost per hr for 2 trucks,  $2 \times \$10.80 = \$21.60$

Cost per hr for 3 trucks,  $3 \times \$10.80 = \$32.40$

Truck cost at loading, per truck,  $\frac{6.4}{60} \times \$10.80 = \$1.15$

Truck cost at loading, per cu yd,  $\$1.15 \div 15$  cu yd =  $\$0.077$

Hauling cost per cu yd, using 2 trucks,  $\$21.60 \div 125 = \$0.173$

Hauling cost per cu yd, using 3 trucks,  $\$32.40 \div 140 = \$0.232$

TABLE 7-2. THE EFFECT OF THE SIZE OF THE EXCAVATOR ON THE COST OF HAULING EARTH WITH 15-CU-YD TRUCKS

Size shovel, cu yd	Output per hr, cu yd	Truck time		No. trucks	Truck cost per hr	Truck cost at loading		Hauling cost per cu yd
		Loading, min	Round trip, min*			Per truck	Per cu yd	
$\frac{1}{2}$	76	11.8	19.8	2	\$21.60	\$2.13	\$0.142	\$0.285
$\frac{3}{4}$	108	8.3	16.3	2	21.60	1.49	0.099	0.200
1	125†	6.4	14.4	2	21.60	1.15	0.077	0.173
1	140	6.4	14.4	3	32.40	1.15	0.077	0.232
$1\frac{1}{2}$	191	4.7	12.7	3	32.40	0.85	0.057	0.170
2	231†	3.8	11.8	3	32.40	0.68	0.045	0.140
2	240	3.8	11.8	4	43.20	0.68	0.045	0.180
$2\frac{1}{2}$	280	3.2	11.2	4	43.20	0.58	0.039	0.154
3	312	2.9	10.9	4	43.20	0.52	0.035	0.139

\* Lost time waiting at the shovel is not included.

† These values are reduced because of the limiting hauling capacities of the trucks.

While the information given in Table 7-2 indicates that the cost of hauling earth is reduced as the size of the shovel is increased, the job planner is concerned with the combined cost of excavating and hauling earth. This cost may be obtained by adding the cost of operating the shovel, including labor, to the cost of the trucks. Table 7-3 gives this information. The costs given in the table do not include the cost of moving the equipment to the project and setting it up. The cost of a shovel is based on the cost of owning and operating, with an allowance for the operator and an oiler.

**The Effect of Grade on the Cost of Hauling Earth with Trucks.** In constructing a fill it frequently is possible to obtain the earth from a borrow pit located either above or below the fill. If the borrow pit is above the fill, the effect of the favorable grade on the loaded truck is to reduce



the required rimpull by 20 lb per gross ton for each 1 per cent of grade. If the borrow pit is below the fill, the effect of the adverse grade on the loaded truck is to increase the required rimpull by 20 lb per gross ton for each 1 per cent of grade. Obviously the grade of the haul road will affect the hauling capacity of a truck, its performance, and the cost of hauling earth. It may be more economical to obtain earth from a borrow pit above, instead of below, the fill, even though the haul distance from the higher pit is greater than from the lower pit. This is an item which should be given consideration in locating borrow pits.

TABLE 7-3. THE COST OF EXCAVATING AND HAULING EARTH, USING VARIOUS SIZES OF POWER SHOVELS AND 15-CU-YD TRUCKS

Size shovel, cu yd	Output per hr, cu yd	Shovel cost per hr	No. trucks	Truck cost per hr	Excavating cost per cu yd	Hauling cost per cu yd	Total cost per cu yd
$\frac{1}{2}$	76	\$ 8.20	2	\$21.60	\$0.108	\$0.285	\$0.393
$\frac{3}{4}$	108	9.30	2	21.60	0.086	0.200	0.286
1	125*	9.60	2	21.60	0.077	0.173	0.250
1	140	9.60	3	32.40	0.069	0.232	0.291
$1\frac{1}{2}$	191	14.25	3	32.40	0.075	0.170	0.245
2	231*	19.85	3	32.40	0.086	0.140	0.226
2	240	19.85	4	43.20	0.083	0.180	0.263
$2\frac{1}{2}$	280	22.45	4	43.20	0.080	0.154	0.234
3	312	26.90	4	43.20	0.086	0.139	0.225

\* These values are reduced because the hauling capacities of the trucks limit the output.

If earth is hauled downhill, it may be possible to add sideboards to the vehicle to increase the hauling capacity, up to the maximum load which the tires can carry. In some instances it will be desirable to use larger tires to permit the trucks to haul greater loads. If the earth is hauled uphill, it may be necessary to reduce the size of the load or the travel speed of the truck, either of which will increase the cost of hauling earth.

**EXAMPLE.** The following example will illustrate the effect of grade on the cost of hauling earth.

The project requires 1,000,000 cu yd of earth, bank measure.

The material will be good common earth, weighing 2,700 lb per cu yd bank measure, with a swell of 25 per cent.

Borrow pit 1 will require an average haul of 0.66 mile up an average grade of 2.2 per cent.

Borrow pit 2 will require an average haul of 0.78 mile down an average slope of 1.4 per cent.

Both borrow pits are easily accessible to the trucks, which will permit spotting on either side of the shovel, whose angle of swing will not exceed 90°. Excavating can be done at optimum depth.

Job conditions will be excellent, and management conditions will be good. The job-management factor should be not less than 0.80.

The earth will be excavated with a 3-cu-yd power shovel, with a probable output of  $0.80 \times 390 = 312$  cu yd per hr bank measure.

The average rolling resistance of the haul road is estimated to be 60 lb per ton.

The coefficient of traction between the truck tires and the haul road will average 0.60.

The earth will be hauled with bottom-dump wagons, whose estimated heaped capacity will be 15 cu yd bank measure.

The average elevation will be 600 ft above sea level.

The specifications for the trucks are as follows:

Pay-load capacity, 40,000 lb  
 Engine, diesel, 200 hp  
 Empty weight, 36,800 lb  
 Gross weight, loaded, 76,800 lb  
 Gross-weight distribution  
   Front axle, 12,000 lb  
   Drive axle, 32,400 lb  
   Trailer axle, 32,400 lb  
 Size tires on drive and trailer axles, 24.00  $\times$  25

Gear	Speed, mph	Rimpull, lb
1st	3.2	19,900
2d	6.3	10,100
3d	11.9	5,350
4th	20.8	3,060
5th	32.7	1,945

The maximum usable rimpull of a loaded truck, as limited by the coefficient of traction, will be  $32,400 \times 0.6 = 19,440$  lb. This is sufficiently high to eliminate the danger of tire slippage, except possibly in first gear.

The cost of hauling earth from borrow pit 1 is determined as follows:

The combined effect of rolling resistance and grade on a loaded truck will be

Rolling resistance = 60 lb per ton  
 Grade resistance,  $2.2 \times 20 = 44$  lb per ton  
 Total resistance = 104 lb per ton  
 Gross weight of truck,  $76,800 \div 2,000 = 38.4$  tons  
 Required rimpull,  $38.4 \times 104 = 3,994$  lb  
 Max speed of loaded truck, 11.9 mph

The combined effect of rolling resistance and grade on an empty truck will be

Rolling resistance = 60 lb per ton  
 Grade resistance,  $2.2 \times 20 = -44$  lb per ton  
 Total resistance = 16 lb per ton  
 Weight of empty truck,  $36,800 \div 2,000 = 18.4$  tons  
 Required rimpull,  $18.4 \times 16 = 294$  lb  
 Max speed of an empty truck, 32.7 mph



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The time required for each operation in a round-trip cycle should be about as follows:

Loading, 15 cu yd ÷ 312 cu yd per hr	= 0.0482 hr
Lost time in pit and accelerating, 1.5 min	= 0.0250 hr
Travel to the fill, 0.66 mile ÷ 11.9 mph	= 0.0555 hr
Dumping, turning, and accelerating, 1 min	= 0.0167 hr
Travel to pit, 0.66 mile ÷ 32.7 mph	= 0.0202 hr
Round-trip time	= 0.1656 hr

Assume that the trucks will operate an average of 50 min per hr.

$$\text{No. trips per hr, } \frac{1}{0.1656} \times \frac{50}{60} = 5.02$$

$$\text{Volume of earth hauled per truck, } 15 \times 5.02 = 75.3 \text{ cu yd per hr}$$

$$\text{No. trucks required, } 312 \div 75.3 = 4.15$$

Use 4 trucks, which will reduce the output of the shovel slightly.

If a truck and driver cost \$10.80 per hour, the cost of hauling earth will be

$$\$10.80 \div 75.3 = \$0.144 \text{ per cu yd}$$

The cost of hauling earth from borrow pit 2 is determined as follows:

The combined effect of rolling resistance and grade on a loaded truck will be

Rolling resistance	= 60 lb per ton
Grade resistance, $1.4 \times 20$	= 28 lb per ton
Total resistance	= 88 lb per ton
Gross weight of truck, 38.4 tons	
Required rimpull, $38.4 \times 88$	= 3,379 lb

The available rimpull in fifth gear is 1,945 lb, which is more than will be required by the truck. Sideboards can be installed to increase the hauling capacity of the truck. The gross load should be limited to a weight that can be pulled by not over 80 per cent of the rimpull, with the remaining rimpull reserved to accelerate the truck and to be used on sections of the haul road having higher rolling resistance or less steep grades.

Net available rimpull, $0.8 \times 1,945$	= 1,556 lb
Required rimpull for 15 cu yd	= 1,229 lb
Surplus rimpull	= 327 lb
Possible additional load, $327 \div 32$	= 10.2 tons
Possible additional volume, $\frac{10.2 \times 2,000}{2,700}$	= 7.55 cu yd

In order to compensate for the additional weight of the sideboards, the volume of the earth should be increased by not more than 7 cu yd. This will give a total volume of 22 cu yd per load.

The combined effect of rolling resistance and grade on the empty truck will be

Rolling resistance	= 60 lb per ton
Grade resistance, $1.4 \times 20$	= 28 lb per ton
Total resistance	= 88 lb per ton
Weight of empty truck, including sideboards, 19 tons	
Required rimpull, $19 \times 88$	= 1,672 lb
Max speed of an empty truck, 32.7 mph	

The time required for each operation in a round-trip cycle should be about as follows:

Loading, 22 cu yd ÷ 312 cu yd per hr	= 0.0707 hr
Lost time in pit and accelerating, 2 min	= 0.0333 hr
Travel to the fill, 0.78 mile ÷ 32.7 mph	= 0.0238 hr
Dumping, turning, and accelerating, 1.5 min	= 0.0250 hr
Travel to the pit, 0.78 mile ÷ 32.7 mph	= 0.0238 hr
Round-trip time	= 0.1766 hr

Assume that the trucks will operate an average of 50 min per hr.

$$\text{No. trips per hr, } \frac{1}{0.1766} \times \frac{50}{60} = 4.72$$

$$\text{Volume of earth hauled per truck, } 22 \times 4.72 = 103.8 \text{ cu yd per hr}$$

$$\text{No. trucks required, } 312 \div 103.8 = 3.01$$

Use 3 trucks.

If a truck and driver cost \$10.80 per hr, the cost of hauling earth will be

$$\$10.80 \div 103.8 = \$0.105 \text{ per cu yd}$$

A comparison of the cost of hauling earth from the two pits will reveal the extent of savings that may be effected by using pit 2.

Hauling cost from pit 1	= \$ 0.144 per cu yd
Hauling cost from pit 2	= 0.105 per cu yd
Reduction in hauling cost	= \$ 0.039 per cu yd

$$\text{Reduction in total hauling cost, } 1,000,000 \times \$0.039 = \$39,000$$

The use of pit 2 instead of pit 1 will result in a saving in hauling cost of

$$\frac{0.039}{0.144} \times 100 = 27.1 \text{ per cent}$$

which is a significant saving.

Another item which is favorable to pit 2 is the reduction in trucks from 4 to 3, which will result in a reduction in investment in hauling equipment amounting to approximately \$28,000.

**The Effect of Rolling Resistance on the Cost of Hauling Earth.** An important factor which affects the production capacity of a truck or a tractor-pulled wagon is the rolling resistance of the haul road. Rolling resistance is determined primarily by two factors, the physical condition of the road and the tires used on the hauling unit. A great deal can be done to reduce rolling resistance by properly maintaining the road and by selecting proper sizes of tires and then keeping them inflated to the correct pressure. Money spent for these purposes may return dividends, through reduced hauling costs, far in excess of the expenditures. This is one field where the application of engineering knowledge will yield excellent returns.

An earth haul road which is given little or no maintenance will soon become rough, loose, and soft and may develop a rolling resistance of 150



lb per ton or more, depending on the type of soil and weather conditions. If a road is properly maintained with a patrol grader, sprinkled with water, and compacted as required, it may be possible to reduce the rolling resistance to 40 lb per ton or less. Also, sprinkling the road will reduce the damage to hauling equipment by eliminating dust, will reduce the danger of vehicular collision by improving visibility, and will prolong the life of tires because of the cooling effect which the moisture has on the tires.

The selection of proper tire sizes and the practice of maintaining correct air pressure in the tires will reduce that portion of the rolling resistance due to tires. A tire supports its load by deforming where it contacts the road surface until the area in contact with the road will, considering the air pressure in the tires, produce a total force on the road equal to the load on the tire. If the load on a tire is 5,000 lb and the air pressure is 50 psi, the area of contact will be 100 sq in. This neglects any supporting resistance furnished by the side walls of the tire. If, for the same tire, the air pressure is permitted to drop to 40 psi, the area of contact will be increased to 125 sq in. The additional area of contact will be produced by additional deformation of the tire. This will increase the rolling resistance because the tire will be continually climbing a steeper grade as it rotates. The size tire selected and the inflated pressure should be based on the resistance which the surface of the road offers to penetration by the tire. For rigid road surfaces, such as concrete, small-diameter high-pressure tires will give lower rolling resistance, while, for soft road surfaces, large-diameter low-pressure tires will give lower rolling resistance because the larger areas of contact will reduce the depth of penetration by the tires.

**EXAMPLE.** This example illustrates the effect which rolling resistance has on the cost of hauling earth.

A project requires a contractor to excavate and haul 1,900,000 cu yd of common earth. The contract must be completed within 1 year. By operating 3 shifts, with 7 hours actual working time per shift, 6 days per week, it is estimated that there will be 5,600 working hours, allowing for lost time due to bad weather. This will require an output of approximately 350 cu yd per hr bank measure, which should be obtained with a 4-cu-yd power shovel.

The job conditions are as follows:

- Length of haul, 1 way, 3.5 miles
- Slope of haul road, minus 0.5% from borrow pit to the fill
- Weight of earth in place, 2,600 lb per cu yd
- Swell, 30%
- Weight of loose earth,  $2,600 \div 1.3 = 2,000$  lb per cu yd
- Elevation, 800 ft above sea level

For hauling the earth the contractor considers using rubber-tire-equipped tractor-pulled bottom-dump wagons, which may be purchased with standard or optional

gears. The optional gears will permit the unit to operate at a higher speed. Specifications and performance data are as follows:

	Standard tractor	Optional tractor
Tractor engine.....	150 bhp	150 bhp
Max speed.....	19.8 mph	27.4 mph
Mechanical efficiency.....	82%	82%
Rimpull at max speed.....	2,330 lb	1,685 lb

Heaped capacity of standard wagon, 32,000 lb or 16 cu yd loose measure, based on 3:1 slope

Inside length of wagon, 14 ft 2 in.

Average inside width of wagon, 7 ft 1 in.

Heaped capacity of wagon with sideboard extensions, 2 ft 0 in. high, 46,800 lb, or 23.4 cu yd loose measure, based on 3:1 slope

	Standard equipment	Optional equipment
Gross weight:		
Tractor and wagon.....	29,400 lb	29,400 lb
Sideboards.....		1,600 lb
Pay load.....	32,000 lb	46,800 lb
Total weight.....	61,400 lb	77,800 lb
Gross weight, tons.....	30.7	38.9
Delivered cost.....	\$20,800	\$21,250
Cost per hr, including driver	\$8.20	\$8.80*

\* The higher cost per hour for the optional equipment is allowed because of the more severe conditions to which it will be subjected.

An analysis of the performance of the standard equipment, operating on a haul road with an estimated rolling resistance of 80 lb per ton, will give the probable hauling cost per cubic yard. This rolling resistance is representative of haul roads which are not carefully maintained.

The combined effect of rolling resistance and grade on a loaded unit will be

Rolling resistance =	80 lb per ton
Grade, $0.5 \times 20$ =	-10 lb per ton
Total =	70 lb per ton
Gross weight of vehicle, 30.7 tons	
Required rimpull, $30.7 \times 70$ =	2,149 lb
Available rimpull =	2,330 lb

The tractor can pull the loaded wagon, with a surplus rimpull for acceleration.

The rimpull required for the return trip to the shovel will be

$$14.7 \text{ tons} \times 90 \text{ lb per ton} = 1,323 \text{ lb}$$

which will permit travel at maximum speed.



The time required for each operation in a round-trip cycle should be about as follows:

Volume of earth per load, $16 \div 1.30$	$= 12.3$ cu yd bm
Loading, $12.3$ cu yd $\div 350$ cu yd per hr	$= 0.0351$ hr
Lost time in pit and accelerating, $1.5$ min	$= 0.0250$ hr
Travel to the fill, $3.5$ miles $\div 19.8$ mph	$= 0.1770$ hr
Dumping, turning, and accelerating, $1.0$ min	$= 0.0167$ hr
Travel to pit, $3.5$ miles $\div 19.8$ mph	$= 0.1770$ hr
Round-trip time	$= 0.4308$ hr

Assume that the wagons will operate an average of 45 min per hr

$$\text{No. trips per hr, } \frac{1}{0.4308} \times \frac{45}{60} = 1.74$$

$$\text{Volume of earth hauled per wagon, } 12.3 \times 1.74 = 21.4 \text{ cu yd per hr}$$

$$\text{No. wagons required, } 350 \div 21.4 = 16.4$$

It will be necessary to provide 17 wagons if the specified output is to be maintained. The actual volume of earth hauled per wagon will be  $350 \div 17 = 20.6$  cu yd per hr.

$$\text{Hauling cost per cu yd, } \$8.20 \div 20.6 = \$0.397$$

Let us analyze the performance of the optional equipment to determine whether it will operate at the maximum possible speed while hauling 23.4 cu yd loose measure. It will be necessary to reduce the rolling resistance of the haul road by providing continuous maintenance. While it is possible to reduce the rolling resistance to 40 lb per ton during most of the time the project is in operation, a value of 50 lb per ton will be used in order to provide a margin of safety.

The combined effect of rolling resistance and grade on a loaded unit will be

Rolling resistance	$= 50$ lb per ton
Grade, $0.5 \times 20$	$= -10$ lb per ton
Total	$= 40$ lb per ton
Gross weight of vehicle	$38.9$ tons
Required rimpull, $38.9 \times 40$	$= 1,556$ lb
Available rimpull at 27.4 mph	$= 1,685$ lb

The tractor can pull the load at the maximum speed, with a surplus for acceleration. The rimpull required for the return trip to the shovel will be

$$15.5 \text{ tons} \times 60 \text{ lb per ton} = 930 \text{ lb}$$

which will permit travel at maximum speed.

The time required for each operation in a round-trip cycle should be about as follows:

Volume of earth per load, $23.4 \div 1.30$	$= 18.0$ cu yd bm
Loading, $18$ cu yd $\div 350$ cu yd per hr	$= 0.0515$ hr
Lost time in pit and accelerating, $2$ min	$= 0.0333$ hr
Travel to the fill, $3.5$ miles $\div 27.4$ mph	$= 0.1277$ hr
Dumping, turning, and accelerating, $1.5$ min	$= 0.0250$ hr
Travel to pit, $3.5$ miles $\div 27.4$ mph	$= 0.1277$ hr
Round-trip time	$= 0.3652$ hr

Assume that the wagons will operate an average of 45 min per hr.

$$\text{No. trips per hr, } \frac{1}{0.3652} \times \frac{45}{60} = 2.05$$

Volume of earth hauled per wagon,  $18 \times 2.05 = 36.9$  cu yd per hr

$$\text{No. wagons required, } 350 \div 36.9 = 9.5$$

It will be necessary to provide 10 wagons if the specified output is to be maintained. The actual volume of earth hauled per hour per wagon will be  $350 \div 10 = 35$  cu yd.

$$\text{Hauling cost per cu yd, } \$8.80 \div 35 = \$0.251$$

The reduction in the cost of hauling the earth with the optional equipment will be

$$\text{Cost using standard equipment} = \$0.397 \text{ per cu yd}$$

$$\text{Cost using optional equipment} = 0.251 \text{ per cu yd}$$

$$\text{Reduction in cost} = \$0.146 \text{ per cu yd}$$

$$\text{Total reduction for project, } 1,900,000 \times \$0.146 = \$277,400$$

The reduction in the amount of money invested in hauling equipment will be

$$\text{Using standard equipment, } 17 \times \$20,800 = \$353,600$$

$$\text{Using optional equipment, } 10 \times \$21,250 = 212,500$$

$$\text{Reduction in investment} = \$141,100$$

The reduction in the cost of hauling earth and in the amount of money invested in hauling equipment resulting from the improvement in the rolling resistance of the haul road illustrates the value of analyzing a project. Although the reduction may appear to be unreasonably large, it is possible to produce similar results for many projects involving the hauling of earth. Even the cost of paving the haul road would be justified if this were the only method of reducing the rolling resistance.

Most manufacturers of trucks and tractor-pulled wagons can furnish units with standard or optional gears. For equipment already in service the standard gears may be replaced with optional gears at reasonable costs. Sideboards may be purchased from the equipment manufacturer, or they may be made locally in a machine shop.

**EXAMPLE.** The effect of rolling resistance on the performance of equipment and the cost of hauling earth is further illustrated in Table 7-4. The information given in the table is based on using the optional tractor-pulled wagons of the previous analysis, an output of 350 cu yd of earth per hour, bank measure, a one-way haul distance of 3.5 miles, and a level haul road. If the haul road is not level, similar information may be obtained by combining the effect of rolling resistance and grade.

The speeds and rimpulls of the hauling units are as follows:

Gear	Speed, mph	Rimpull, lb
1st	4.1	11,250
2d	6.5	7,120
3d	10.6	4,360
4th	17.0	2,720
5th	27.4	1,685



The following sample calculations will show how the information given in the table is obtained. Consider a haul road with a rolling resistance of 100 lb per ton.

Gross weight of loaded unit, 38.9 tons  
 Weight of empty unit, 15.5 tons  
 Required rimpull for loaded unit,  $38.9 \times 100 = 3,890$  lb  
 Max speed, 10.6 mph  
 Required rimpull for empty unit,  $15.5 \times 100 = 1,550$  lb  
 Max speed, 27.4 mph

The round-trip time will include fixed time, which should be reasonably constant regardless of the condition of the haul road, plus the travel time to and from the fill.

The fixed time will be

Loading, 18 cu yd  $\div$  350 cu yd per hr = 0.0515 hr  
 Lost time in pit and accelerating, 2 min = 0.0333 hr  
 Dumping, turning, and accelerating, 1.5 min = 0.0250 hr  
 Total fixed time = 0.1098 hr  
 Travel to the fill, 3.5 miles  $\div$  10.6 mph = 0.3310 hr  
 Travel to shovel, 3.5 miles  $\div$  27.4 mph = 0.1277 hr  
 Round-trip time = 0.5685 hr

Trips per 45-min hr,  $\frac{1}{0.5685} \times \frac{45}{60} = 1.32$

Volume per wagon,  $18 \times 1.32 = 23.75$  cu yd per hr

No. wagons required,  $350 \div 23.75 = 15$

Actual volume per wagon,  $350 \div 15 = 23.3$  cu yd per hr

Hauling cost per cu yd,  $\$8.80 \div 23.3 = \$0.378$

TABLE 7-4. THE EFFECT OF ROLLING RESISTANCE ON THE COST OF HAULING EARTH

	Rolling resistance, lb per ton			
	40	60	100	150
Max speed loaded, mph.....	27.4	17.0	10.6	6.5
Max speed empty, mph.....	27.4	27.4	27.4	17.0
No. units required.....	10	12	15	22
Hauling cost per cu yd.....	\$ 0.251	\$ 0.301	\$ 0.378	\$ 0.555
Investment in hauling equipment...	\$212,500	\$255,000	\$318,750	\$467,500

### The Effect of Altitude on the Performance of Hauling Equipment.

Contractors who have established satisfactory production rates for earth-hauling equipment at one altitude frequently find it desirable to bid a project located at a different altitude. Unless an adjustment is made for the performance of the equipment at the higher altitude, it is possible that a substantial error may be made in estimating the cost of hauling the earth. As previously discussed, the effect of altitude is to reduce the sea level power of a four-cycle internal-combustion engine by approximately 3 per cent for each additional 1,000 ft of altitude above 1,000 ft unless a

supercharger is installed on the engine. Power losses of this magnitude are too large to ignore in analyzing a project for bid purposes.

**EXAMPLE.** This example will illustrate the effect of altitude on the performance of hauling equipment and the cost of hauling earth. The hauling units are commonly used in the construction industry.

The job conditions are as follows:

Weight of earth, 2,700 lb per cu yd bm  
 Swell, 25%  
 Weight of loose earth,  $2,700 \div 1.25 = 2,160$  lb per cu yd  
 Haul distance, 1.5 miles, over level road  
 Rolling resistance, 50 lb per ton

The earth will be excavated with a power shovel, whose output will be 280 cu yd per hr.

The specifications for the hauling units are as follows:

Type, tractor-pulled bottom-dump wagons  
 Tractor engine, 200 bhp  
 Wagon capacity, 16 cu yd heaped volume  
 Wagon capacity,  $16 \div 1.25 = 12.8$  cu yd bm  
 Weight of tractor and wagon = 36,800 lb  
 Weight of load, 16 cu yd @ 2,160 lb = 34,560 lb  
 Gross loaded weight = 71,360 lb, or 35.68 tons  
 Cost per hr, including operator, \$9.40  
 Tractor-performance data at sea level:

Gear	Speed, mph	Rimpull, lb
1st	3.0	20,250
2d	5.8	10,450
3d	11.1	5,520
4th	19.4	3,130
5th	30.5	1,990

Compare the performance of a hauling unit at sea level with its performance at 5,000 ft above sea level, all other conditions remaining constant.

Performance at sea level:

Required rimpull for loaded unit,  $35.68 \times 50 = 1,784$  lb

Max speed loaded, 30.5 mph

Max speed empty, 30.5 mph

The probable round-trip time should be as follows:

Loading,  $12.8 \text{ cu yd} \div 280 \text{ cu yd per hr} = 0.0458 \text{ hr}$

Lost time in pit and accelerating, 1.5 min = 0.0250 hr

Travel to the fill, 1.5 miles  $\div$  30.5 mph = 0.0493 hr

Dumping, turning, and accelerating, 1.5 min = 0.0250 hr

Travel to pit, 1.5 miles  $\div$  30.5 mph = 0.0493 hr

Round-trip time = 0.1944 hr



Assume that units will operate an average of 45 min per hr

$$\text{No. trips per hr, } \frac{1}{0.1944} \times \frac{45}{60} = 3.86$$

$$\text{Volume per hr, } 12.8 \times 3.86 = 49.5 \text{ cu yd hr}$$

$$\text{No. units required, } 280 \div 49.5 = 5.7$$

It will be necessary to use 6 units

$$\text{Volume hauled per unit, } 280 \div 6 = 46.7 \text{ cu yd per hr}$$

$$\text{Hauling cost per cu yd, } \$9.40 \div 46.7 = \$0.201$$

Performance at 5,000-ft elevation:

$$\text{Loss in available rimpull, } \frac{0.03(5,000 - 1,000)}{1,000} \times 100 = 12\%$$

Correction factor for rimpull at 5,000 ft, 0.88

The available rimpull will be

Gear	Speed, mph	Rimpull at sea level, lb	Rimpull at 5,000 ft, lb
1st	3.0	20,250	17,820
2d	5.0	10,450	9,196
3d	11.1	5,250	4,620
4th	19.4	3,150	2,772
5th	30.5	1,990	1,751

Required rimpull for loaded unit, 1,784 lb

Max speed loaded, 19.4 mph

$$\text{Required rimpull empty, } 15.5 \times 50 = 775 \text{ lb}$$

Max speed empty, 30.5 mph

The probable round-trip time should be as follows:

$$\text{Loading, } 12.8 \text{ cu yd} \div 280 \text{ cu yd per hr} = 0.0458 \text{ hr}$$

$$\text{Lost time in pit and accelerating, } 1.75 \text{ min} = 0.0290 \text{ hr}$$

$$\text{Travel to the fill, } 1.5 \text{ miles} \div 19.4 \text{ mph} = 0.0773 \text{ hr}$$

$$\text{Dumping, turning, and accelerating, } 1.75 \text{ min} = 0.0290 \text{ hr}$$

$$\text{Travel to pit, } 1.5 \text{ miles} \div 30.5 \text{ mph} = 0.0493 \text{ hr}$$

$$\text{Round-trip time} = 0.2304 \text{ hr}$$

$$\text{No. trips per hr, } \frac{1}{0.2304} \times \frac{45}{60} = 3.25$$

$$\text{Volume per hr, } 12.8 \times 3.25 = 41.6 \text{ cu yd}$$

$$\text{No. units required, } 280 \div 41.6 = 6.7$$

It will be necessary to use 7 units

$$\text{Volume hauled per unit, } 280 \div 7 = 40 \text{ cu yd per hr}$$

$$\text{Hauling cost per cu yd, } \$9.40 \div 40 = \$0.235$$

In the calculations for the 5,000-ft altitude the time lost by a unit in the pit and at the dump was increased by 0.25 min to allow for the effect of the loss in power at this altitude.

### PROBLEMS

7-1. Use the basic information given in Table 7-1 to compare the costs of hauling common earth with various sizes of trucks employing a 1-cu-yd power shovel for excavating. Show your results in the same form as that used in the table.

**7-2.** Prepare a table similar to Table 7-2, but use trucks having a capacity of 10 cu yd bank measure. The trucks will cost \$7.20 per hour each, including the driver.

**7-3.** Prepare an analysis showing the relative economy of loading and hauling earth with several types of equipment. Information concerning the equipment to be considered is as follows:

a. Excavate with a  $2\frac{1}{2}$ -cu-yd power shovel whose average output will be 60 per cent of the ideal output. Haul the earth with bottom-dump wagons whose heaped capacity will be 12 cu yd. The effective speeds of the wagons will be 8 mph for the 1,000-ft haul, 11 mph for the 3,000-ft haul, and 12 mph for the 6,000-ft haul distance.

b. Use wheel tractors and 12-cu-yd scrapers, heaped capacity. The effective speed between the borrow pit and the dump will be 10 mph for all distances.

c. Use crawler tractors and 12-cu-yd scrapers, heaped capacity. The effective speed between the borrow pit and the dump will be 5 mph for all distances.

It will be necessary to use a bulldozer to help both types of tractors load the scrapers. Assume that a bulldozer can load a scraper every 2 min.

The job conditions are as follows: The earth is sandy clay. The haul distances are 1,000, 3,000, and 6,000 ft, one way.

a. For each wagon allow a total of 4 min in the borrow pit and 3 min at the dump.

b. For each wheel tractor allow 2 min in the borrow pit and 1 min at the dump.

c. For each crawler tractor allow  $1\frac{1}{2}$  min in the borrow pit and 1 min at the dump.

The cost of each type of equipment, including operators, is as follows:

Shovel, \$22.50 per hr

Wagon, \$8.00 per hr

Wheel tractor and scraper, \$9.50 per hr

Crawler tractor and scraper, \$9.00 per hr

Bulldozer, \$5.50 per hr

Prepare your results in tabular form, showing for each type of equipment and haul distance the volume of earth hauled per hour and the cost per cubic yard bank measure for loading and hauling.

**7-4.** A tractor, with a rimpull of 1,600 lb at a maximum speed of 25 mph, is used to pull a wagon whose heaped capacity is 20 cu yd. The weight of the tractor and wagon is 30,000 lb. This unit is used to haul earth whose bank-measure weight is 2,600 lb per cu yd and whose swell is 30 per cent. The rolling resistance of the haul road is 60 lb per ton.

If only 80 per cent of the rimpull is used, find the maximum load that can be hauled up a road with a slope of 2 per cent. Express the load in cubic yards bank measure.

If the slope of the haul road is changed to -2 per cent, find the maximum load that can be hauled, both theoretical and actual, expressed in cubic yards bank measure.

**7-5.** A wheel-type tractor has the following specifications:

a. Powered by a 4-cycle diesel engine

b. Operating weight, 24,000 lb

c. Maximum rimpull in top gear, at sea level, 8,600 lb

Find the maximum towing force that can be exerted by the tractor when it is pulling a trailer up a hill at a uniform speed, under the following conditions:

a. Rolling resistance of the haul road, 90 lb per ton

b. Slope of the haul road, 4 per cent

c. Altitude, 4,000 ft above sea level



## CHAPTER 8

### RUBBER TIRES

**General Information.** The development of larger units of construction and hauling equipment has produced a need for many different sizes and

types of rubber tires. Some types of equipment in use today require tire sizes up to  $36.00 \times 41$ , standing better than 9 ft high. Such tires, which are designed primarily to operate on unpaved surfaces, are known as off-the-road tires. Depending on the service for which they will be used and the design of the tread, they may be classed as earth-moving, rock service lug, sand, mining, and grader tires. Each is designed to give best performance for a particular type of service.

Specifications for tires and rims have been developed by the Tire and Rim Association, Akron, Ohio.

**Tire Dimensions and Nomenclature.** The dimensions of a tire indicate the nominal width of the section and the diameter of the rim, expressed in inches. Thus, a  $21.00 \times 24$  tire has a section width of 21 in. and is to be used on a rim whose diameter is 24 in. Figure 8-1 illustrates the nomenclature used in specifying the dimensions of tires, as prescribed by the Tire and Rim Association.

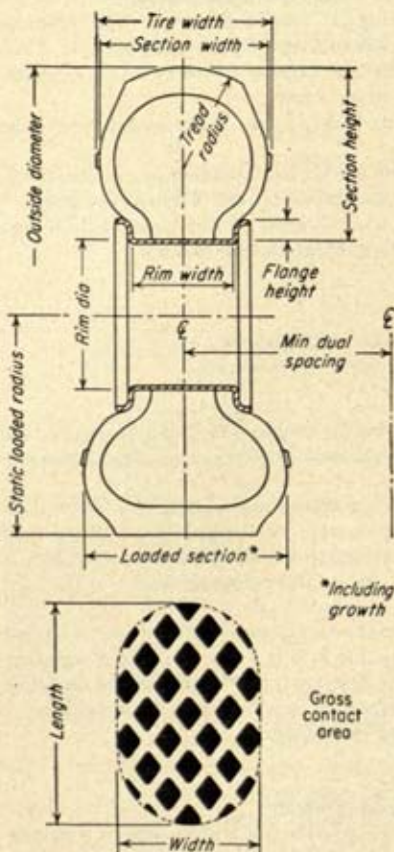


FIG. 8-1. Tire dimensions. (Goodyear Tire and Rubber Co.)

and other terms which apply to tires are as follows:

**Section Width.** This is the width of a new tire, including normal side walls; it does not include protective ribs, bars, and decorations.

*Over-all Tire Width.* This is the width of a new tire, including protective side ribs, bars, and decorations.

*Outside Diameter.* This is the greatest diametrical distance between the tread surfaces of a mounted, inflated, and unloaded tire and is measured in the same plane as the direction of travel.

*Static-load Radius.* This radius is the radial distance from the center of the axle, perpendicular to the surface on which a mounted, inflated, and loaded tire rests.

*Tread Radius.* This is the radius of curvature of the tread arc of a mounted, inflated, and unloaded tire.

*Loaded Section.* This is the over-all width of a mounted, loaded, and inflated new tire, measured at the widest section.

*Rim Diameter.* Rim diameter is the nominal full inch diameter, measured at the outside of the rim base adjacent to the flange. This applies to both flat base and tapered bead rims.

*Rim Width.* This width is measured between the inside parallel surfaces of rim flanges.

*Section Height.* This is the distance from the outside of the rim base, adjacent to the flange, to the outside diameter of the mounted and inflated tire.

*Gross Contact Area.* This is the area enclosed by the outer periphery of the tread pattern of a mounted, inflated, and loaded tire in contact with a supporting rigid plane surface.

*Deflection.* Deflection is the distance that a mounted, inflated, and loaded tire is depressed toward the axle, where it is in contact with the supporting surface.

*Ply Rating.* This is a term used to identify a given tire with its maximum recommended load when used in a specific type of service. It is an index of tire strength and does not necessarily represent the number of cord plies in the tire.

*Inflation Pressure.* As used in tables for tires, this should be taken when the tire is at atmospheric temperature. After the vehicle has been in operation, the pressure will increase owing to the heat generated in the tire.

**Selecting the Proper Tread Design.** Experience gained from studies of the performance of tires on various types of equipment has disclosed that certain tread designs perform best for each type of service.

For use on the driving wheels of earth-moving equipment, which frequently must operate on soft ground, the self-cleaning directional-bar-type tire gives maximum traction. This tire is illustrated in Fig. 8-2.

For use on the free-rolling or trailing wheels of earth-moving equipment the button-type tire, illustrated in Fig. 8-3, is satisfactory.

For use on equipment which must operate over rocky roads or other



hazardous terrain, a tire that will withstand severe abuse, such as the hard lug rock tire, illustrated in Fig. 8-4, should be used.

Other types of tires are available for use on equipment that operates under varying conditions. The catalogues of the tire manufacturers give the information needed to permit a purchaser to obtain the most desirable type tire for any given service.

**Selecting the Most Suitable Size Tire.** A tire performs either two or three services, depending on whether it is a free-rolling or a driving tire. If it is a free-rolling tire, it supports a load and provides a low rolling resistance for the vehicle. If it is a driving tire, it also provides traction for the vehicle.

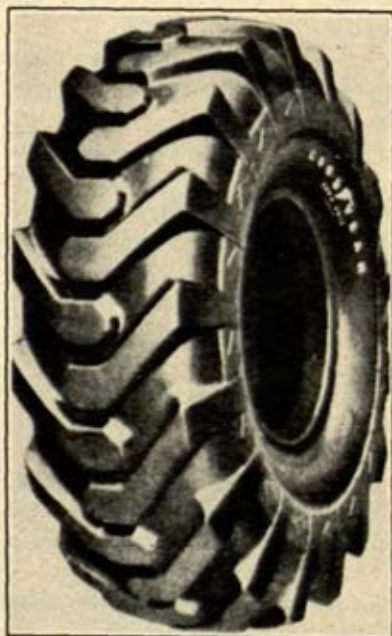


FIG. 8-2. Self-cleaning directional-bar-type tire. (Goodyear Tire and Rubber Co.)

The load which a tire can support varies with the size of the tire, the ply rating, the inflated pressure, and the speed at which it travels. A tire supports a load by deflecting until the contact area between the tire and the supporting surface is large enough to produce a total pressure force equal to the load. For example, if the load is 10,000 lb and the inflated pressure is 100 psi, the contact area will be 100 sq in., neglecting any load supported by the side walls of the tire. If the pressure is reduced to 50 psi, the tire will deflect until the contact area is 200 sq in. If the surface of a haul road is soft, the necessary contact area may be produced by the combined deflection of the tire and the penetration of the tire into the surface.

A term which is frequently applied to a tire is flotation. This term is a measure of the ability of a tire to move over a soft surface without excessive penetration. Large-size, low-pressure tires provide better flotation than small, high-pressure tires when operating on soft surfaces, because of the large contact area which is obtained without excessive penetration.

When a tire is operated on a hard surface, which offers high resistance to penetration, a high-pressure tire will usually give lower rolling resistance than a low-pressure tire. When a tire is operated on a soft surface,

the low-pressure tire will give lower rolling resistance. Thus, the type of haul surface should be considered in selecting tires.

The speed of the vehicle on which a tire will be used will influence the selection of a tire. When a tire travels long distances at relatively high speeds, the effect of the frequent deflecting of the side walls generates high temperatures, which are detrimental to the tire. Also, the high torque and more severe braking and impact effects demand a tire with a stronger carcass than is required for a tire that operates at lower speeds.

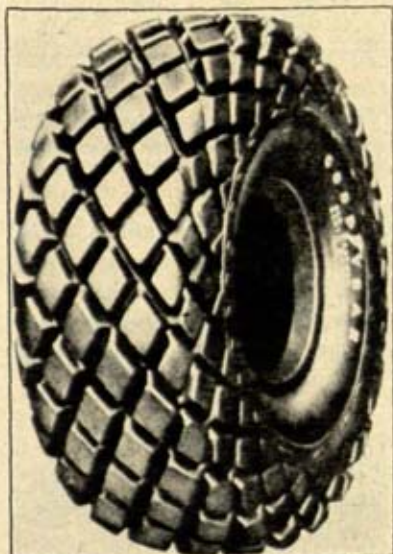


FIG. 8-3. Button-type tire. (Goodyear Tire and Rubber Co.)

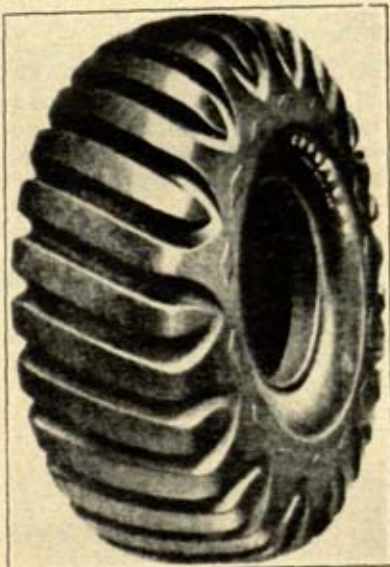


FIG. 8-4. Hard-lug rock tire. (Goodyear Tire and Rubber Co.)

**The Carrying Capacities of Tires.** The Tire and Rim Association publishes a yearbook which gives the recommended capacities of various types and sizes of tires. This information is available through the tire manufacturers. Table 8-1 gives representative capacities for tires operated at a maximum speed of 25 mph. If the tires are operated at a maximum speed of 10 mph, the capacities may be increased by approximately 12 per cent.

**EXAMPLE.** The information given in Table 8-1 may be used to determine the most suitable size tires for any given vehicle. Consider a four-wheel tractor-pulled wagon for off-road hauling service. The manufacturer gives the following distribution of weight for the loaded vehicle:

Front tires, 6,400 lb each  
Driving tires, 16,600 lb each  
Trailer tires, 16,600 lb each  
The maximum speed is 24.6 mph



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For the front tires select 12.00 × 24, 14-ply, inflated to 55 psi. For the driving and trailer tires, select one of the following tires:

Tire size	Ply rating	Inflation pressure, psi	Over-all diameter, in.	Max load, lb	Rev. per mile	Gross contact area, sq in.	Weight of casing, lb
21.00 × 24	24	50	68.0	17,630	326	334	771
21.00 × 29	20	40	72.0	16,830	306	336	771
24.00 × 25	18	35	73.5	16,900	301	467	915

If the haul roads are well maintained, the 21.00 × 24 tire, which will safely support a higher load, should be selected. However, if the haul roads are poor, the 24.00 × 25 tire, which provides greater contact area and flotation, should be selected, even though the cost will be higher than for either of the other two tires.

**The Effect of the Size Tire on the Hauling Capacity of a Vehicle.** The maximum hauling capacity of equipment, such as a four-wheel tractor-pulled self-loading scraper, may be limited by the capacity of the tires. An analysis of the possible performance of a popular unit, based on using two tire sizes, reveals some interesting information. The analysis is as follows:

	Tire size		Per cent increase
	21.00 × 29 20-ply	24.00 × 29 24-ply	
Gross contact area, per tire, sq in.....	376	444	18.1
Over-all diameter, in.....	71.7	78	8.8
Rpm.....	306	282	— 7.8
Speed, 4th gear, mph.....	18.0	19.4	7.8
Weight of tire, tube, and flap, lb.....	853	1,209	41.7
Weight of 4 tires, etc., lb.....	3,412	4,836	41.7
Inflation pressure, psi.....	40	45	11.3
Approximate cost of 1 tire and tube.....	\$868.00*	1275.00*	46.8
Rated capacity, 1 tire, lb.....	16,830	21,300	25.6
Rated capacity, 4 tires, lb.....	67,320	85,200	25.6
Weight of hauling unit, empty, lb.....	45,000	46,424†	3.2
Net pay-load capacity, lb.....	22,320	38,776	74.0

\* Based on the wholesale price, including tax.

† Includes the extra weight of the larger tires.

The previous analysis indicates that the use of the larger-size tires will permit an increase in the hauling capacity of the vehicle by approximately

TABLE 8-1. REPRESENTATIVE CAPACITIES OF TIRES FOR EARTH-MOVING SERVICE  
(Maximum speed 25 mph)

Tire size	Ply rating	Tire loads at various inflation pressures, psi								
		25	30	35	40	45	50	55	60	65
8.25 × 20	10	.....	2,390	2,620	2,820	3,030	3,220	3,400		
8.25 × 20	12	.....	2,390	2,620	2,820	3,030	3,220	3,400	3,580	3,750
9.00 × 20	10	.....	2,840	3,100	3,360	3,590	3,830			
9.00 × 20	12	.....	2,840	3,100	3,360	3,590	3,830	4,050	4,250	
10.00 × 20	12	.....	3,200	3,500	3,780	4,050	4,310	4,560		
10.00 × 20	14	.....	3,200	3,500	3,780	4,050	4,310	4,560	4,780	5,020
12.00 × 20	12	.....	4,020	4,390	4,750	5,080				
12.00 × 20	14	.....	4,020	4,390	4,750	5,080	5,400	5,740		
12.00 × 24	14	.....	4,520	4,930	5,330	5,720	6,080	6,450		
12.00 × 24	16	.....	4,520	4,930	5,330	5,720	6,080	6,450	6,780	
14.00 × 20	12	5,040	5,620	6,140	6,630					
14.00 × 20	16	5,040	5,620	6,140	6,630	7,100	7,550	8,000		
14.00 × 24	16	5,630	6,270	6,850	7,400	7,920	8,430	8,920		
14.00 × 24	20	5,630	6,270	6,850	7,400	7,920	8,430	8,920	9,400	9,830
16.00 × 24	16	7,070	7,880	8,610	9,330	9,980				
16.00 × 24	20	7,070	7,880	8,610	9,330	9,980	10,620	11,250	11,820	
18.00 × 24	16	9,200	10,230	11,200	12,130	13,020	13,800			
18.00 × 24	20	9,200	10,230	11,200	12,130	13,020	13,800	14,630	15,360	
18.00 × 24	24	9,200	10,230	11,200	12,130	13,020	13,800	14,630	15,360	
21.00 × 24	16	11,770	13,110							
21.00 × 24	20	11,770	13,110	14,310	15,490					
21.00 × 24	24	11,770	13,110	14,310	15,490	16,570	17,630			
21.00 × 29	20	12,800	14,260	15,570	16,830					
21.00 × 29	24	12,800	14,260	15,570	16,830	18,030	19,170			
24.00 × 25	18	13,960	15,560	17,000						
24.00 × 25	24	13,960	15,560	17,000	18,380	19,700				
24.00 × 29	24	15,070	18,330	19,890	21,300					
24.00 × 29	36	15,070	18,330	19,890	21,300	22,600	24,000	25,200	26,400	
24.00 × 32	24	16,220	18,080	19,720	21,400	22,900				
27.00 × 33	24	20,960	23,360	25,530	27,600					
27.00 × 33	30	20,960	23,360	25,530	27,600	29,600	31,450			
27.00 × 33	36	20,960	23,360	25,530	27,600	29,600	31,450	33,270	34,990	
30.00 × 33	28	25,700	28,600	31,200						
30.00 × 33	34	25,700	28,600	31,200	33,800	36,200				

74 per cent if the speed can be maintained with the heavier load. This provides an opportunity to increase the output of earth-moving equipment, where job conditions permit the use of larger tires, that may be overlooked by a contractor. It emphasizes the importance of applying engineering analysis to a job.

**The Effect of Inflation on Tire Performance.** When a tire is placed in service, the owner wishes it to provide good flotation and traction and



high mileage. Proper control of the inflation pressure will have a direct effect on each of these performances. In general, decreasing the pressure in a tire will increase the flotation and tractive efficiency. The increase in tractive efficiency seems to be primarily the result of lower rolling resistance, especially in hauling over soft surfaces. The lower penetration resulting from the use of lower pressure causes the reduction in rolling resistance.

Although the operation of a tire at a reduced pressure may be justified under some conditions, such a practice will reduce the normal life of the

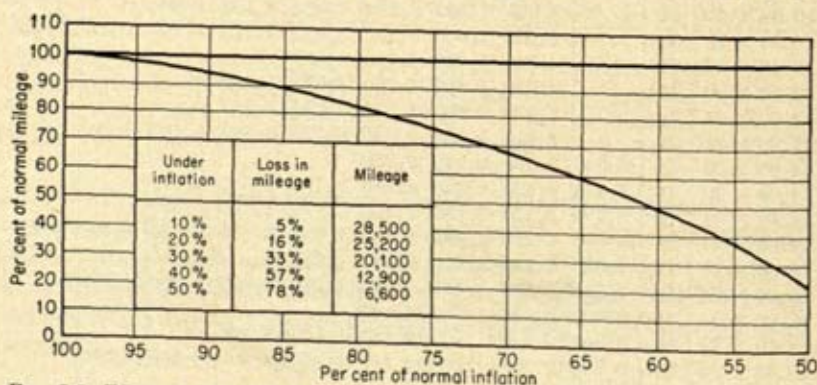


FIG. 8-5. Effect of inflation on tire performance. (Goodyear Tire and Rubber Co.)

tire, measured in miles of service. Low pressure will increase the flexing of the side walls, which will increase the heat generated in a tire and reduce its life. Figure 8-5 illustrates the effect which inflation pressure has on the normal mileage of a tire.

**The Effect of the Load on Tire Performance.** Overloading a tire causes rapid and uneven tread wear, increases flexing, and generates excessive heat in the tire. As the effect of overloading is more pronounced at high than at low speeds, overloaded vehicles should be operated at lower speeds where possible. Figure 8-6 illustrates the effect of load on tire performance.

**The Effect of Speed on Tire Performance.** Although information on the effect of speed on the performance of off-the-road tires is difficult to correlate because of the many other factors which affect the performance of such tires, tests made on automobile tires indicate that the mileage obtained at high speeds is considerably less than for lower speeds. It is reasonable to assume that the results of tests made on the performance of automobile tires may at least be indicative of the performance of tires used on construction equipment.

Figure 8-7 gives the results of tests performed by the Public Roads

Administration, for  $6.00 \times 16$  tires operated on concrete highways. The performance of tires operated on other surfaces will not necessarily be the same.

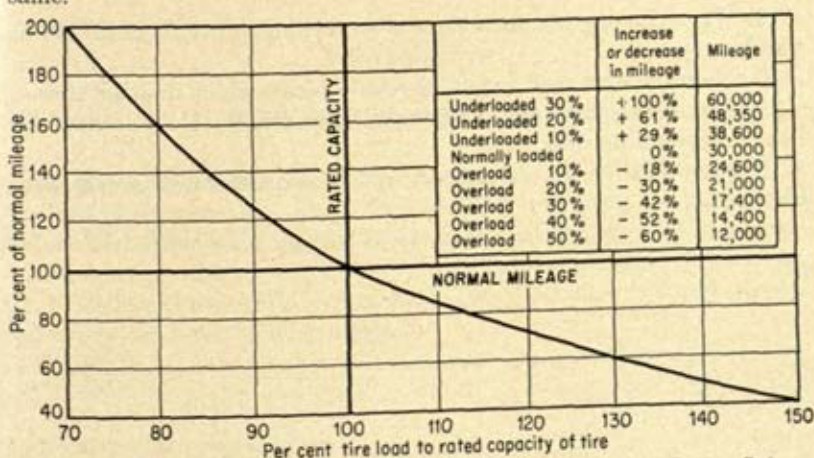


FIG. 8-6. Effect of load on tire performance. (Goodyear Tire and Rubber Co.)

**Suggestions for Increasing the Mileage Obtained from Tires.** The investment in tires used on a unit of construction equipment frequently amounts to several thousand dollars. Such an investment justifies a definite program designed to provide the greatest possible mileage from the tires. Several practices can be adopted to increase the mileage. Among them are the following:

1. Select tires that will safely carry the load at the recommended pressure.

2. Keep the tires inflated to the correct pressure.

3. Operate a vehicle at a lower speed when it is necessary to overload.

4. Maintain correct alignment of wheels.

5. Keep the brakes adjusted to provide equalized braking on the tires.

6. Inspect tires frequently, and have minor breaks and cuts repaired before the fabric is damaged.

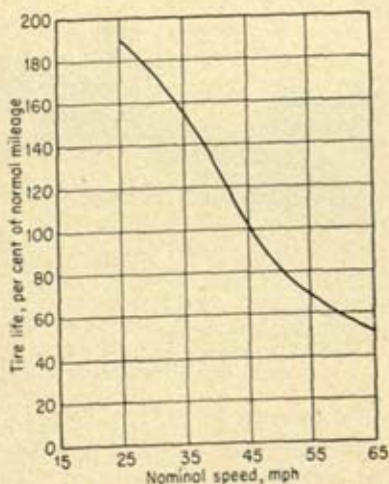


FIG. 8-7. Effect of speed on tire performance.



7. Have major breaks and cuts vulcanized as quickly as possible.
8. When operating dual tires on an axle, use tires having the same over-all diameter.
9. When driving tires show signs of weakening, move them to trailing wheels.
10. Keep the haul road free of physical objects which damage tires.
11. Sprinkle the haul road frequently, when practicable, to reduce the temperature of the tires.
12. Have new treads vulcanized on tire carcasses that are in good condition.

## CHAPTER 9

### BELT-CONVEYOR SYSTEMS

**General Information.** Belt-conveyor systems are used extensively in the field of construction, where they frequently provide the most satisfactory and economical method of handling and transporting materials, such as earth, sand, gravel, crushed stone, mine ores, cement, concrete, etc. Because of the continuous flow of materials at relatively high speeds, belt conveyors have high capacities.

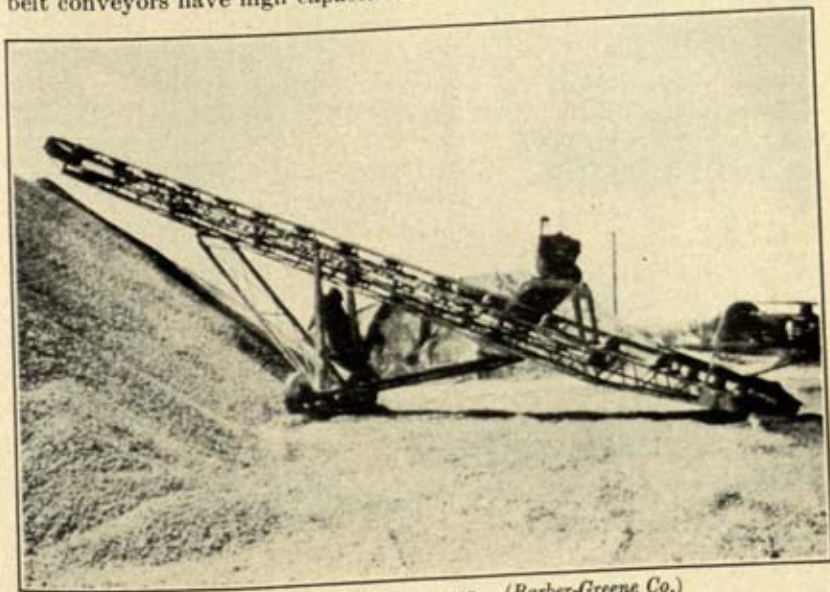


FIG. 9-1. Portable belt conveyor. (Barber-Greene Co.)

The essential parts of a belt-conveyor system include a continuous belt, idlers, a driving unit, driving and tail pulleys, take-up equipment, and a supporting structure. Additional accessories, as described later, may be included when desirable or necessary.

A conveyor for transporting materials a short distance may be a portable unit or a fixed installation. Figure 9-1 illustrates a portable conveyor used to stock-pile aggregate which is delivered by trucks. This machine is available in lengths of 33 to 60 ft, with belt widths of 18, 24,



and 30 in. It is self-powered with a gasoline-engine drive through a shaft and gearbox to the driving pulley. The operating features include swivel wheels, V-type truck, hydraulic hoist, low-mast height, and anti-friction bearings throughout.

When a belt-conveyor system is used to transport materials a considerable distance, up to several miles in some instances, the system should consist of a number of different flights, as there is a limit to the maximum length of a belt. Each flight is a complete conveyor unit which dis-

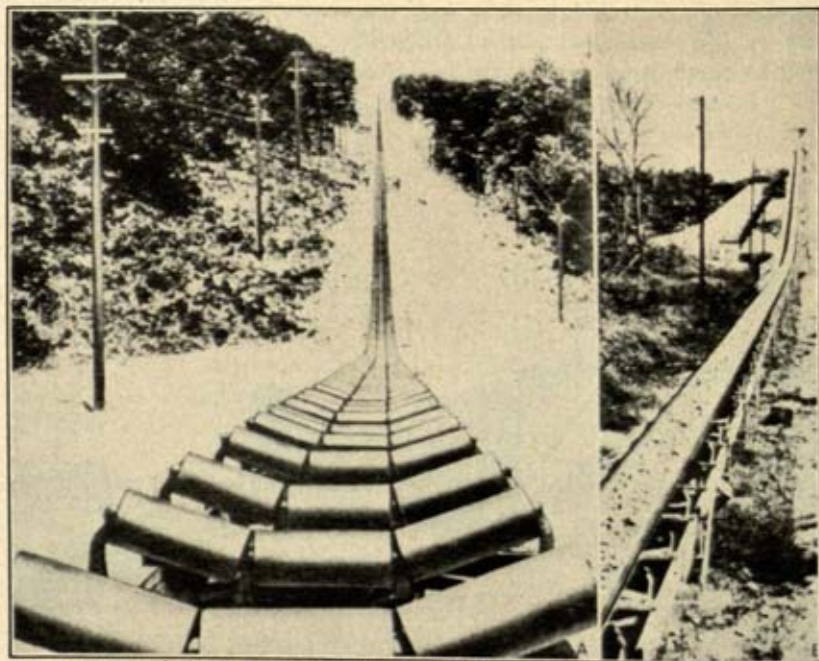


FIG. 9-2. (A) Troughing idlers installed. (B) Belt transporting aggregate. (Hewitt-Robbins, Inc.)

charges its load onto the tail end of the succeeding unit. Such a system will operate over any terrain provided the slopes do not exceed those for which the given material may be transported.

The limestone rock for the Bull Shoals Dam, whose maximum size was 6 in., was transported 7 miles from the primary crushing plant at the quarry to the dam site. The conveyor system consisted of 21 flights, varying in length from 600 to 2,800 ft, each powered with a 100-hp electric motor. The belts, which were 30 in. wide, were operated at a speed of 525 fpm to deliver 350 cu yd of material per hour. The entire system

required 14,000 idlers, which were supported primarily by wood structures [1].<sup>1</sup>

**The Economy of Transporting Materials with a Belt Conveyor.** One of the first questions that arises in considering the use of a belt conveyor is whether this method of transportation is the most dependable and economical when compared with other methods. The proper way to answer this question is to estimate the cost of transporting the material by each method under consideration. Assume that a belt conveyor is to be compared with trucks for hauling aggregate for a large concrete project.

The net total cost of the conveyor system will include the installed cost of the system, an access road for installing and servicing the system, maintenance, replacements, and repairs, fuel, or electrical energy, and labor, less the net salvage value of the system upon completion of its use. Interest on the investment, plus taxes and insurance, if they apply, should be included. Likewise, any cost of obtaining a right of way for the system should be included. The unit cost of moving the material, per ton or cubic yard, may be obtained by dividing the net total cost of the system by the number of units to be transported.

The cost of transporting the materials by truck will include the cost of constructing and maintaining a haul road, plus the cost of operating the trucks. The unit cost of moving the materials may be obtained by dividing the net total cost by the number of units to be transported.

If either method requires additional handling costs at the source or at the destination, these costs should be included prior to determining the unit cost of moving the materials.

In constructing the Bull Shoals Dam more than 4,500,000 tons of aggregate was transported on belt conveyors at a reported cost of \$0.045 per ton-mile. It was estimated that the contractors saved \$560,000 on the purchase and installation of the conveyor system compared with a fleet of trucks, plus a haul road and incidentals required for the trucks. In addition, it was estimated that there was a saving of \$375,000 on labor operating the system compared with trucks [1].

**Conveyor Belts.** The belt is the moving and supporting surface on which the material is transported. Many types, sizes, and grades are available, from which the most suitable belt for a given service may be selected.

Belts are manufactured by joining several layers or plies of woven cotton duck into a carcass which provides the necessary strength to resist the tension in the belt. The layers are covered with an adhesive which combines them into a unified structure. Special types of reinforcing, such as rayon, nylon, and steel cables, are employed sometimes to

<sup>1</sup> The bracketed figures refer to the Bibliography appearing at the end of some chapters.



increase the strength of a belt. A measure of the strength of a belt is indicated by the number and weight of the several layers of fabric. The number of layers is expressed as 4-, 6-, 7-, 8-, etc., ply. The weight of each layer of fabric is expressed as 28-, 32-, 36-, 42-, etc., oz, the number indicating the weight of a piece of duck 42 in. wide and 36 in. long. The width of a belt is expressed in inches. Thus, a belt might be specified as a 36-in.-wide 6-ply 42-oz belt.

The top and bottom surfaces of a belt are covered with rubber to protect the carcass from abrasion and injury from the impact at loading. Various thicknesses of covers may be specified. Figure 9-3 illustrates cross sections of belts having different types of construction.

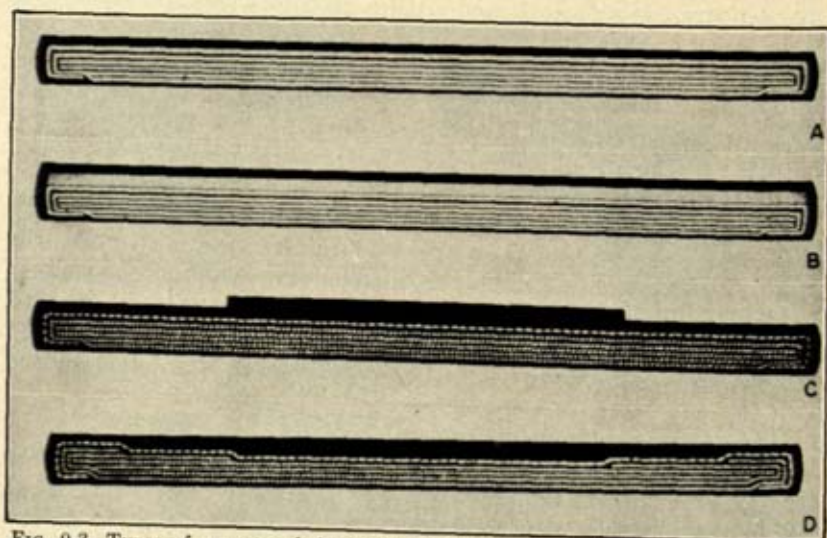


Fig. 9-3. Types of conveyor-belt construction: (A) standard; (B) shock pad; (C) stepped pad; (D) stepped ply. (Hewitt-Robins, Inc.)

It is necessary to select a belt with sufficient strength to resist the maximum tension to which it will be subjected, as determined by methods which will be developed later.

Also, it is necessary to select a belt that is wide enough to transport the material at the required rate. Most belts used on construction projects travel over troughing rollers to increase the carrying capacities. The number of tons that can be transported in an hour will equal the product of the cross-section area of the material in square feet times the belt speed in feet per hour times the weight of the material in pounds per cubic foot divided by 2,000 lb per ton. The area of the cross section will depend on the width of the belt, the depth of troughing, the angle of repose for the material, and the extent to which the belt is loaded to capacity. Figure

9-4 illustrates how the cross-section area may vary with the width of a belt and the angle of repose for the material. In the figure the troughing idlers are set at an angle of  $20^\circ$  above the horizontal. In order to eliminate side spillage, it is assumed that materials will not be placed closer than  $0.05W + 1$  in. from the sides of the belt, where  $W$  is the width of the belt in inches. It is assumed that the top surface of the material will be an arc of a circle. Table 9-1 gives the cross-section areas for various belt widths and loading conditions. These areas are subject to variation and should not be considered as exact unless the loading conditions are as stated. The area of surcharge is the area above line  $B$  of Fig. 9-4.

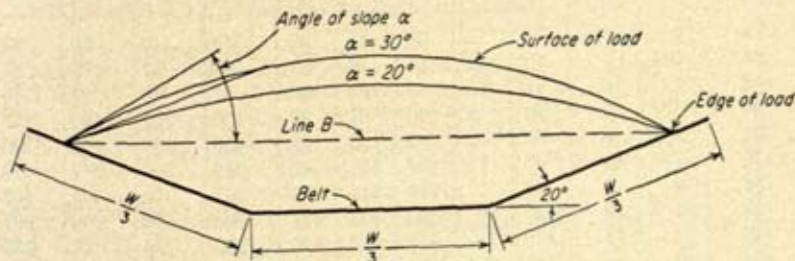


FIG. 9-4. Cross-section area of a load on a conveyor belt.

TABLE 9-1. AREAS OF CROSS SECTIONS OF MATERIALS FOR LOADED BELTS

Width of belt, in.	$0.05W + 1$ , in.	Area of level load, sq ft	Area of surcharge, sq ft, for angle of repose, deg			Total area, sq ft, for angle of repose, deg		
			10	20	30	10	20	30
16	1.8	0.072	0.029	0.059	0.090	0.101	0.131	0.162
18	1.9	0.096	0.038	0.078	0.118	0.134	0.174	0.214
20	2.0	0.122	0.048	0.098	0.150	0.170	0.220	0.272
24	2.2	0.185	0.072	0.146	0.225	0.257	0.331	0.410
30	2.5	0.303	0.118	0.238	0.365	0.421	0.541	0.668
36	2.8	0.450	0.174	0.351	0.540	0.624	0.801	0.990
42	3.1	0.627	0.241	0.488	0.749	0.868	1.115	1.376
48	3.4	0.833	0.321	0.649	0.992	1.154	1.482	1.825
54	3.7	1.068	0.408	0.826	1.264	1.476	1.894	2.332
60	4.0	1.333	0.510	1.027	1.575	1.843	2.360	2.908

The carrying capacity of a 42-in. belt, moving 100 fpm, loaded with sand weighing 100 lb per cu ft, with a  $20^\circ$  angle of repose, will be  $100 \text{ fpm} \times 100 \text{ lb} \times 1.115 \text{ sq ft} \times 60 \text{ min} \div 2,000 \text{ lb per ton} = 334.5$  tons per hr. The carrying capacity of this belt for other speeds may be obtained by multiplying 334.5 by the ratio of the speed divided by 1.00.



Table 9-2 gives the approximate carrying capacities of troughed conveyor belts, in tons per hour, for various widths and materials for a speed of 100 fpm. Table 9-3 gives the suggested maximum speeds which are

TABLE 9-2. CARRYING CAPACITIES OF TROUGHED CONVEYOR BELTS, IN TONS PER HOUR FOR A SPEED OF 100 FPM\*

Width of belt, in.	Max lumps		Weight of material, lb per cu ft								
	Sized, in.	Un- sized, in.									
			30	50	90	100	125	150	160	180	200
14	2	2½	9	15	28	31	39	46	49	56	62
16	2½	3	13	21	38	42	52	63	67	75	83
18	3	4	16	27	48	54	67	81	86	97	107
20	3½	5	20	33	60	67	83	100	107	120	133
24	4½	8	30	50	90	100	125	150	160	180	200
30	7	14	47	79	142	158	197	236	252	284	315
36	9	18	70	117	210	234	292	351	374	421	467
42	11	20	100	167	300	333	417	500	534	600	667
48	14	24	138	230	414	460	575	690	736	828	920
54	15	28	178	297	534	593	741	890	948	1,070	1,190
60	16	30	222	369	664	738	922	1,110	1,180	1,330	1,480

\* Courtesy Hewitt-Robins, Inc.

considered good practice for conveyor belts of different widths when handling various kinds of materials. Table 9-4 gives representative allowable working tensions in duck belts for various thicknesses and widths. The pulley diameter is the minimum size that should be used for the indicated service.

TABLE 9-3. MAXIMUM SPEEDS OF CONVEYOR BELTS, IN FPM\*

Kind and condition of material handled	Width of belt, in.										
	14	16	18	20	24	30	36	42	48	54	60
Unsize coal, gravel, stone, ashes, ore, or similar material.....	300	300	350	350	400	450	500	550	600	600	600
Sized coal, coke, or other breakable material.....	250	250	250	300	300	350	350	400	400	400	400
Wet or dry sand.....	400	400	500	600	600	700	800	800	800	800	800
Crushed coke, crushed slag, or other fine abrasive material.....	250	250	300	400	400	500	500	500	500	500	500
Large lump ore, rock, slag, or other large abrasive material.....					350	350	400	400	400	400	400

\* Courtesy of Hewitt-Robins, Inc.

TABLE 9-4. ALLOWABLE WORKING TENSION AND PULLEY DIAMETER FOR CONVEYOR BELTS\*

No. plies	Weight per ply, oz	Width of belt, in.								Diameter of pulley, in.		
		16	18	20	24	30	36	42	48	Head, drive, tripper	Tail, take-up, snub	Bend
3	32	1,440	1,620							16	12	12
3	36			1,800	2,160					20	16	12
3	42			2,200	2,640	3,300				20	16	12
3	48				3,840					24	20	16
4	28	1,600	1,800	2,000	2,400	3,000				20	16	12
4	32	1,920	2,160	2,400	2,880	3,600	4,320			20	16	12
4	36			2,600	3,120	3,900	4,680			24	20	16
4	42					4,800	5,760	6,720		24	20	20
4	48					6,450	7,750	9,020		30	24	20
5	28	2,000	2,250	2,500	3,000	3,750	4,500			24	20	16
5	32		2,700	3,000	3,480	4,500	5,400			24	20	16
5	36			3,400	4,080	5,100	6,120	7,140		30	24	20
5	42					6,600	7,920	9,240	10,560	30	24	20
5	48					8,700	10,400	12,180	13,920	36	30	24
6	28			3,000	3,600	4,500	5,400			30	24	20
6	32				4,320	5,400	6,480	7,560		30	24	20
6	36					6,300	7,560	8,820	10,080	36	30	24
6	42						9,720	11,340	12,900	36	30	24
6	48						13,000	15,120	17,300	42	36	30
7	28					5,250	6,300			36	30	24
7	32					6,300	7,560	8,820	10,080	36	30	24
7	36						8,820	10,300	11,780	42	36	30
7	42							13,200	15,140	42	36	30
7	48							17,640	20,180	48	42	36
8	32						8,640	10,080	11,520	42	30	24
8	36							11,760	13,450	48	42	30
8	42								17,300	48	42	30
8	48								23,050	54	48	42
9	32							11,340	12,900	48	36	30
9	36							13,200	15,140	54	48	36

\* Courtesy Hewitt-Robins, Inc.

**Idlers.** Idlers provide the supports for a belt conveyor. For the load-carrying portion of a belt the idlers are designed to provide the necessary troughing, while for the return portion of a belt the idlers provide flat supports. The essential parts of a troughing idler include the rolls, brackets, and base. Antifriction bearings are generally used in idlers, with high-pressure grease fittings to permit periodic lubrication of the bearings. The rolls may be made of steel tubing or cast iron, either plain or covered with a composition, such as rubber, where it is necessary to protect a belt against damage due to impact. The diameters of the rolls most commonly used are 4, 5, 6, and 7 in. Large-diameter rolls give lower friction and better belt protection, especially when the load includes large lumps of material.

**Spacing of Idlers.** Troughing idlers should be spaced close enough to



TABLE 9-5. RECOMMENDED MAXIMUM SPACING OF TROUGHING IDLERS\*

Width of belt, in.	Weight of material, lb per cu ft		
	30-70	70-120	120-150
14	5 ft 6 in.	5 ft 0 in.	4 ft 9 in.
16	5 ft 6 in.	5 ft 0 in.	4 ft 9 in.
18	5 ft 6 in.	5 ft 0 in.	4 ft 9 in.
20	5 ft 6 in.	5 ft 0 in.	4 ft 9 in.
24	5 ft 6 in.	5 ft 0 in.	4 ft 9 in.
30	5 ft 0 in.	4 ft 6 in.	4 ft 3 in.
36	5 ft 0 in.	4 ft 6 in.	4 ft 3 in.
42	4 ft 6 in.	4 ft 0 in.	3 ft 9 in.
48	4 ft 0 in.	3 ft 3 in.	3 ft 0 in.
54	4 ft 0 in.	2 ft 9 in.	2 ft 6 in.
60	4 ft 0 in.	2 ft 3 in.	2 ft 0 in.

\* Courtesy Hewitt-Robins, Inc.

prevent excessive deflection of the loaded belt between the idlers. As indicated in Table 9-5, the maximum spacing will vary with the width of the belt and the weight of the load carried. The idler spacing should be reduced at the point where the load is fed onto the belt.

As the sole function of the return idlers is to support the empty belt, the spacing can be increased to approximately 10 ft.

**Training Idlers.** Sometimes a conveyor belt is operated under conditions which make it difficult to keep the belt centered on the troughing

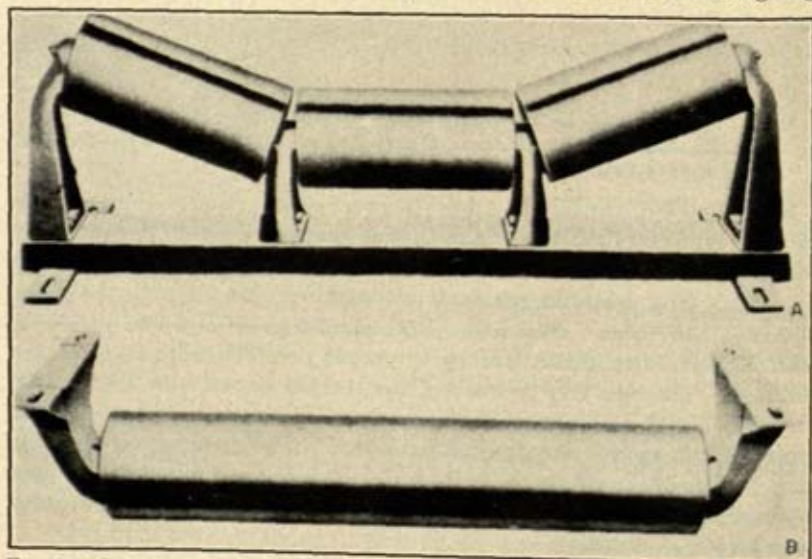


FIG. 9-5. Belt idlers: (A) heavy-duty troughing; (B) return. (Hewitt-Robins, Inc.)

idlers. If the conditions cannot be corrected sufficiently to keep the belt centered, it may be necessary to install training idlers, spaced 50 to 60 ft apart. Figure 9-6 illustrates a set of training idlers.

**Idler Friction.** In analyzing a belt conveyor to determine the horsepower required, it is necessary to include the power needed by the idlers. This power will depend on the type and size idler, the kind of bearings, the weight of the revolving parts, the weight of the belt, and the weight of the load. Table 9-6 gives representative friction factors for idlers

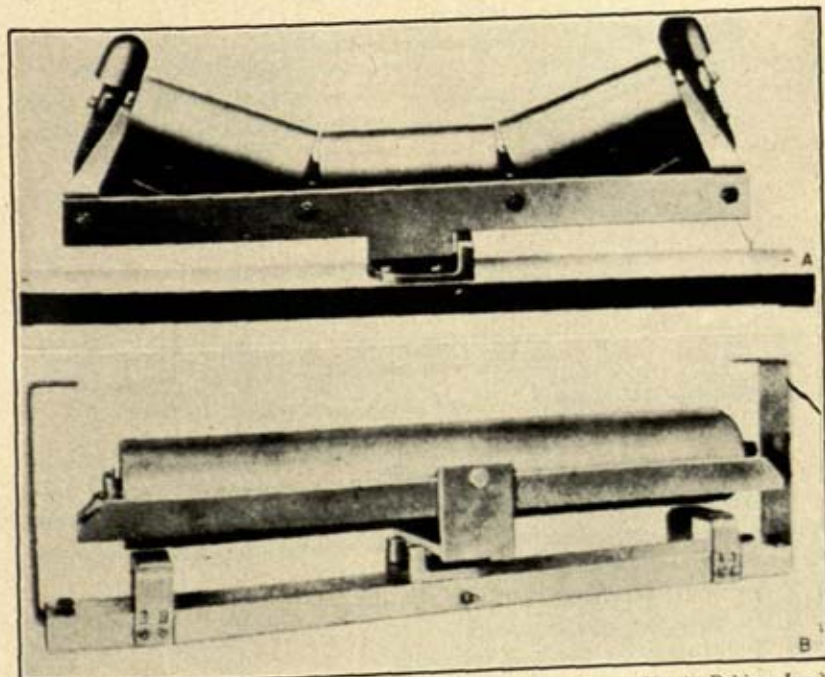


FIG. 9-6. Training idlers: (A) reversible troughing; (B) return. (Hewitt-Robins, Inc.)

equipped with antifriction bearings. Manufacturers of idlers will furnish information giving the weights of the revolving parts of their idlers.

TABLE 9-6. FRICTION FACTORS FOR CONVEYOR-BELT IDLERS EQUIPPED WITH ANTI-FRICTION BEARINGS\*

Diameter of idler pulley, in.	Friction factor
4	0.0375
5	0.036
6	0.030
7	0.025

\* Courtesy Hewitt-Robins, Inc.



The information in Table 9-6 is used as follows:

**EXAMPLE.** Consider a conveyor 100 ft long, with a 5-ply 32-oz 30-in.-wide belt, weighing 6.8 lb per ft. The load will weigh 100 lb per cu ft, or 54 lb per ft of conveyor. The revolving parts will weigh 50 lb for a troughing idler and 31 lb for a return idler. Both idlers are 6 in. in diameter.

From Table 9-6 the idler friction factor is 0.030

No. troughing idlers required,  $100 \div 4.5 = 22$

Add extra idlers at loading point  $= 3$

Total no. troughing idlers  $= 25$

No. of return idlers,  $100 \div 10 = 10$

Total weight of the revolving parts of idlers will be

Troughing,  $25 \times 50 = 1,250$  lb

Return,  $10 \times 31 = 310$  lb

Weight of belt,  $200 \times 6.8 = 1,360$  lb

Weight of load,  $100 \times 54 = 5,400$  lb

Total weight  $= 8,320$  lb

The force required to overcome idler friction,  $8,320 \times 0.03 = 249.6$  lb

For a belt speed of 100 fpm the energy required per min will be

$$100 \times 249.6 = 24,960 \text{ ft-lb}$$

The horsepower required to overcome idler friction will be

$$P = \frac{24,960 \text{ ft-lb per min}}{33,000 \text{ ft-lb per min per hp}} = 0.76$$

For other belt speeds the required horsepower will be

$$P = \frac{0.76 \times \text{speed, fpm}}{100}$$

**Power Required to Drive a Belt Conveyor.** The total external power required to drive a loaded belt conveyor is the algebraic sum of the power required by each of the following:

1. To move the empty belt over the idlers
2. To move the load horizontally
3. To lift or lower the load vertically
4. To turn all pulleys
5. To compensate for drive losses
6. To operate a tripper, if one is used

The power required for each of these operations can be determined with reasonable accuracy for any given conveyor system, as explained later.

**Power Required to Move an Empty Belt.** The power required to move an empty conveyor belt over the idlers will vary with the type of idler bearings, the diameter and spacing of the idlers, and the length, weight, and speed of the belt. The energy required to move an empty belt is given by the equation

$$E = LSCQ \quad (9-1)$$

where  $E$  = energy, ft-lb per min

$L$  = length of conveyor, ft

$S$  = belt speed, fpm

$C$  = idler-friction factor, from Table 9-6

$Q$  = weight of moving parts per ft of conveyor

Equation (9-1) may be expressed as horsepower by dividing by 33,000 to give

$$P = \frac{LSCQ}{33,000} \quad (9-2)$$

Representative values of  $Q$  are given in Table 9-7. If more accurate values are desired for a given conveyor, they may be determined from the design of the particular conveyor and the weight of the belt used.

TABLE 9-7. REPRESENTATIVE VALUES OF  $Q^*$

Width of belt, in.	Idlers, 5 in. diam, steel pulleys				Weight of belt, lb per ft	Weight of conveyor, lb per ft			Q, lb per ft
	Troughing		Return			Idlers		Belt	
	Wt of revolving parts, lb	Spac- ing	Wt of revolving parts, lb	Spac- ing		Trough- ing	Re- turn		
14	18	5'0"	9	10'0"	2.8	3.6	0.9	5.6	10.1
16	20	5'0"	11	10'0"	3.3	4.0	1.1	6.6	11.7
18	22	5'0"	12	10'0"	4.1	4.4	1.2	8.2	13.8
20	24	5'0"	14	10'0"	4.6	4.8	1.4	9.2	15.4
24	26	5'0"	17	10'0"	7.0	5.2	1.7	14.0	20.9
30	31	4'6"	21	10'0"	8.5	6.9	2.1	17.0	26.0
36	36	4'6"	25	10'0"	11.3	8.0	2.5	22.6	33.1
42	40	4'0"	29	10'0"	17.0	10.0	2.9	34.0	46.0
48	45	3'3"	34	10'0"	23.8	13.8	3.4	47.6	64.8
54	74	2'9"	54	10'0"	29.2	26.9	5.4	73.2	105.5
60	80	2'3"	60	10'0"	32.5	35.6	6.0	74.0	115.6

\* Courtesy Hewitt-Robins, Inc.

**EXAMPLE.** The use of equation (9-2) is illustrated by determining the horsepower required to move a 30-in.-wide belt on a conveyor whose length is 1,800 ft, equipped with 5-in.-diameter idler pulleys, with antifriction bearings. Assume a belt speed of 100 fpm.

From Table 9-6 the value of  $C$  will be 0.036.

From Table 9-7 the value of  $Q$  will be 26 lb per ft of conveyor length.

The power required to move the empty belt will be

$$P = \frac{1,800 \times 100 \times 0.036 \times 26}{33,000} = 5.10 \text{ hp}$$



Table 9-8 gives representative values for the horsepower required to move empty conveyor belts. The values are based on using 5-in.-diameter idlers with antifriction bearings, and the belt widths given in Table 9-7.

TABLE 9-8. HORSEPOWER REQUIRED TO MOVE EMPTY CONVEYOR BELTS FOR A SPEED OF 100 FPM\*

Length of conveyor, ft	Width of belt, in.										
	14	16	18	20	24	30	36	42	48	54	60
50	0.05	0.06	0.07	0.08	0.11	0.14	0.18	0.25	0.35	0.54	0.63
100	0.11	0.13	0.15	0.17	0.23	0.28	0.36	0.51	0.70	1.14	1.25
150	0.16	0.19	0.22	0.25	0.34	0.42	0.53	0.76	1.05	1.71	1.88
200	0.22	0.25	0.30	0.33	0.45	0.56	0.71	1.01	1.40	2.28	2.50
250	0.27	0.32	0.37	0.42	0.56	0.70	0.89	1.27	1.75	2.85	3.13
300	0.33	0.38	0.45	0.50	0.68	0.84	1.07	1.52	2.10	3.42	3.76
400	.....	.....	0.60	0.66	0.90	1.12	1.43	2.03	2.80	4.56	5.01
500	.....	.....	.....	0.83	1.13	1.40	1.79	2.53	3.50	5.70	6.26
600	.....	.....	.....	1.00	1.35	1.68	2.14	3.04	4.20	6.84	7.51
800	.....	.....	.....	.....	1.80	2.25	2.86	4.05	5.60	9.12	10.00
1,000	.....	.....	.....	.....	2.26	2.81	3.57	5.07	7.00	11.40	12.50
1,200	.....	.....	.....	.....	.....	3.37	4.29	6.08	8.40	13.70	15.00
1,400	.....	.....	.....	.....	.....	3.93	5.00	7.09	9.80	16.00	17.50
1,600	.....	.....	.....	.....	.....	4.49	5.72	8.10	11.20	18.30	20.10
1,800	.....	.....	.....	.....	.....	5.05	6.43	9.12	12.60	20.50	22.60
2,000	.....	.....	.....	.....	.....	5.62	7.15	10.10	14.00	22.80	24.90
2,200	.....	.....	.....	.....	.....	.....	7.86	11.10	15.40	25.10	27.60
2,400	.....	.....	.....	.....	.....	.....	8.58	12.20	16.80	27.40	30.10
2,600	.....	.....	.....	.....	.....	.....	9.29	13.20	18.20	29.60	32.60
2,800	.....	.....	.....	.....	.....	.....	10.00	14.20	19.60	31.90	35.00
3,000	.....	.....	.....	.....	.....	.....	10.70	15.20	21.00	34.20	37.60

\* Courtesy Hewitt-Robins, Inc.

The power values given in this table are based on the use of 5-in.-diameter idlers. For 4-in.-diameter idlers increase the values by 4 per cent. For 6-in.-diameter idlers decrease the values by 17 per cent.

**Power Required to Move a Load Horizontally.** The power required to move a load horizontally may be expressed by equation (9-2) if  $Q$  is replaced by  $W$ , the weight of the load in pounds per foot of belt.

$$P = \frac{LSCW}{33,000} \quad (9-3)$$

This equation may be expressed in terms of the load moved in tons per hour. Let

$T$  = tons material moved per hr

$SW$  = lb material moved per min

$60SW$  = lb material moved per hr

$$T = \frac{60SW}{2,000} = \frac{3SW}{100}$$

Solving,

$$SW = \frac{100T}{3} \quad (9-4)$$

Substituting this value of  $SW$  in equation (9-3), the horsepower required to move a load horizontally is

$$P = \frac{100LCT}{3 \times 33,000} = \frac{LCT}{990} \quad (9-5)$$

Table 9-9 gives values for the horsepower required to move loads horizontally on conveyor belts. The values are based on using 5-in.-diameter idlers with antifriction bearings. For 6-in.-diameter idlers decrease the values by 17 per cent.

TABLE 9-9. HORSEPOWER REQUIRED TO MOVE LOADS HORIZONTALLY ON CONVEYOR BELTS\*

CONVEYOR BELTS															
Length of conveyor, ft	Load, tons per hr														
	50	100	150	200	250	300	350	400	500	600	700	800	900	1,000	
50	0.09	0.18	0.27	0.36	0.46	0.55	0.64	0.73	0.91	1.1	1.3	1.5	1.6	1.8	
100	0.18	0.36	0.55	0.74	0.91	1.1	1.3	1.5	1.8	2.2	2.6	2.9	3.3	3.6	
150	0.27	0.55	0.82	1.1	1.4	1.6	1.9	2.2	2.7	3.3	3.8	4.4	4.9	5.5	
200	0.36	0.73	1.1	1.5	1.8	2.2	2.6	2.9	3.6	4.4	5.1	5.8	6.6	7.3	
250	0.46	0.91	1.4	1.8	2.3	2.7	3.2	3.6	4.6	5.5	6.4	7.3	8.2	9.1	
300	0.55	1.1	1.6	2.2	2.7	3.3	3.8	4.4	5.5	6.6	7.7	8.8	9.9	10.9	
400	0.73	1.5	2.2	2.9	3.6	4.4	5.1	5.8	7.3	8.7	10.2	11.6	13.1	14.6	
500	0.91	1.8	2.7	3.6	4.6	5.5	6.4	7.4	8.5	10.6	12.7	14.8	17.0	19.1	
600	1.10	2.1	3.2	4.2	5.3	6.4	7.4	8.5	10.6	12.7	14.8	17.0	19.1	21.0	
800	1.40	2.7	4.1	5.5	7.5	8.2	9.5	10.8	13.7	16.4	19.1	22.0	25.0	27.0	
1,000	1.70	3.3	5.0	6.7	9.2	10.0	11.7	13.3	16.7	20.0	23.0	27.0	30.0	33.0	
1,200	2.0	3.9	5.9	7.9	10.8	11.8	13.8	15.7	19.8	24.0	28.0	32.0	36.0	39.0	
1,400	2.3	4.5	6.8	9.1	12.4	13.7	15.9	18.1	23.0	27.0	32.0	36.0	41.0	45.0	
1,600	2.6	5.2	7.7	10.3	12.9	15.5	18	21	26	31	36	41	46	52	
1,800	2.9	5.8	8.7	11.5	14.4	17.3	20	23	28	35	40	46	52	58	
2,000	3.2	6.4	9.6	12.7	15.9	19.1	22	25	32	38	45	51	57	64	
2,200	3.5	7.0	10.5	13.9	17.4	20.4	24	28	35	42	49	56	63	70	
2,400	3.9	7.6	11.4	15.2	18.9	23.0	27	30	38	46	53	61	68	76	
2,600	4.1	8.2	12.3	16.4	20.0	25.0	29	33	41	49	57	65	74	82	
2,800	4.4	8.8	13.2	17.6	22.0	26.0	31	35	44	53	62	70	79	88	
3,000	4.7	9.4	14.1	18.8	23.0	28.0	33	37	47	56	66	75	85	94	

\* Courtesy Hewitt-Robin, Inc.

The power values given in this table are based on the use of 5-in.-diameter idlers. For 4-in.-diameter idlers increase the values by 4 per cent.

### Power Required to Move a Load up an Inclined Belt Conveyor.

When a load is moved up an inclined belt conveyor, the power required may be divided into two components: the power required to move the load horizontally and the power required to lift the load through the net change in elevation. The power required to move the load horizontally



may be determined from equation (9-5). The power required to lift the load through the net change in elevation may be determined as follows: Let

$H$  = net change in elevation, ft

$T$  = tons material per hr

From equation (9-4)

$$\frac{100T}{3} = \text{lb material per min}$$

$$\frac{100TH}{3} = \text{energy, ft-lb per min}$$

Dividing by 33,000 gives the horsepower,

$$P = \frac{100TH}{3 \times 33,000} = \frac{TH}{990} \quad (9-6)$$

If the load is moved up an inclined conveyor, the power given in equation (9-6) must be supplied from an outside source. If the load is moved down an inclined conveyor, the power will be supplied to the belt by the load.

TABLE 9-10. HORSEPOWER REQUIRED TO LIFT A LOAD\*

Net lift, ft	Load, tons per hr											
	50	100	150	200	250	300	350	400	500	600	800	1,000
5	0.3	0.5	0.8	1.0	1.3	1.5	1.8	2.0	2.5	3.0	4.0	5.1
10	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	5.1	6.1	8.1	10.0
15	0.8	1.5	2.3	3.0	3.8	4.5	5.3	6.1	7.6	9.1	12.0	15.0
20	1.0	2.0	3.0	4.0	5.1	6.1	7.1	8.1	10.0	12.0	16.0	20.0
25	1.3	2.5	3.8	5.1	6.3	7.6	8.8	10.0	13.0	15.0	20.0	25.0
30	1.5	3.0	4.5	6.1	7.6	9.1	11.0	12.0	15.0	18.0	24.0	30.0
40	2.0	4.0	6.1	8.1	10.0	12.0	14.0	16.0	20.0	24.0	32.0	40.0
50	2.5	5.1	7.6	10.0	13.0	15.0	18.0	20.0	25.0	30.0	40.0	51.0
75	3.8	7.6	11.0	15.0	19.0	23.0	27.0	30.0	38.0	45.0	61.0	76.0
100	5.1	10.0	15.0	20.0	25.0	30.0	35.0	40.0	51.0	61.0	81.0	101
125	6.3	13.0	19.0	25.0	32.0	38.0	44.0	51.0	63.0	76.0	101	126
150	7.6	15.0	23.0	30.0	38.0	45.0	53.0	61.0	76.0	91.0	121	152
200	10.0	20.0	30.0	40.0	51.0	61.0	71.0	81.0	101	121	162	202
300	15.0	30.0	45.0	61.0	76.0	91.0	106	121	152	185	242	303
400	20.0	40.0	61.0	81.0	101	121	141	162	202	242	323	404
500	25.0	51.0	76.0	101	126	151	177	202	252	303	404	505

\* Courtesy Hewitt-Robins, Inc.

**Driving Equipment.** A belt conveyor may be driven through the head or tail pulley or through an intermediate pulley. In the event high

driving forces are required, it may be necessary to use more than one pulley, with the pulleys arranged in tandem to increase the areas of contact with the belt. Smooth-faced or lagged pulleys may be used, depending on the desired coefficient of friction between the belt and the pulley surface. The pulley may be driven by an electric motor or a gasoline or diesel engine. It is usually necessary to install a suitable speed reducer, such as gears, chain drives, or belt drives, between the power unit and the driving pulley. The power loss in the speed reducer should be included in determining the total power required to drive a belt conveyor. This loss may amount to 5 to 10 per cent, or more, depending on the type of speed reducer.

The coefficient of friction between a steel shaft and babbitted bearings will be approximately 0.10.

When power is transmitted from a driving pulley to a belt, the effective driving force, which is transmitted to the belt, is equal to the tension in the tight side less the tension in the slack side of the belt, expressed in pounds.

$$T_e = T_1 - T_2 \quad (9-7)$$

where  $T_e$  = effective tension or driving force between pulley and belt

$T_1$  = tension in tight side of belt

$T_2$  = tension in slack side of belt

The coefficient of friction between a rubber belt and a bare steel or cast-iron pulley is approximately 0.25. If the surface of a pulley is lagged with a rubberized fabric, the coefficient of friction will be increased to approximately 0.35.

When power is transmitted from a pulley to a belt, the tension in the slack side of the belt should not exceed the amount required to prevent slippage between the pulley and the belt. For a driving pulley with a given diameter and speed, the effective tension  $T_e$  required to transmit a given horsepower to the belt may be determined from the following equation,

$$P = \frac{\pi D T_e N}{33,000} \quad (9-8)$$

where  $P$  = hp transmitted to belt

$D$  = diameter of pulley, ft

$T_e$  = effective force between pulley and belt, lb

$N$  = no. rpm

The equation may be rewritten as

$$T_e = \frac{33,000P}{\pi D N} \quad (9-9)$$



TABLE 9-11. TENSION FACTORS FOR DRIVING PULLEYS\*

Arc of contact, deg	Bare pulley	Lagged pulley
Single-pulley drive		
200	1.72	1.42
210	1.70	1.40
215	1.65	1.38
220	1.62	1.35
240	1.54	1.30
Tandem drive		
360	1.26	1.13
380	1.23	1.11
400	1.21	1.10
450	1.18	1.09
500	1.14	1.06

\* Courtesy Hewitt-Robins, Inc.

The ratio  $T_1/T_e$  is defined as the pulley tension factor. This factor varies with the type of pulley surface, bare or lagged, and the arc of contact between the belt and the pulley. Values for the factor are given in Table 9-11. The factor may be expressed as

$$F = \frac{T_1}{T_e} \quad (9-10)$$

If the required effective force  $T_e$  between a pulley and a belt whose arc of contact is  $210^\circ$ , is 3,000 lb, the minimum tension in the tight side of the belt may be determined from equation (9-10) and Table 9-11. From Table 9-11,  $F = 1.70$  for a bare pulley.

$$\begin{aligned} T_1 &= FT_e \\ &= 1.70 \times 3,000 = 5,100 \text{ lb} \end{aligned}$$

If the same pulley is lagged, the value of  $F$  will be 1.40 and

$$T_1 = 1.40 \times 3,000 = 4,200 \text{ lb}$$

For these conditions the minimum values of  $T_1$ ,  $T_2$ , and  $T_e$  will be as follows:

	Bare pulley	Lagged pulley
$T_1$	5,100 lb	4,200 lb
$T_e$	3,000 lb	3,000 lb
$T_2$	2,100 lb	1,200 lb

Thus, it is evident that by lagging a drive pulley the tension in a belt may be reduced, possibly enough to permit the use of a lighter and less expensive belt.

**Power Required to Turn Pulleys.** A belt conveyor includes several pulleys, around which the belt is bent. For the shaft of each pulley there is a bearing friction that requires the consumption of power. The power required will vary with the tension in the belt, the weight of the pulley and shaft, and the type of bearing, babbitted or antifriction. For a given conveyor the friction factors for each pulley may be determined reasonably accurately, and from this information the additional power required to compensate for the loss due to pulley friction may be obtained. Table 9-12 gives the per cent of the power delivered to a conveyor required to overcome pulley friction for conveyors with head drive and babbitted bearings for all pulley shafts.

TABLE 9-12. PER CENT OF SHAFT HORSEPOWER REQUIRED TO OVERCOME PULLEY FRICTION FOR CONVEYORS WITH HEAD DRIVE AND BABBITTED BEARINGS\*

Length of conveyor, ft	Slope of conveyor, deg				
	0	1-6	6-11	11-16	16-20
20	112	93	53	35	28
30	76	63	36	25	19
50	45	38	22	15	13
75	30	25	15	12	9
100	22	19	11	8	7
150	15	14	9	7	6
200	14	11	8	6	5
250	12	10	7	5	5
300	11	8	6	5	4
400	9	6	5	4	4
500	7	6	5	4	3
600	6	5	4	3	3
700	5	4	4	3	3
800	4	4	3	3	3
1,000	4	4	3	3	3
2,000	4	4	3		
3,000	4	3	3		

\* Courtesy Hewitt-Robins, Inc.

For antifriction bearings use one-half of the above percentages.

**Conveyor-belt Take-ups.** Because of the tendency of a conveyor belt to elongate after it is put into operation, it is necessary to provide a method of adjusting for the increase in length.



A screw take-up may be used to increase the length of the conveyor by moving the head or tail pulley. This adjustment may be sufficient for a short belt but not for a long belt.

Another take-up, which is more satisfactory, depends on forcing the returning belt to travel under a weighted pulley, which provides a uniform tension in the belt regardless of the variation in length.

**Holdbacks.** If a belt conveyor is operated on an incline, it is advisable to install a holdback on the driving pulley to prevent the load from causing the belt to run backward in the event of a power failure. A holdback is a mechanical device which permits a driving pulley to rotate in the normal direction but prevents it from rotating in the opposite direction. The operation of a holdback should be automatic. At least three types are available. They are the roller, ratchet, and differential band brake, all of which operate automatically.

A holdback must be strong enough to resist the force produced by the load less the sum of the forces required to move the empty belt, move the load horizontally, turn the pulleys, drive the tripper and to overcome drive losses.

If a belt conveyor is operated on a decline, the effect of the load is to move the belt forward. If this effect exceeds the total forces of friction, it will be necessary to install a suitable braking unit to regulate the speed of the belt. To overcome this difficulty, an electric motor or generator may be used as the driving unit. In starting an empty belt, the unit will act as a motor, but when the effect of the load is sufficient to overcome all resistances, the unit will act as a generator to regulate the belt speed.

**Feeders.** The purpose of a feeder is to deliver material to a belt at a uniform rate. A feeder may discharge directly onto a belt, or it may discharge the material through a chute in order to reduce the impact of the falling material on the belt. Several types of feeders are available, each of which has advantages and disadvantages when compared with another type. Among the more popular types are the following:

Apron

Reciprocating

Rotary vane

Rotary plow

An apron feeder usually receives the material from a gated hopper, which regulates the flow onto the feeder. The feeder consists of a moving, flat rubber-covered belt or a number of flat steel plates connected to two moving chains. This feeder moves the material from under the hopper and discharges it through a receiving unit onto the conveyor belt. A belt feeder is suitable for handling material consisting of relatively small pieces. If the material contains large pieces of highly abrasive rock or

stone, a steel-plate-type feeder will usually prove more satisfactory than a belt type.

A reciprocating feeder consists of a steel plate placed under a hopper. The plate is operated through an eccentric drive to produce the reciprocating effect, which moves the material onto the conveyor belt.

A rotary-vane feeder consists of a number of vanes mounted on a horizontal shaft. As the material flows down an inclined plane, the rotating vanes deliver measured amounts to the conveyor. The rate of feeding may be regulated by varying the speed of the rotating vanes.

A rotary-plow feeder consists of a number of plows, or vanes, mounted on a vertical shaft. The plows rotate over a horizontal table onto which

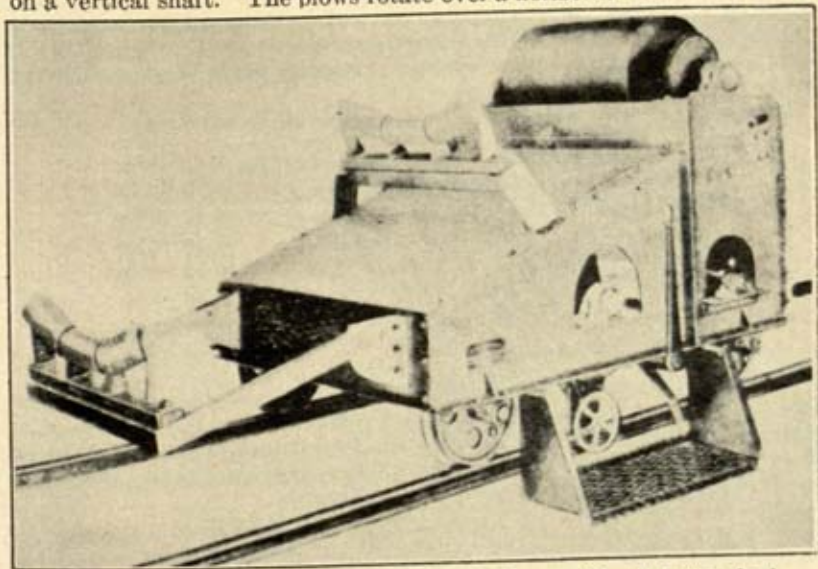


Fig. 9-7. Belt-propelled automatic-controlled tripper. (Hewitt-Robins, Inc.)

the material is allowed to flow. The rate of feeding may be regulated by varying the speed of the plows.

**Trippers.** When it is necessary to remove material from a belt conveyor before it reaches the end of the belt, a tripper should be installed on the conveyor. A tripper consists of a pair of pulleys which are so located that the loaded belt must pass over one pulley and under the other. As the belt passes over the top pulley, the load will be discharged from the belt into an auxiliary hopper or chute.

A tripper may be a stationary or a traveling type. The latter type may be propelled by a hand-operated crank, a separate motor, or the conveyor belt. If a tripper is installed on a conveyor, additional power should be provided to operate it.



**Belt-conveyor Design.** Design a belt conveyor to transport unsized crushed limestone. The essential information is as follows:

- Capacity, 300 tons per hr
- Horizontal distance, 360 ft
- Vertical lift, 40 ft
- Max size stone, 6 in.
- Weight of stone, 100 lb per cu ft
- Required belt width, from Table 9-2, 24 in.
- Max speed, from Table 9-3, 400 fpm
- Capacity at 400 fpm, from Table 9-2,  $4 \times 100 = 400$  tons per hr
- Required belt speed,  $\frac{400 \times 300}{400} = 300$  fpm

This speed, 300 fpm, will be satisfactory provided the feeder supplies material at a uniform rate. If the rate of feeding is irregular, it may be necessary to increase the speed to assure the specified rate of delivery. The design will be based on a speed of 350 fpm to provide a margin of safety.

The power required to operate the loaded conveyor will be as follows:

To drive the empty belt, Table 9-8, $0.81 \times 350/100 =$	2.84 hp
To move the load horizontally, Table 9-9	= 3.92 hp
To lift the load, Table 9-10	= 12.00 hp
Subtotal	= 18.76 hp
For pulley friction, Table 9-12, 4% of 18.76	= 0.75 hp
Subtotal required by belt	= 19.51 hp
For drive losses, 10% of 19.51	= 1.95 hp
Total power required	= 21.46 hp

Determine the type, size, and number of driving pulleys required to operate the belt. The belt will be driven through a head pulley. When a belt is driven by a pulley, the effective driving force transmitted to the belt is equal to the difference in the belt tensions on the tight side and the slack side, expressed in pounds. This difference is referred to as the effective driving force or tension.

Let

- $T_1$  = tight-side tension
- $T_2$  = slack-side tension
- $T_e$  = effective tension
- $T_e = T_1 - T_2$

The value of  $T_e$  can be determined from the horsepower transmitted to the belt and the belt speed in fpm.

$$T_e = \frac{\text{hp} \times 33,000}{\text{belt speed, fpm}} = \frac{19.5 \times 33,000}{350} = 1,838 \text{ lb}$$

$$T_1 - T_2 = 1,838 \text{ lb}$$

It is desirable to operate the belt at the lowest practical tight-side and slack-side tensions. The necessary tensions are maintained by the take-ups. The maximum slack-side tension will occur as the belt leaves the driving pulley. This tension will equal the tension at the tail pulley plus the weight of the vertical component of the belt. Field observations indicate that the tension in the belt at the loading point should be not less than 20 lb per in. of belt width. This tension will be transmitted

to the slack side of the belt at the tail pulley. The minimum possible slack-side tension will be

$$\begin{aligned}\text{Tension at tail pulley, } 20 \times 24 &= 480 \text{ lb} \\ \text{Weight of belt, } 40 \text{ ft} \times 6 \text{ lb per ft} &= 240 \text{ lb} \\ \text{Total tension} &= 720 \text{ lb}\end{aligned}$$

The values of  $T_1$  and  $T_2$  for each of three driving arrangements will be as follows:

Arc of contact, deg	$T_1$ , lb	$T_2$ , lb
Single bare drive		
215	$1,838 \times 1.65 = 3,035$	$3,035 - 1,838 = 1,197$
220	$1,838 \times 1.62 = 2,980$	$2,980 - 1,838 = 1,142$
Single lagged drive		
215	$1,838 \times 1.38 = 2,538$	$2,538 - 1,838 = 700$
220	$1,838 \times 1.35 = 2,480$	$2,480 - 1,838 = 642$
Tandem bare drive		
400	$1,838 \times 1.21 = 2,225$	$2,225 - 1,838 = 387$
450	$1,838 \times 1.18 = 2,170$	$2,170 - 1,838 = 332$

Regardless of the type of drive selected, the minimum slack-side tension in the belt just as it leaves the head pulley will be 720 lb. Adding the effective tension  $T_e$  gives a minimum tight-side tension of  $720 + 1,838 = 2,558$  lb. If, for a single lagged drive, with an arc of contact of  $215^\circ$ ,  $T_2$  is increased to 720 lb,  $T_1$  will be 2,558 lb, which satisfies the tension requirements. Thus, a single lagged pulley will be used to drive the belt.

Reference to Table 9-4 indicates that a three-ply 42-oz belt has a safe working stress of 2,640 lb, which is satisfactory. The thickness of this belt will permit it to trough satisfactorily. Reference to Table 9-4 indicates that the minimum pulley diameters should be head, 20 in.; tail, take-up, and snub, 16 in.; bend, 12 in.

The troughing idlers should be 5 in. in diameter spaced 5 ft 0 in. apart, with a maximum spacing of 1 ft 6 in. at the loading point. The return idlers should be 5 in. in diameter, spaced 10 ft 0 in. apart.

### PROBLEMS

9-1. What are the minimum width belt and the minimum belt speed, in fpm, required to transport 350 tons per hr of unsized crushed stone, weighing 100 lb per cu ft? The maximum size stone is 6 in.

9-2. What is the capacity of a 30-in.-wide troughed belt, in tons per hour, when the belt is handling crushed stone weighing 100 lb per cu ft and is moving at the maximum recommended speed?

9-3. A conveyor 250 ft long with a 36-in.-wide 6-ply 32-oz belt is used to transport material weighing 125 lb per cu ft. The angle of repose for the material is  $20^\circ$ . The revolving parts will weigh 36 lb for each troughing idler and 25 lb for each return idler.



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Both idlers, which are 5 in. in diameter, are equipped with antifriction bearings. Determine the horsepower required to overcome idler friction for the conveyor when the belt speed is 180 fpm.

9-4. Using the information given in Table 9-7, determine the horsepower required to move an empty conveyor belt 36 in. wide, on a conveyor whose length is 1,200 ft, equipped with 5-in.-diameter idlers, with antifriction bearings, when the belt speed is 250 fpm.

9-5. If the belt conveyor of Prob. 9-4 is installed in a horizontal position, determine the horsepower required to operate the belt when it is transporting 200 tons per hr of material at a speed of 100 fpm.

9-6. If the belt conveyor of Prob. 9-4 is installed on a 15° slope, determine the horsepower required to operate the belt when it is moving 200 tons per hr of material up the slope at a speed of 100 fpm.

9-7. Design a conveyor belt to transport 250 tons per hr of crushed stone. The essential information is as follows:

- Horizontal distance, 1,420 ft
- Vertical lift, 120 ft
- Max size stone, 5 in.
- Weight of stone, 96 lb per cu ft
- Use 5-in.-diameter idlers, with antifriction bearings

Your design should furnish the following information:

- The required width of the belt
- The required belt speed
- The required horsepower
- The type, size, and number of driving pulleys required to operate the belt
- The thickness and weight of belt required
- The diameters of the head, tail, take-up, snub, and bend pulleys
- The spacing of the idlers

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## CHAPTER 10

### COMPRESSED AIR

**General Information.** Compressed air is used extensively on construction projects for drilling rock or other hard formations, loosening earth, operating air motors, hand tools, pile drivers, pumps, mucking equipment, cleaning, etc. In many instances the energy supplied by compressed air is the most convenient method of operating equipment and tools.

When air is compressed, it receives energy from the compressor. This energy is transmitted through a pipe or hose to the operating equipment, where a portion of the energy is converted into mechanical work. The operations of compressing, transmitting, and using air will always result in a loss of energy, which will give an over-all efficiency less than 100 per cent, sometimes considerably less.

**Fundamental Gas Laws.** As air is a gas, it obeys, within reason, the fundamental laws which apply to gases. The laws with which we are concerned are related to the pressure, volume, temperature, and transmission of air.

**Definitions of Gas-law Terms.** In order to understand the laws which relate to compressed air, it is necessary to define certain terms which are used in developing and applying these laws. The essential definitions are as follows:

**Gauge Pressure.** This is the pressure exerted by the air in excess of atmospheric pressure. It is usually expressed in psi or inches of mercury and is measured by a pressure gauge or a mercury manometer.

**Absolute Pressure.** This is the total pressure measured from absolute zero. It is equal to the sum of the gauge and the atmospheric pressure, corresponding to the barometric reading. The absolute pressure should be used in dealing with the laws of gases.

Psi is the abbreviation for pounds per square inch of pressure.

Psf is the abbreviation for pounds per square foot of pressure.

**Vacuum.** This is a measure of the extent to which pressure is less than atmospheric pressure. For example, a vacuum of 5 psi is equivalent to an absolute pressure of  $14.7 - 5 = 9.7$  psi.

**Standard Conditions.** Because of the variations in the volume of air with pressure and temperature it is necessary to express the volume at standard conditions if it is to have a definite meaning. Standard conditions are an absolute pressure of 14.7 psi and a temperature of 60°F.



**Temperature.** Temperature is a measure of the amount of heat contained by a unit quantity of gas. It is measured with a thermometer or some other suitable temperature-indicating device.

**Fahrenheit Temperature.** This is the temperature indicated by a thermometer calibrated according to the Fahrenheit scale. For this thermometer pure water freezes at 32 deg and boils at 212 deg, at a pressure of 14.7 psi. Thus, the number of degrees between freezing and boiling water is 180.

**Centigrade Temperature.** This is the temperature indicated by a thermometer calibrated according to the centigrade scale. For this thermometer pure water freezes at 0 deg and boils at 100 deg, at a pressure of 14.7 psi.

**Relation between Fahrenheit and Centigrade Temperatures.** As 180 deg on the Fahrenheit scale equals 100 deg on the centigrade scale,  $1^{\circ}\text{C}$  equals  $1.8^{\circ}\text{F}$ . A Fahrenheit thermometer will read 32 deg when a centigrade thermometer reads 0 deg.

Let  $T_F$  = Fahrenheit temperature and  $T_C$  = centigrade temperature. For any given temperature the thermometer readings are expressed by the following equation:

$$T_F = 32 + 1.8T_C \quad (10-1)$$

**Absolute Temperature.** This is the temperature of a gas measured above absolute zero. It equals degrees Fahrenheit plus 459.6 or, as more commonly used, 460.

**Isothermal Compression.** When a gas undergoes a change in volume without any change in temperature, this is referred to as isothermal expansion or compression.

**Adiabatic Compression.** When a gas undergoes a change in volume without gaining or losing heat, this is referred to as adiabatic expansion or compression.

Boyle's law states that when a gas is subjected to a change in volume due to a change in pressure, at a constant temperature, the product of the pressure times the volume will remain constant. This relation is expressed by the equation

$$P_1V_1 = P_2V_2 = K \quad (10-2)$$

where  $P_1$  = initial absolute pressure

$V_1$  = initial volume

$P_2$  = final absolute pressure

$V_2$  = final volume

$K$  = a constant

**EXAMPLE.** Determine the final volume of 1,000 cu ft of air when the gauge pressure is increased from 20 to 120 psi, with no change in temperature. The barometer indicates an atmospheric pressure of 14.7 psi.

$$\begin{aligned}
 P_1 &= 20 + 14.7 = 34.7 \text{ psi} \\
 P_2 &= 120 + 14.7 = 134.7 \text{ psi} \\
 V_1 &= 1,000 \text{ cu ft}
 \end{aligned}$$

From equation (10-2)

$$V_2 = \frac{P_1 V_1}{P_2} = \frac{34.7 \times 1,000}{134.7} = 257.8 \text{ cu ft}$$

**Boyle's and Charles' Laws.** When a gas undergoes a change in volume or pressure with a change in temperature, Boyle's law will not apply. Charles' law introduces the effect of absolute temperature on the volume of a gas when the pressure is maintained constant. It states that the volume of a given weight of gas at constant pressure varies in direct proportion to its absolute temperature. It may be expressed mathematically by the equation

$$\frac{V_1}{T_1} = \frac{V_2}{T_2} = C \quad (10-3)$$

where  $V_1$  = initial volume

$T_1$  = initial absolute temperature

$V_2$  = final volume

$T_2$  = final absolute temperature

$C$  = a constant

The laws of Boyle and Charles may be combined to give the equation

$$\frac{P_1 V_1}{T_1} = \frac{P_2 V_2}{T_2} = \text{a constant} \quad (10-4)$$

Equation (10-4) may be used to express the relations between pressure, volume, and temperature for any given gas, such as air. It is illustrated by the following example.

**EXAMPLE.** One thousand cubic feet of air, at an initial gauge pressure of 40 psi and temperature of 50°F, is compressed to a volume of 200 cu ft at a final temperature of 110°F. Determine the final gauge pressure. The atmospheric pressure is 14.46 psi.

$$P_1 = 40 + 14.46 = 54.56 \text{ psi}$$

$$V_1 = 1,000 \text{ cu ft}$$

$$T_1 = 460 + 50 = 510^\circ\text{F}$$

$$V_2 = 200 \text{ cu ft}$$

$$T_2 = 460 + 110 = 570^\circ\text{F}$$

Rewriting equation (10-4) and substituting these values, we get

$$P_2 = \frac{P_1 V_1 T_2}{T_1 V_2} = \frac{54.56 \times 1,000 \times 570}{510 \times 200} = 304 \text{ psi}$$

$$\text{Final gauge pressure} = 304 - 14.46 = 289.54 \text{ psi}$$

**Energy Required to Compress Air.** Equation (10-2) may be expressed as  $PV = K$ , where  $K$  is a constant so long as the temperature remains



constant. However, in actual practice the temperature usually will not remain constant, and the equation must be modified to provide for the effect of changes in temperature. The effect of temperature may be provided for by introducing an exponent  $n$  to  $V$ . Thus, equation (10-2) may be rewritten as

$$P_1 V_1^n = P_2 V_2^n = K \quad (10-5)$$

For air the values of  $n$  will vary from 1.0 for isothermal compression to 1.4 for adiabatic compression. The actual value for any compression condition may be determined experimentally from an indicator card obtained from a given compressor.

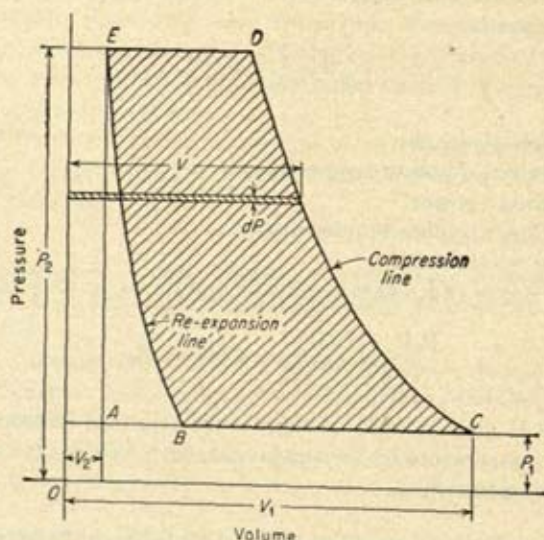


FIG. 10-1. Cycle for isothermal compression of air.

When the pressure of a given volume of air is increased by an air compressor, it is necessary to furnish energy to the air. Consider a single compression cycle for an air compressor, as indicated in Fig. 10-1. Air is drawn into the cylinder at pressure  $P_1$  and is discharged at pressure  $P_2$ .  $P_1$  does not need to be atmospheric pressure. The initial volume is  $V_1$ . As the piston compresses the air, the pressure-volume will follow the curve  $CD$ . At  $D$ , when the pressure is  $P_2$ , the discharge valve will open and the pressure will remain constant while the volume decreases to  $V_2$ , as indicated by line  $DE$ . Point  $E$  represents the end of the piston stroke. At point  $E$  the discharge valve will close, and as the piston begins its return stroke, the pressure will decrease along line  $EB$  to a value of  $P_1$ ,

when the intake valve will open and allow additional air to enter the cylinder. This will establish line  $BC$ .

The work done along the line  $CD$  may be obtained by integrating the equation  $dW = V dP$

From equation (10-5),  $V^n = K/P$ . If both sides of the equation are raised to the  $1/n$  power, the equation will be

$$V = \left(\frac{K}{P}\right)^{1/n}$$

Substituting this value of  $V$  gives

$$dW = \left(\frac{K}{P}\right)^{1/n} dP$$

Integrating gives

$$W = K^{1/n} \int_1^2 \frac{dP}{P} \quad (10-6)$$

For isothermal compression  $n = 1$ . Substituting this value in equation (10-6) gives

$$\begin{aligned} W &= K \int_1^2 \frac{dP}{P} \\ &= -K \log_e \frac{P_2}{P_1} + C \end{aligned}$$

When  $P_2 = P_1$  and no work is done, the constant of integration is equal to zero. The minus sign may be disregarded. Thus, for isothermal compression of air, the equation may be written as

$$W = K \log_e \frac{P_2}{P_1} \quad (10-7)$$

If it is desired to convert from natural to common logarithms,  $\log_e (P_2/P_1)$  may be replaced by  $2.302 \log_{10} (P_2/P_1)$ . For the given compression conditions  $n = 1$ , and  $K = P_1 V_1$ . If the compression is started on air at standard conditions,  $P_1$  will be 14.7 psi at 60°F. Since work is commonly expressed in foot-pounds, it is necessary to express  $P_1$  in psf. This is done by multiplying  $P_1$  by 144. When these substitutions are made, equation (10-7) may be written as

$$\begin{aligned} W &= 14.7 \times 144 V_1 \times 2.302 \log_{10} \frac{P_2}{P_1} \\ &= 4,883 V_1 \log_{10} \frac{P_2}{P_1} \end{aligned} \quad (10-8)$$



The value of  $W$  is in foot-pounds per cycle. One horsepower is equivalent to 33,000 ft-lb per min. If  $V_1$  in equation (10-8) is replaced by  $V$ , the volume of free air per minute at standard conditions, the horsepower required to compress  $V$  cu ft of air from an absolute pressure of  $P_1$  to  $P_2$  psi will be

$$\begin{aligned} \text{hp} &= \frac{4833 V \log_{10} (P_2/P_1)}{33,000} \\ \text{hp} &= 0.1479 V \log_{10} \frac{P_2}{P_1} \end{aligned} \quad (10-9)$$

**EXAMPLE.** Determine the theoretical horsepower required to compress 100 cu ft of free air per minute, measured at standard conditions, from atmospheric pressure to 100 psi gauge pressure. Substituting in equation (10-9), we get

$$\begin{aligned} \text{hp} &= 0.1479 \times 100 \times \log_{10} \frac{114.7}{14.7} \\ &= 14.79 \times \log_{10} 7.8 \\ &= 14.79 \times 0.892 \\ &= 13.2 \end{aligned}$$

If air is compressed under other than isothermal conditions, the equation for the required horsepower may be derived in a similar manner. However, since  $n$  will not equal 1, it must appear as an exponent in the equation. Equation (10-10) gives the horsepower for nonisothermal conditions.

$$\text{hp} = \frac{n}{n-1} 0.0643 V \left[ \left( \frac{P_2}{P_1} \right)^{\frac{n-1}{n}} - 1 \right] \quad (10-10)$$

where the terms are the same as those used in equation (10-9).

**EXAMPLE.** Determine the theoretical horsepower required to compress 100 cu ft of free air per minute, measured at standard conditions, from atmospheric pressure to 100 psi gauge pressure, under adiabatic conditions. The value of  $n$  will be 1.4 for air for adiabatic compression. Substituting in equation (10-10), we get

$$\begin{aligned} \text{hp} &= \frac{1.4}{1.4-1} \times 0.0643 \times 100 \left( 7.8^{\frac{0.4}{1.4}} - 1 \right) \\ &= 22.5 (7.8^{0.286} - 1) \\ &= 22.5 \times 0.79 \\ &= 17.8 \end{aligned}$$

For air compressors used on construction projects the compression will be performed under conditions between isothermal and adiabatic. Thus, the theoretical horsepower will be between 13.2 and 17.8, the actual value depending on the extent to which the compressor is cooled during operation. The difference in the horsepower required illustrates the importance of operating an air compressor at the lowest practical temperature.

**Air-compressor Definitions and Terms.** Many terms related to air compressors and compressed air have assumed uniform meanings. The essential terms are defined hereafter.

*Air Compressor.* This is a machine which is used to increase the pressure of air by reducing its volume.

*Reciprocating Compressor.* This is a machine which compresses air by means of a piston reciprocating in a cylinder.

*Single-acting Compressor.* This compressor is a machine which compresses air in only one end of a cylinder.

*Double-acting Compressor.* The double-acting compressor is a machine which compresses air in both ends of a cylinder.

*Single-stage Compressor.* This is a machine which compresses air from atmospheric pressure to the desired discharge pressure in a single operation.

*Two-stage Compressor.* This is a machine which compresses air in two separate operations. The first operation compresses the air to an intermediate pressure, while the second operation further compresses it to the desired final pressure.

*Multistage Compressor.* This is a compressor which produces the desired final pressure through two or more stages.

*Rotary Compressor.* The rotary compressor is a machine in which the compression is effected by the action of rotation elements.

*Centrifugal Compressor.* This compressor is a machine in which the compression is effected by a rotating vane or impeller that imparts velocity to the flowing air to give it the desired pressure.

*Intercooler.* The intercooler is a heat exchanger which is placed between two compression stages to remove the heat of compression from the air.

*Aftercooler.* This is a heat exchange which cools the air after it is discharged from a compressor.

*Inlet Pressure.* This is the absolute pressure of the air at the inlet to a compressor.

*Discharge Pressure.* Discharge pressure is the absolute pressure of the air at the outlet from a compressor.

*Compression Ratio.* This is the ratio of the absolute discharge pressure to the absolute inlet pressure.

*Free Air.* Free air is air as it exists under atmospheric conditions at any given location.

*Cfm.* is an abbreviation for cubic feet per minute.

*Capacity.* Capacity is the volume of air delivered by a compressor, expressed in cfm of free air.

*Theoretical Horsepower.* This is the horsepower required to compress



adiabatically the air delivered by a compressor through the specified pressure range, without any provision for lost energy.

**Brake Horsepower.** Brake horsepower is the actual horsepower input required by a compressor.

**Compressor Efficiency.** This is the ratio of the theoretical horsepower to the brake horsepower.

**Volumetric Efficiency.** This is the ratio of the capacity of a compressor to the piston displacement of the compressor.

**Density of Air.** This is the weight of a unit volume of air, usually expressed as pounds per cubic foot. Density varies with the pressure and

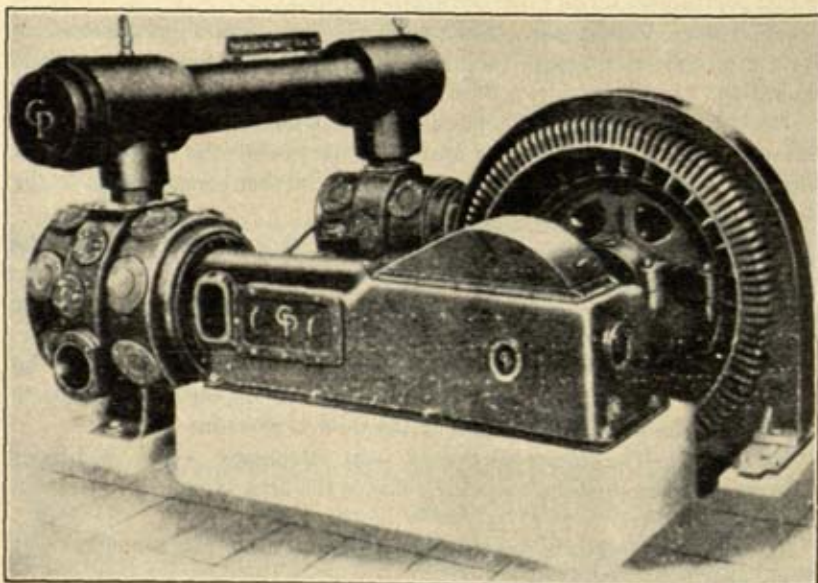


FIG. 10-2. Two-stage stationary air compressor. (Chicago Pneumatic Tool Co.)

temperature of the air. The weight of air at 60°F and 14.7 psi, absolute pressure, is 0.07658 lb per cu ft. The volume per pound is 13.059 cu ft.

**Load Factor.** The load factor is the ratio of the average load during a given period of time to the maximum rated load of a compressor.

**Diversity Factor.** This is the ratio of the actual quantity of air required for all uses to the sum of the individual quantities required for each use.

**Stationary Compressors.** Stationary compressors are generally used for installations where compressed air is required for a long period of time. The compressors may be reciprocating or rotary types, single-stage or multistage. The total quantity of air may be supplied by one or more compressors. The installed cost of a single compressor will usually

be less than for several compressors having the same capacity. However, several compressors provide better flexibility for varying load demands, and, in the event of a shutdown for repairs, the entire plant does not need to be stopped.

Stationary compressors may be driven by steam, electric motors, or internal-combustion engines.

**Portable Compressors.** Portable compressors are used when it is necessary to move the equipment frequently to meet job demands. The

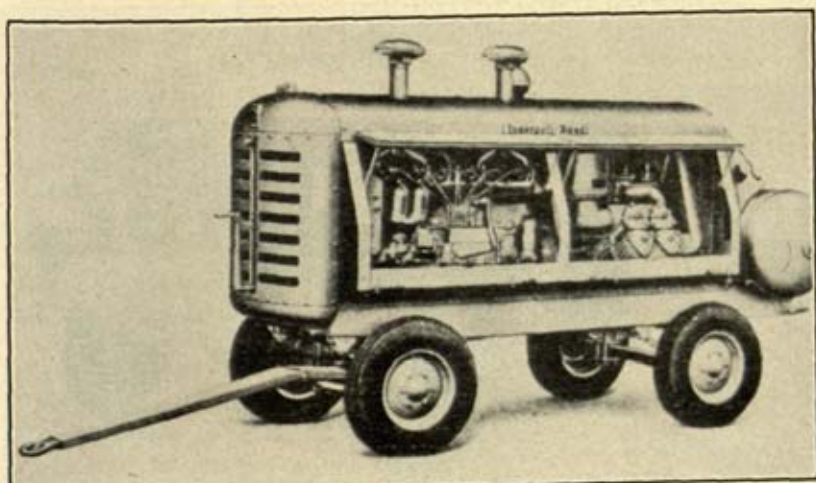


FIG. 10-3. Two-stage diesel-engine-operated portable air compressor, 315 cfm. (Ingersoll-Rand Co.)

compressors may be mounted on rubber tires, steel wheels, or skids. They may be driven by gasoline or diesel engines. They are available in single- or two-stage, reciprocating or rotary types.

**Reciprocating Compressors.** A reciprocating compressor depends on a piston, which moves back and forth in a cylinder, for the compressing action. The piston may compress air while moving in one or both directions. For the former it is defined as single-acting, while for the latter it is defined as double-acting. A compressor may have one or more cylinders.

**Rotary Compressors.** In recent years considerable effort has been directed toward the development of rotary compressors. These machines offer several advantages compared with reciprocating compressors, such as compactness, light weight, uniform flow, variable output, carefree operation, and long life.

Figure 10-4 illustrates a 600-cfm two-stage rotary compressor which



has given excellent performance in the construction industry. Its operating weight is 9,500 lb, which is comparable with the weight of a 315-cfm portable reciprocating unit. The cost is approximately the same as for a 600-cfm reciprocating compressor.

**Compressor Capacity.** Air compressors are rated by the piston displacement in cfm. However, the capacity of a compressor will be less than the piston displacement because of valve and piston leakage and the air left in the end-clearance spaces of the cylinders.

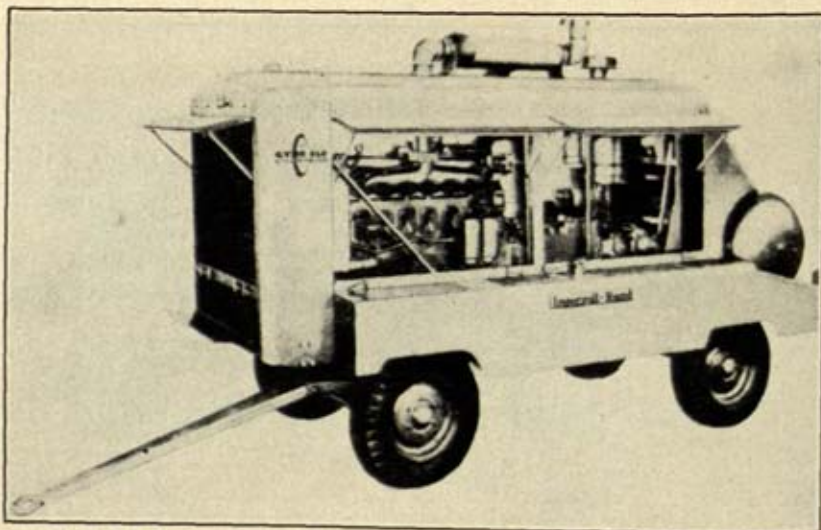


FIG. 10-4. Two-stage rotary air compressor, 600 cfm. (Ingersoll-Rand Co.)

The capacity of a compressor is the actual volume of free air drawn into a compressor in a minute. It is expressed in cubic feet. For a reciprocating compressor in good mechanical condition the actual capacity should be 80 to 90 per cent of the piston displacement. This is illustrated by an analysis of a 315-cfm two-stage portable compressor. The manufacturer's specifications give the following information:

- No. low-pressure cylinders, 4
- No. high-pressure cylinders, 2
- Diameter of low-pressure cylinders, 7 in.
- Diameter of high-pressure cylinders, 5 $\frac{3}{4}$  in.
- Length of stroke, 5 in.
- Rpm, 870

Consider the piston displacement of the low-pressure cylinders only as they determine the capacity of the unit.

Area of cylinder,  $\frac{\pi \times 7^2}{4 \times 144} = 0.267$  sq ft

Displacement per cylinder per stroke,  $0.267 \times \frac{5}{2} = 0.111$  cu ft

Displacement per minute,  $4 \times 0.111 \times 870 = 386$  cu ft

Specified capacity, 315 cu ft

Volumetric efficiency,  $\frac{315}{386} \times 100 = 81.6\%$

**Intercoolers.** Intercoolers frequently are installed between the stages of a compressor to reduce the temperature of the air and to remove moisture from the air. The reduction in temperature prior to additional compression can reduce the total power required by as much as 10 to 15 per cent. Unless an intercooler is installed, the power required by a two-stage compressor will be the same as for a single-stage compressor.

An intercooler requires a continuous supply of circulating cool water to remove the heat from the air. It will require 1.0 to 1.5 gal of water per minute for each 100 cfm of air compressed, the actual amount depending on the temperature of the water.

**Aftercoolers.** Aftercoolers are installed sometimes at the discharge side of a compressor to cool the air to the desired temperature and to remove moisture from the air. It is highly desirable to remove excess moisture from the air as it tends to freeze during expansion in air tools, and it washes the lubricating oil out of tools, thereby reducing the lubricating efficiency.

**Receivers.** An air receiver should be installed on the discharge side of a compressor to equalize the compressor pulsations and to serve as a condensing chamber for the removal of water and oil vapors. A receiver should have a drain cock at its bottom to permit the removal of the condensate. Its volume should be one-tenth to one-sixth of the capacity of the compressor. A blowoff valve, to limit the maximum pressure, is desirable.

**Loss of Air Pressure in Pipe Due to Friction.** The loss in pressure due to friction as air flows through a pipe or a hose is a factor which must be considered in selecting the size of a pipe or hose. Failure to use a sufficiently large line may cause the air pressure to drop so low that it will not satisfactorily perform the service for which it is provided.

The selection of the size of line is a problem in economy. The efficiency of most equipment operated by compressed air drops off rapidly as the pressure of the air is reduced. When the cost of lost efficiency exceeds the cost of providing a larger line, it is good economy to install a larger line. The manufacturers of pneumatic equipment generally specify the minimum air pressure at which the equipment will operate satisfactorily. However, these values should be considered as minimum and not desirable operating pressures. The actual pressure should be higher than the specified minimum.



**EXAMPLE.** The cost of lost efficiency on a project resulting from the operation of pneumatic equipment at reduced pressure is estimated to be \$1,000. The lost efficiency can be eliminated by installing a larger pipe line at an additional cost of \$600. In this instance the contractor will save \$400 by installing the larger pipe. Thus, it is good economy to use a larger pipe. However, it is not good economy to spend \$1,000 to eliminate an operating loss of \$600.

Several formulas are used to determine the loss of pressure in a pipe due to friction. The following formula has been used extensively [1]:

$$f = \frac{CLQ^2}{r \cdot d^5} \quad (10-11)$$

where  $f$  = pressure drop, psi

$L$  = length of pipe, ft

$Q$  = cu ft of free air per sec

$r$  = ratio of compression

$d$  = actual ID of pipe, in.

$C$  = experimental coefficient

For ordinary steel pipe the value of  $C$  has been found to equal  $0.1025/d^{0.31}$ . If this value is substituted in equation (10-11), we get

$$f = \frac{0.1025LQ^2}{r \cdot d^{5.31}} \quad (10-12)$$

A chart for determining the loss in pressure in a pipe is given in Fig 10-5.

**EXAMPLE.** This example illustrates the use of the chart in Fig. 10-5.

Determine the pressure loss per 100 ft of pipe resulting from transmitting 1,000 cfm of free air, at 100 psi gauge pressure, through a 4-in. standard-weight steel pipe. Enter the chart at the top at 100 psi; then proceed vertically downward to a point opposite 1,000 cfm; thence proceed parallel to the sloping guide lines to a point opposite the 4-in. pipe; thence proceed vertically downward to the bottom of the chart, where the pressure drop is indicated to be 0.225 psi.

Table 10-1 gives the loss of air pressure in 1,000 ft of standard-weight pipe due to friction. For longer or shorter lengths of pipe the friction loss will be in proportion to the length. The losses given in the table are for an initial gauge pressure of 100 psi. If the initial pressure is other than 100 psi, the corresponding losses may be obtained by multiplying the values in Table 10-1 by a suitable factor. Reference to equation (10-12) reveals that for a given rate of flow through a given size pipe the only variable is  $r$ , which is the ratio of compression, based on absolute pressures. For a gauge pressure of 100 psi,  $r = \frac{114.7}{14.7} = 7.8$ , while, for a gauge pressure of 80 psi,  $r = \frac{94.7}{14.7} = 6.44$ . The ratio of these values of  $r = \frac{7.8}{6.44} = 1.21$ . Thus, the loss for an initial pressure of 80 psi will be

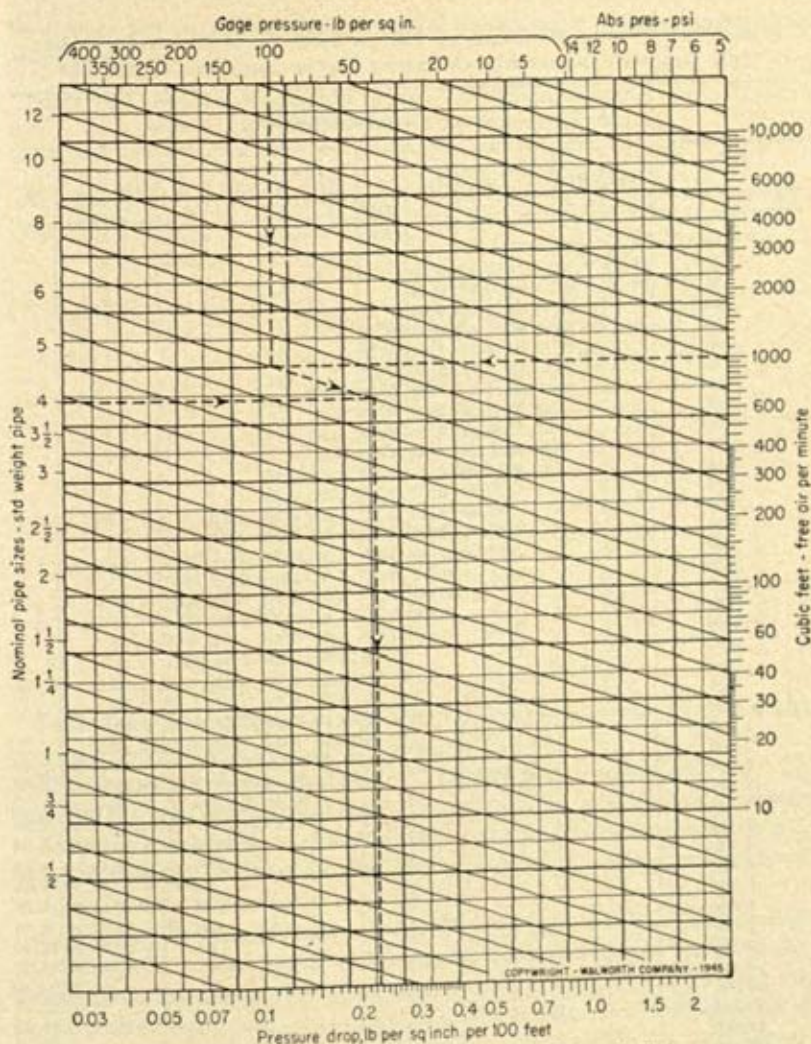


FIG. 10-5. Compressed-air-flow chart. (Walworth Co.)

1.21 times the loss for an initial pressure of 100 psi. For other initial pressures the factors are given below:

Gauge pressure, psi	Factor
80	1.210
90	1.095
100	1.000
110	0.912
120	0.853
125	0.822



TABLE 10-1. LOSS OF PRESSURE, IN PSI, IN 1,000 FT OF STANDARD-WEIGHT PIPE DUE TO FRICTION FOR AN INITIAL GAUGE PRESSURE OF 100 PSI

Free air per min, cu ft	Nominal diameter, in.												
	½	¾	1	1¼	1½	2	2½	3	3½	4	4½	5	6
10	6.50	0.99	0.28										
20	25.90	3.90	1.11	0.25	0.11								
30	68.50	9.01	2.51	0.57	0.26								
40		16.00	4.45	1.03	0.46								
50		25.10	6.96	1.61	0.71	0.19							
60		36.20	10.00	2.32	1.02	0.28							
70		49.30	13.70	3.16	1.40	0.37							
80		64.50	17.80	4.14	1.83	0.49	0.19						
90		82.80	22.60	5.23	2.32	0.62	0.24						
100			27.90	6.47	2.86	0.77	0.30						
125			48.60	10.20	4.49	1.19	0.46						
150			62.80	14.60	6.43	1.72	0.66	0.21					
175				19.80	8.72	2.36	0.91	0.28					
200				25.90	11.40	3.06	1.19	0.37	0.17				
250				40.40	17.90	4.78	1.85	0.58	0.27				
300				58.20	25.80	6.85	2.67	0.84	0.39	0.20			
350					35.10	9.36	3.64	1.14	0.53	0.27			
400					45.80	12.10	4.75	1.50	0.69	0.35	0.19		
450					58.00	15.40	5.98	1.89	0.88	0.46	0.25		
500					71.60	19.20	7.42	2.34	1.09	0.55	0.30		
600						27.60	10.70	3.36	1.56	0.79	0.44		
700						37.70	14.50	4.55	2.13	1.09	0.59		
800						49.00	19.00	5.89	2.77	1.42	0.78		
900						62.30	24.10	7.60	3.51	1.80	0.99		
1,000						76.90	29.80	9.30	4.35	2.21	1.22		
1,500							67.00	21.00	9.80	4.90	2.73	1.51	0.57
2,000								37.40	17.30	8.80	4.90	2.73	0.99
2,500								58.40	27.20	13.80	8.30	4.20	1.57
3,000								84.10	39.10	20.00	10.90	6.00	2.26
3,500									58.20	27.20	14.70	8.20	3.04
4,000									69.40	35.50	19.40	10.70	4.01
4,500										45.00	24.50	13.50	5.10
5,000										55.60	30.20	16.80	6.30
6,000										80.00	43.70	24.10	9.10
7,000											59.50	32.80	12.20
8,000											77.50	42.90	16.10
9,000												54.30	20.40
10,000													25.10
11,000													30.40
12,000													36.20
13,000													42.60
14,000													49.20
15,000													56.60

**Loss of Air Pressure through Screw-pipe Fittings.** In order to provide for the loss of pressure resulting from the flow of air through fittings, it is common practice to convert a fitting to its equivalent length of pipe having the same nominal diameter. This equivalent length should be added to the actual length of the pipe in determining losses in pressure. Table

10-2 gives the equivalent length of standard weight pipe for computing pressure losses.

TABLE 10-2. EQUIVALENT LENGTH IN FEET OF STANDARD-WEIGHT PIPE HAVING THE SAME PRESSURE LOSSES AS SCREWED FITTINGS

Nominal pipe size, in.	Gate valve	Globe valve	Angle valve	Long-radius ell or on run of standard tee	Standard ell or on run of tee	Tee through side outlet
$\frac{1}{2}$	0.4	17.3	8.6	0.6	1.6	3.1
$\frac{3}{4}$	0.5	22.9	11.4	0.8	2.1	4.1
1	0.6	29.1	14.6	1.1	2.6	5.2
$1\frac{1}{4}$	0.8	38.3	19.1	1.4	3.5	6.9
$1\frac{1}{2}$	0.9	44.7	22.4	1.6	4.0	8.0
2	1.2	57.4	28.7	2.1	5.2	10.3
$2\frac{1}{2}$	1.4	68.5	34.3	2.5	6.2	12.3
3	1.8	85.2	42.6	3.1	6.2	15.3
4	2.4	112.0	56.0	4.0	7.7	20.2
5	2.9	140.0	70.0	5.0	10.1	25.2
6	3.5	168.0	84.1	6.1	15.2	30.4
8	4.7	222.0	111.0	8.0	20.0	40.0
10	5.9	278.0	139.0	10.0	25.0	50.0
12	7.0	332.0	166.0	11.0	29.8	59.6

**Loss of Air Pressure in Hose.** The loss of pressure resulting from the flow of air through hose is given in Table 10-3.

**Recommended Sizes of Pipe for Transmitting Compressed Air.** In transmitting air from a compressor to pneumatic equipment it is necessary to limit the pressure drop along the line. If this precaution is not taken, the pressure may drop below that for which the equipment was designed and production will suffer.

At least two factors should be considered in determining the minimum size pipe. One is the necessity of supplying air at the required pressure. The other is the desirability of supplying energy, through compressed air, at the lowest total cost, considering the cost of the pipe and the cost of production obtained from the equipment. Considering the first factor, a smaller pipe may be used for a short run than for a long run. While this is possible, it may not be economical. For the latter factor, economy may dictate the use of a pipe larger than the minimum possible size. The cost of installing large pipe will be more fully justified for an installation that will be used for a long period of time than for one that will be used for a short period of time.

No book, table, or fixed data can give the correct size pipe for all installations. The correct method of determining the size pipe for a



TABLE 10-3. LOSS OF PRESSURE, IN PSI, IN 50 FT OF HOSE AND  
END COUPLINGS

Size of hose, in.	Gauge pressure at line, psi	Volume of free air through hose, cfm														
		20	30	40	50	60	70	80	90	100	110	120	130	140	150	
½	50	1.8	5.0	10.1	18.1											
	60	1.3	4.0	8.4	14.8	23.5										
	70	1.0	3.4	7.0	12.4	20.0	28.4									
	80	0.9	2.8	6.0	10.8	17.4	25.2	34.6								
	90	0.8	2.4	5.4	9.5	14.8	22.0	30.5	41.0							
	100	0.7	2.3	4.8	8.4	13.3	19.3	27.2	36.6							
	110	0.6	2.0	4.3	7.6	12.0	17.6	24.6	33.3	44.5						
¾	50	0.4	0.8	1.5	2.4	3.5	4.4	6.5	8.5	11.4	14.2					
	60	0.3	0.6	1.2	1.9	2.8	3.8	5.2	6.8	8.6	11.2					
	70	0.2	0.5	0.9	1.5	2.3	3.2	4.2	5.5	7.0	8.8	11.0				
	80	0.2	0.5	0.8	1.3	1.9	2.8	3.6	4.7	5.8	7.2	8.8	10.6			
	90	0.2	0.4	0.7	1.1	1.6	2.3	3.1	4.0	5.0	6.2	7.5	9.0			
	100	0.2	0.4	0.6	1.0	1.4	2.0	2.7	3.5	4.4	5.4	6.6	7.9	9.4	11.1	
	110	0.1	0.3	0.5	0.9	1.3	1.8	2.4	3.1	3.9	4.9	5.9	7.1	8.4	9.9	
1	50	0.1	0.2	0.3	0.5	0.8	1.1	1.5	2.0	2.6	3.5	4.8	7.0			
	60	0.1	0.2	0.3	0.4	0.6	0.8	1.2	1.5	2.0	2.6	3.3	4.2	5.5	7.2	
	70	...	0.1	0.2	0.4	0.5	0.7	1.0	1.3	1.6	2.0	2.5	3.1	3.8	4.7	
	80	...	0.1	0.2	0.3	0.5	0.7	0.8	1.1	1.4	1.7	2.0	2.4	2.7	3.5	
	90	...	0.1	0.2	0.3	0.4	0.6	0.7	0.9	1.2	1.4	1.7	2.0	2.4	2.8	
	100	...	0.1	0.2	0.2	0.4	0.5	0.6	0.8	1.0	1.2	1.5	1.8	2.1	2.4	
	110	...	0.1	0.2	0.2	0.3	0.4	0.6	0.7	0.9	1.1	1.3	1.5	1.8	2.1	
1¼	50	...	...	0.2	0.2	0.2	0.3	0.4	0.5	0.7	1.1					
	60	...	...	...	0.1	0.2	0.3	0.3	0.5	0.6	0.8	1.0	1.2	1.5		
	70	...	...	...	0.1	0.2	0.2	0.3	0.4	0.4	0.5	0.7	0.8	1.0	1.3	
	80	...	...	...	...	0.1	0.2	0.2	0.3	0.4	0.5	0.6	0.7	0.8	1.0	
	90	...	...	...	...	0.1	0.2	0.2	0.3	0.3	0.4	0.5	0.6	0.7	0.8	
	100	...	...	...	...	...	0.1	0.2	0.2	0.3	0.4	0.4	0.5	0.6	0.7	
	110	...	...	...	...	...	0.1	0.2	0.2	0.3	0.3	0.4	0.5	0.5	0.6	
1½	50	...	...	...	...	...	0.1	0.2	0.2	0.2	0.3	0.3	0.4	0.5	0.6	
	60	...	...	...	...	...	...	0.1	0.2	0.2	0.2	0.3	0.3	0.4	0.5	
	70	...	...	...	...	...	...	...	0.1	0.2	0.2	0.2	0.3	0.3	0.4	
	80	...	...	...	...	...	...	...	...	0.1	0.2	0.2	0.2	0.3	0.4	
	90	...	...	...	...	...	...	...	...	...	0.1	0.2	0.2	0.2	0.2	
	100	...	...	...	...	...	...	...	...	...	...	0.1	0.2	0.2	0.2	
	110	...	...	...	...	...	...	...	...	...	...	...	0.1	0.2	0.2	

given installation is to make a complete engineering analysis of the particular installation.

Table 10-4 gives recommended sizes of pipe for transmitting compressed air for various lengths of run. This information is useful as a guide in selecting pipe sizes.

TABLE 10-4. RECOMMENDED PIPE SIZES FOR TRANSMITTING COMPRESSED AIR AT 80 TO 125 PSI GAUGE

Volume of air, cfm	Length of pipe, ft				
	50-200	200-500	500-1,000	1,000-2,500	2,500-5,000
	Nominal size pipe, in.				
30-60	1	1	1¼	1½	1½
60-100	1	1¼	1¼	2	2
100-200	1¼	1½	2	2½	2½
200-500	2	2½	3	3½	3½
500-1,000	2½	3	3½	4	4½
1,000-2,000	2½	4	4½	5	6
2,000-4,000	3½	5	6	8	8
4,000-8,000	6	8	8	10	10

**Recommended Sizes of Hose for Transmitting Compressed Air.** Most pneumatic equipment and tools require a length of flexible hose between the source of air and the equipment. As the loss of pressure in the hose is relatively high, the length should be no greater than is required for satisfactory operation.

Table 10-5 gives the recommended sizes of hose for transmitting various quantities of compressed air and for various types of pneumatic equipment and tools frequently used on construction projects.

**Diversity or Capacity Factor.** While it is necessary to provide as much compressed air as will be required to supply the needs of all operating equipment, it is unnecessarily extravagant to provide more air capacity than will be needed. It is probable that all equipment nominally used on a project will not be in operation at any given time. An analysis of the job should be made to determine the maximum actual need prior to designing the compressed-air system.

If 10 jackhammers are nominally drilling, it is probable that not more than 5 or 6 will be consuming air at a given time. The others will be out of use temporarily for changes in bits or drill steel or moving to new locations. Thus, the actual amount of air demand will be based on 5 or 6 drills instead of 10. The same condition will apply to other pneumatic tools.



TABLE 10-5. RECOMMENDED SIZES OF HOSE, IN INCHES, FOR TRANSMITTING COMPRESSED AIR AT 80 TO 125 PSI GAUGE

Volume of air, cfm	Types of air tools	Length of hose, ft		
		0-25	25-50	50-200
0-15	Spray guns			
	¼-in. drills			
	Light chipping and scaling hammers	¾	¾	¾
	¾-in. impact wrenches			
15-30	¾-1-in. drills			
	¾-in. impact wrenches			
	Chipping hammers	¾	¾	¾
	15-lb rock drills			
30-60	¾-1 in. drills			
	¾-in. impact wrenches			
	Light grinders			
	Rivet hammers			
	Clay diggers	¾	¾	¾
	Backfill tampers			
	Small concrete vibrators			
	Light and medium demolition tools			
60-100	25-lb rock drills			
	1-2-in. drills			
	1¼-1¾-in. impact wrenches			
	Heavy grinders			
	Large concrete vibrators	¾	¾	1
	Sump pumps			
100-200	35-55-lb rock drills			
	Heavy demolition tools			
	Winches and hoists			
	Drifters			
	Wagon drills	1	1	1¼
	75-lb rock drills			

Capacity factor is the ratio of the average load to the maximum mathematical load that would exist if all tools were operating at the same time. This ratio is also referred to as a diversity factor. For example, if a jackhammer required 90 cfm of air, 10 hammers would require a total of 900 cfm if they were all operated at the same time. However, with only 5 hammers operating at one time, the demand for air would be 450 cfm. Thus, the diversity factor would be  $450 \div 900 = 0.5$ .

Table 10-6 illustrates a method of applying diversity factors to a project in which excavation is the primary operation.

**Air Required by Pneumatic Equipment and Tools.** The approximate quantities of compressed air required by pneumatic equipment and tools is given in Table 10-7. The quantities are based on continuous operation at a pressure of 90 psi gauge.

**The Cost of Compressed Air.** The cost of compressed air may be determined at the compressor or at the point of use. The former will

TABLE 10-6. ILLUSTRATION OF THE APPLICATION OF DIVERSITY FACTORS IN DESIGNING A COMPRESSED-AIR SYSTEM

Equipment	Air re- quired per unit, cfm	Number of units		Maxi- mum air de- mand, cfm	Di- ver- sity factor	Prob- able air de- mand, cfm
		On job	Work- ing			
Wagon drills.....	200	6	4	1,200	0.67	800
Jackhammers.....	100	16	8	1,600	0.50	800
Drill sharpeners.....	160	2	1	320	0.50	160
Oil furnaces.....	80	2	2	160	1.00	160
Grinders.....	50	2	1	100	0.50	50
Sump pumps.....	160	3	2	480	0.67	320
Line loss.....	..	..	..	220	....	220
Total.....	..	..	..	4,080	....	2,510
Job diversity factor.....	..	..	..	....	0.80	..
Total actual demand, $0.80 \times 2,510$ .....	..	..	..	....	....	2,008

include the cost of compressing, while the latter will include the cost of compressing plus transmitting, including line losses.

The cost of compressing should include the cost of the compressor, insurance, taxes, interest, maintenance, repair, fuel, lubrication, and labor. The cost is usually based on 1,000 cu ft of free air.

**EXAMPLE.** Determine the cost of compressing 1,000 cu ft of free air to a gauge pressure of 100 psi, using a 600-cfm two-stage portable compressor driven by a 180-hp diesel engine. The following information will apply:

Cost f.o.b. factory	= \$13,960
Freight, 11,000 lb @ \$1.50 per cwt	= 165
Total cost delivered	= \$14,125
Life, 5 yr at 2,000 hr per yr	
Average investment, $0.6 \times \$14,125$	= \$8,475
Fuel consumed per hr, full load, $0.04 \times 180$	= 7.2 gal
Lubricating oil consumed per hr, 0.125 gal	
The costs will be	



## Annual costs:

Depreciation, \$14,125 ÷ 5 = \$2,825

Repairs, 75% of \$2,825 = 2,119

Investment, 10% of \$8,475 = 848

Total annual fixed cost = \$5,792

## Hourly costs:

Fixed cost, \$5,792 ÷ 2,000 = \$2.90

Fuel, 7.2 gal @ \$0.15 = 1.08

Lubricating oil, 0.125 gal @ \$1.00 = 0.13

Operator,  $\frac{1}{2}$  time = 1.00

Total cost per hr = \$5.11

Volume of air compressed per hr, 60 × 600 = 36,000 cu ft

Cost per 1,000 cu ft, \$5.11 ÷ 36 = \$0.142

The cost per 1,000 cu ft for a compressor operating under various load factors will be as follows:

	Load factor, %		
	100	75	50
Hourly costs:			
Fixed cost*	\$2.90	\$2.90	\$2.90
Fuel	1.08	0.86	0.65
Lubricating oil	0.13	0.11	0.10
Operator	1.00	1.00	1.00
Total cost per hr	\$5.11	\$4.87	\$4.65
Volume per hr, cu ft	36,000	27,000	18,000
Cost per 1,000 cu ft	\$0.142	\$0.180	\$0.259

\* This cost might be reduced slightly with the load factor.

**The Cost of Air Leaks.** The loss of air through leakage in a transmission line can be surprisingly large and costly. It results from poor pipe connections, loose valve stems, deteriorated hose, and loose hose connections. If the cost of such leaks were more fully known, most of them would be eliminated. The rate of leakage through an opening of known size can be determined by applying a formula for the flow of air through an orifice.

Table 10-8 illustrates the cost of air leakage for various sizes of openings and costs per 1,000 cu ft of air.

**The Cost of Low Air Pressure.** The effect of operating pneumatic equipment at less than the recommended pressure can be demonstrated by analyzing the performance of a group of jackhammers. The hammers receive the air from a common pipe line.

TABLE 10-7. QUANTITIES OF COMPRESSED AIR REQUIRED BY PNEUMATIC EQUIPMENT AND TOOLS  
(Air pressure 90 psi gauge)

Equipment or tools	Capacity or size	Air consumption, cfm	
Chipping hammers	Light Heavy	15-25 25-30	
Clay diggers	Light, 20 lb Medium, 25 lb Heavy, 35 lb	20-25 25-30 30-35	
Concrete vibrators	2½-in. tube diameter 3-in. tube diameter 4-in. tube diameter 5-in. tube diameter	20-30 40-50 45-55 75-85	
Drills or borers	1-in. diameter 2-in. diameter 4-in. diameter	35-40 50-75 50-75	
Hoist	Single-drum, 2,000 lb pull Double-drum, 2,400 lb pull	200-220 250-260	
Impact wrenches	⅝-in. bolt ¾-in. bolt 1¼-in. bolt 1½-in. bolt 1¾-in. bolt	15-20 30-40 60-70 70-80 80-90	
	Weight, lb	Depth of hole, ft	
Jackhammers	10 15 25 35 45 55 75	0-2 0-2 2-8 8-12 12-16 16-24 8-24	15-25 20-35 30-50 55-75 80-100 90-110 150-175
Paving breakers	35 lb 60 lb 80 lb		30-35 40-45 50-50



TABLE 10-7. QUANTITIES OF COMPRESSED AIR REQUIRED BY PNEUMATIC EQUIPMENT AND TOOLS (Continued)

Equipment or tools	Capacity or size	Air consumption, cfm
Riveting hammers	$\frac{5}{8}$ -in. rivet	25-30
	$\frac{3}{4}$ -in. rivet	30-35
	$\frac{7}{8}$ -in. rivet	35-40
	1 $\frac{1}{8}$ -in. rivet	40-45
	1 $\frac{1}{4}$ -in. rivet	40-45
Saws:		
Circular	12-in. blade	40-60
Chain	18-30-in. blade	85-95
	36-in. blade	135-150
	48-in. blade	150-160
Reciprocating	20-in.	45-50
Spray guns	Light-duty	2-3
	Medium-duty	8-15
	Heavy-duty	14-30
Sump pump	Single-stage, 10-40 ft head	80-90
	Single-stage, 100-150 ft head	150-170
	Two-stage, 100-150 ft head	160-180
Tampers, earth	35 lb	30-35
	60 lb	40-45
	80 lb	50-60
Wagon drills—drifters	3-in. piston	150-175
	3 $\frac{1}{2}$ -in. piston	180-210
	4-in. piston	225-275

TABLE 10-8. COST PER MONTH FOR AIR LEAKAGE

Size of opening, in.	Cu ft of air lost per month, 100 psi	For indicated cost per 1,000 cu ft			
		\$0.10	\$0.15	\$0.20	\$0.25
$\frac{1}{32}$	45,500	\$ 4.55	\$ 6.83	\$ 9.10	\$ 11.38
$\frac{1}{16}$	182,300	18.25	27.40	36.50	45.65
$\frac{1}{8}$	740,200	74.00	111.00	148.00	185.00
$\frac{1}{4}$	2,920,800	292.00	438.00	594.00	730.00
$\frac{3}{8}$	6,671,900	671.00	1,006.00	1,342.00	1,677.00

**EXAMPLE.** Determine the economy of using a 3-in. pipe instead of a 2½-in. pipe to transmit air to jackhammers. The air will be supplied to the pipe at a pressure of 100 psi.

Length of pipe, 1,000 ft  
 Installed cost of 3-in. pipe, \$1,680  
 Installed cost of 2½-in. pipe, \$1,260  
 Excess cost of 3-in. pipe, \$420  
 Excess cost chargeable to this project, considering the salvage value of the pipe, \$300  
 Est. length of job, 4 months  
 Hr worked per month, 180  
 No. jackhammers on the job, 16  
 No. jackhammers operating at one time, 8  
 Size of jackhammers, 55 lb  
 Air required per hammer at 90 psi, 100 cfm  
 Total air required,  $8 \times 100 = 800$  cfm  
 Loss of pressure in 3-in. pipe, from Table 10-1,  $1.0 \times 5.89 = 5.9$  psi  
 Loss of pressure in 50 ft of ¾-in. hose, from Table 10-3 = 4.7 psi  
 Total loss in pressure, through pipe and hose, 10.6 psi  
 Pressure at hammer,  $100 - 10.6 = 89.4$  psi  
 Air required per hammer at 80 psi, 90 cfm  
 Total air required,  $8 \times 90 = 720$  cfm  
 Loss of pressure in 2½-in. pipe,  $1.0 \times 13.7 = 13.7$  psi  
 Loss of pressure in 50 ft of ¾-in. hose, 4.4 psi  
 Total loss in pressure through pipe and hose, 18.1 psi  
 Pressure at hammer,  $100 - 18.1 = 81.9$  psi  
 Hammer efficiency at 89.4 psi, 100%  
 Hammer efficiency at 81.9 psi, 85%  
 Assume cost of air to be \$0.15 per 1,000 cu ft  
 Consider effect of using each size of pipe, based on 1 hr of operation

	Size of pipe, in.	
	2½	3
Volume of air consumed, cu ft.....	43,200	48,000
Cost of air @ \$0.15 per 1,000 cu ft.....	\$6.48	\$7.20
Cost of labor, 20 men @ \$2.00 per hr.....	40.00	40.00
Cost of jackhammers*.....	8.00	9.60
Total cost per hr.....	\$54.48	\$56.80
Hammer efficiency, %.....	85	100

\* The hourly cost of a jackhammer is increased from \$0.50 to \$0.60 to provide for the extra cost of bits resulting from faster drilling.

Increase in production by using 3-in. pipe, 15%  
 Value of increased production, 15% of \$54.48 = \$8.17  
 Increase in cost per hr using 3-in. pipe,  $\$56.80 - \$54.48 = 2.32$   
 Net value of increased production, per hr = \$5.85  
 Total No. hours for job, 4 months @ 180 hr = 720  
 Total value of increased production, 720 hr @ \$5.85 = \$4,212



Thus, it may be seen that by spending \$300 more to install a larger pipe the cost of drilling can be reduced by approximately \$4,212. This example illustrates the importance of analyzing a compressed-air system. The same type of analysis may be applied to compressed-air systems designed for other uses.

### PROBLEMS

**10-1.** An air compressor draws in 315 cfm of air at a gauge pressure of 0 psi and a temperature of 70 deg. After compression, the pressure is 100 psi, and the temperature is 140 degrees. The atmospheric pressure is 14.26 psi. Determine the volume of air, in cfm, after it is compressed.

**10-2.** Determine the theoretical horsepower required to compress 315 cfm of free air, measured at standard conditions, from atmospheric pressure to 100 psi gauge pressure, when the compression is performed under isothermal conditions.

**10-3.** Use the chart in Fig. 10-5 to determine the pressure loss in 1,000 ft of 6-in. pipe when 2,000 cfm of free air is transmitted at an initial gauge pressure of 150 psi.

**10-4.** Determine the pressure loss in 850 ft of 4-in. pipe when 1,500 cfm of free air is transmitted at an initial gauge pressure of 120 psi.

**10-5.** A 2-in. pipe with screwed fittings is used to transmit 300 cfm of free air at an initial pressure of 110 psi. The pipe line includes the following items:

- 400 ft of pipe
- 2 gate valves
- 4 standard ells

Determine the total loss of pressure through this pipe.

**10-6.** The pipe line of Prob. 10-5 is used to supply compressed air to three rock drills, each requiring 90 cfm of air. Each drill is connected to the end of the pipe through 50 ft of  $\frac{3}{4}$ -in. hose. Determine the air pressure at the drills when all drills are operating simultaneously.

**10-7.** Determine the probable cfm of air required to operate the following equipment on a construction project:

- 4 wagon drills, 3-in. piston
- 8 jackhammers, 55 lb
- 4 clay diggers, medium
- 6 earth tampers, 60 lb
- 4 sump pumps, single-stage, 100-ft head

Apply a diversity factor to each type of equipment and to the entire project.

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## CHAPTER 11

### DRILLING AND BLASTING ROCK

#### DRILLING ROCK

**General Information.** Prior to excavating rock, it is necessary to loosen it in order that the excavating equipment may handle it. The loosening is done by drilling holes, filling them with explosives and discharging the explosives.

Various types of drilling equipment are used to drill the holes, the type selected depending on the size of the project, the nature of the terrain, the kind of rock, the depth and size of holes, the type of rock to be produced, such as aggregate or dimension blocks, etc. More detailed information on the selection of drilling equipment will be given later in this chapter.

**Definitions of Terms.** Terms which are commonly used in describing drilling equipment and procedures are given below as a guide for the reader.

*Percussion Drill.* This is a drill which breaks rock into small particles by the impact from repeated blows.

*Abrasion Drill.* This drill grinds rock into small particles through the abrasive effect of a bit that rotates in the hole.

*Cuttings.* Cuttings are the disintegrated rock particles that are removed from a hole.

*Jackhammer, or Sinker.* This device is an air-operated percussion-type drill that is small enough to be handled by a man.

*Drifter.* A drifter is an air-operated percussion-type drill, similar to a jackhammer, but so large that it requires mechanical mounting.

*Wagon Drill.* This is a drifter mounted on a mast supported by two or more wheels.

*Stoper.* A stoper is an air-operated percussion-type drill, similar to a drifter, that is used for overhead drilling, as in a tunnel.

*Churn Drill.* The churn drill is a percussion-type drill consisting of a long steel bit that is mechanically lifted and dropped to disintegrate the rock. It is used to drill deep holes, usually 6 in. in diameter or larger.

*Blast-hole Drill.* This is a rotary drill consisting of a steel-pipe drill stem on the bottom of which is a roller bit that disintegrates the rock as it rotates over it. The cuttings are removed by a stream of compressed air.



**Shot Drill.** This is a rotary abrasive-type drill whose bit consists of a section of steel pipe with a roughened surface at the bottom. As the bit is rotated under pressure, chilled-steel shot are supplied under the bit to accomplish the disintegration of the rock. The cuttings are removed by water.

**Diamond Drill.** The diamond drill is a rotary abrasive-type drill whose bit consists of a metal matrix in which there are embedded a large number of diamonds. As the drill rotates, the diamonds disintegrate the rock. This drill is used extensively to obtain core samples.

**Dry Drill.** This is a drill which uses compressed air to remove the cuttings from a hole.

**Wet Drill.** A wet drill is one that uses water to remove the cuttings from a hole.

**Core Drilling.** Core drilling is the obtaining of core samples of rock from a hole, usually for exploratory purposes. The diamond and shot drills are used for core drilling.

**Bit.** This is the portion of a drill which contacts the rock and disintegrates it. Many types are used.

**Detachable Bit.** This is a bit which may be attached to or removed from the drill steel or drill stem.

**Forged Bit.** This is a bit which is forged on drill steel.

**Carbide-insert Bit.** The carbide-insert bit is a detachable bit whose cutting edges consist of tungsten carbide embedded in a softer steel base.

**Diamond Bit.** The diamond bit is a detachable bit whose cutting elements consist of diamonds embedded in a metal matrix.

**Depth per Bit.** This is the depth of hole that can be drilled by a bit before it is replaced.

**Drilling Rate.** Drilling rate is the number of feet of hole drilled per hour per drill.

**Face.** Face is the approximately vertical surface extending upward from the floor of a pit to the level at which drilling is being done.

**Burden.** This is the horizontal distance from a face back to the first row of drill holes.

**Drilling Pattern.** Drilling pattern is the spacing of the drill holes.

Figure 11-1 illustrates the dimension terminology frequently used for drilling.

**Bits.** The bit is the essential part of a drill, as it is the part which must engage and disintegrate the rock. The success of a drilling operation depends on the ability of the bit to remain sharp under the impact of the drill. Many types and sizes are available.

Until recent years the bits for jackhammers and drifters were forged on one end of the drill steel. This practice has been pretty well discontinued in favor of detachable bits, which are screwed to the drill steel.

Detachable bits have many advantages compared with forged bits. They are easily replaced and resharpened, are available in various sizes, shapes, and hardness, and are relatively inexpensive. They are usually resharpened on a grinder.

Steel bits for jackhammers and drifters are illustrated in Fig. 11-2. They are available in sizes from 1 to 4½ in., the gauge size varying in steps of ⅛ in. These bits may be resharpened two to six times.

The depth of hole that can be drilled with a steel bit will vary from a few inches to 30 or 40 ft or more, depending on the type of rock.

**Carbide-insert Bits.** Some types of rock are so abrasive that steel bits must be replaced after they have drilled only a few inches of hole.

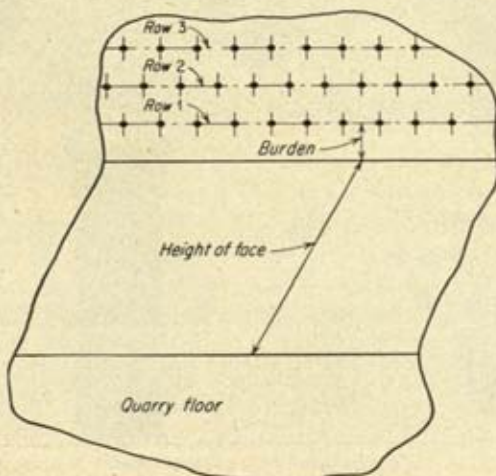


FIG. 11-1. Dimensional terminology for drilling rock.

The cost of the bits and the time lost in changing are so great that it will usually be economical to use carbide-insert bits. This bit is illustrated in Fig. 11-3. As noted in the figure, the actual drilling points consist of a very hard metal, tungsten carbide, which is embedded in steel. Although these bits are considerably more expensive than steel bits, the increased drilling rate and depth of hole obtained per bit will give an over-all economy in drilling hard rock.

In drilling hard granite for a highway project in Idaho, 2 in. carbide bits averaged 25 to 30 ft of hole per bit.

A contractor on a highway project in Pennsylvania found that when drilling diabase rock the depth per steel bit was ½ to 2 in. When he changed to carbide bits, he obtained an average depth per bit of 1,992 ft. The estimated saving by using carbide bits, in drilling 300,000 cu yd of



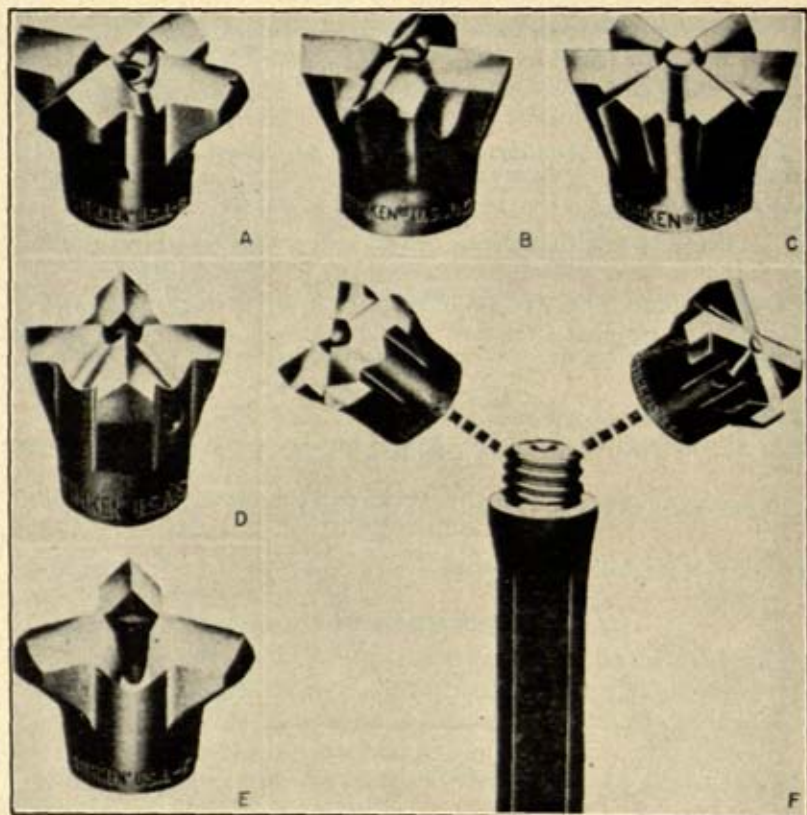


FIG. 11-2. Multiuse rock bits. (Timken Roller Bearing Co.)

rock, was in excess of \$100,000. The cost analysis for this project is as follows:

Total quantity of rock, 300,000 cu yd  
 Depth of holes, 12 ft  
 Size of bits,  $2\frac{1}{4}$  in.  
 Cu yd of rock per ft of hole,  $2\frac{1}{2}$   
 Total depth of hole required,  $300,000 \div 2.5 = 120,000$  ft  
 Average depth of hole per steel bit, with resharpening, 0.48 ft  
 No. bits required,  $120,000 \div 0.48 = 250,000$   
 Cost of bits, 250,000 @ \$0.45 each = \$112,500  
 Average depth of hole per carbide bit, 1,992 ft  
 No. bits required,  $120,000 \div 1,992 = 60$   
 Cost of bits, 60 @ \$15.90 each = \$954.00  
 Saving through the use of carbide bits, \$111,546

This figure does not include the value of time saved in changing bits. This is an exceptional case, as such savings are not possible on all projects.

**Jackhammers.** Jackhammers are hand-held air-operated percussion-type drills which are used primarily for drilling down holes. For this reason, they are frequently called sinkers. They are classified according to their weight, such as 45 or 55 lb. A complete drilling unit consists of a hammer, drill steel, and bit. As the compressed air flows through a hammer, it causes a piston to reciprocate at a speed up to 2,200 blows per minute, which produces the hammer effect. The energy of this piston is transmitted to a bit through the drill steel. Some of the air flows through

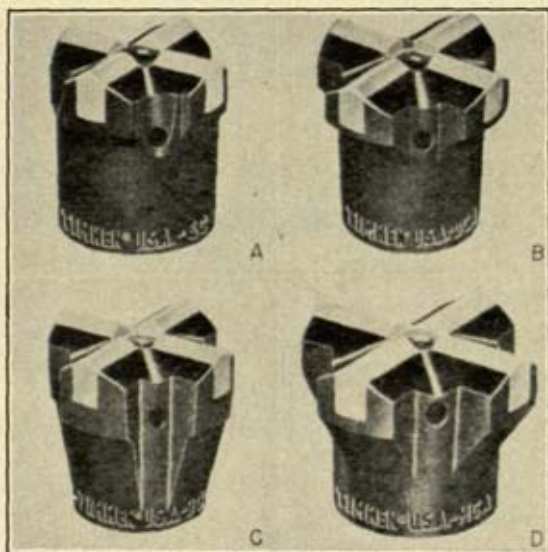


FIG. 11-3. Carbide-insert bits. (Timken Roller Bearing Co.)

a hole in the drill steel and the bit to remove the cuttings from the hole and to cool the bit. For wet drilling, water is used instead of air to remove the cuttings. Figure 11-4 shows a sectionalized jackhammer with the essential parts indicated. The drill steel is rotated slightly fol-

TABLE 11-1. REPRESENTATIVE SPECIFICATIONS FOR JACKHAMMERS

Model	S33	S55	S73
Length over-all, in.....	20 $\frac{1}{8}$	23 $\frac{3}{8}$	25
Cylinder bore, in.....	2 $\frac{3}{8}$	2 $\frac{3}{8}$	2 $\frac{3}{4}$
Weight, lb.....	31	56 $\frac{1}{2}$	67
Size steel recommended, in.....	$\frac{3}{8}$	$\frac{3}{8}$ -1	$\frac{3}{8}$ -1 $\frac{1}{4}$
Size air hose recommended, in.....	$\frac{3}{4}$	$\frac{3}{4}$ -1	$\frac{3}{4}$ -1
Size water hose recommended, in.....	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$



lowing each blow so that the points of the bit will not strike at the same spot each time.

Although jackhammers may be used to drill holes in excess of 20 ft deep, they seldom are used for holes exceeding 10 ft deep. The heavier hammers will drill holes up to 2½ in. in diameter. Drill steel usually is supplied in 2-ft-length variations, but longer lengths are available.

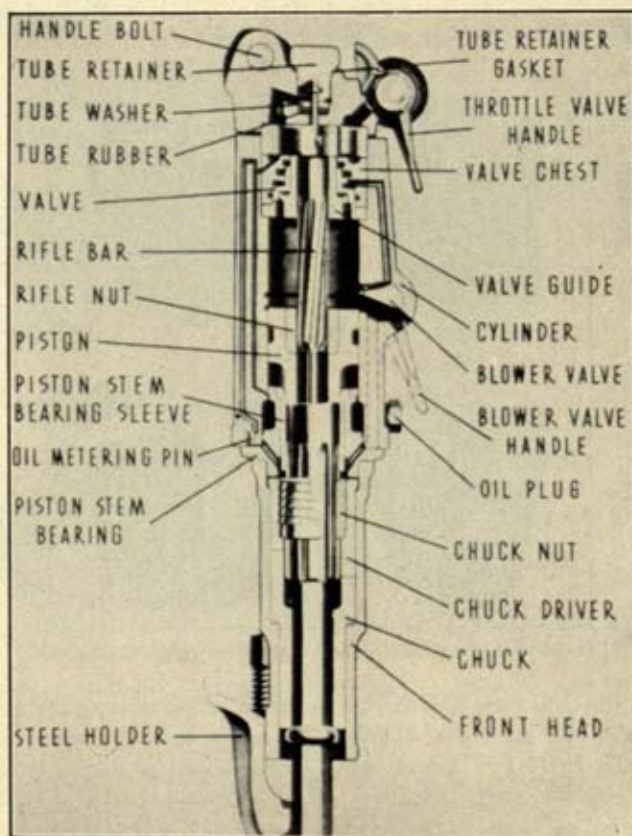


FIG. 11-4. Section through a jackhammer. (Ingersoll-Rand Co.)

**Drifters.** Drifter drills are similar to jackhammers in operation, but they are larger and are used as mounted tools for drilling down, horizontal, or up holes. They vary in weight from 75 to 260 lb and are capable of drilling holes up to 4½ in. in diameter. These tools are used extensively in mining and tunneling. Either air or water may be used to remove the cuttings.

When drifters are used for horizontal or up drilling, the feed pressure is

supplied by a hand-operated screw or a pneumatic or hydraulic piston. The weight is usually sufficient to supply the necessary pressure for down drilling. Steel changes may be obtained in lengths of 24, 30, 36, 48, and 60 in.

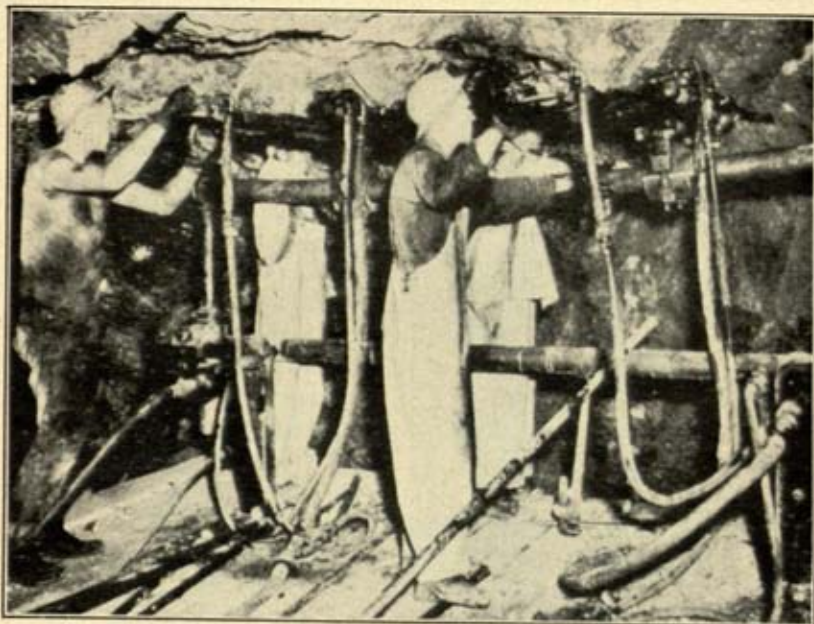


FIG. 11-5. Drifters used to drill horizontal holes. (*Western Construction.*)

TABLE 11-2. REPRESENTATIVE SPECIFICATIONS FOR AUTOMATIC-FEED DRIFTERS

Model	79	89	93	99
Cylinder bore, in. ....	3	3½	3½	4
Size chuck available, in. ....	¾-1¼	¾-1¼	¾-1¼	1¼-1½
Size air hose recommended, in. ....	1	1	1	1
Size water hose recommended, in. ....	½	½	½	½
Weight of drill, less mounting, lb. ....	111	134	140	181
Over-all length, in. ....	31¾	34	35	35½

**Wagon Drills.** Wagon drills consist of drifters mounted on masts which are mounted on wheels to provide portability. They are used extensively to drill holes up to 4½ in. in diameter and up to 30 ft or more in depth. They give better performance than jackhammers when used on terrain where it is possible for them to operate. They may be used to drill at any angle from down to slightly above horizontal. The length



TABLE 11-3. REPRESENTATIVE SPECIFICATIONS FOR WAGON DRILLS

Model	73	89	599
Height over-all.....	10 ft 2 in.	10 ft 2 in.	14 ft 9 in.
Width over-all.....	5 ft 0 in.	5 ft 7 in.	6 ft 1 in.
Length over-all.....	6 ft 2 in.	7 ft 2 in.	9 ft 10 in.
Maximum-feed travel.....	7 ft 2 in.	7 ft 11 in.	11 ft 4 in.
Length of drill steel, ft.....	6	6	10
Drill-cylinder bore, in.....	2¾	3½	4
Drill steel recommended, in.....	1¾	1¾	1¾
Size air hose recommended, in.....	1	1¾	1¾
Weight of complete unit, lb.....	813	1,535	1,615

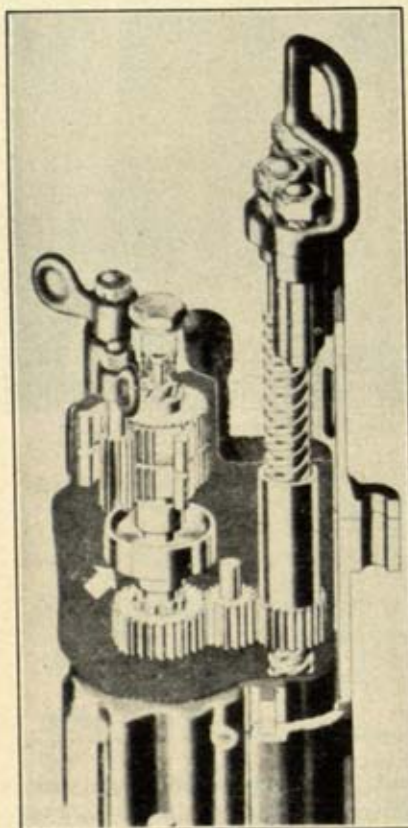
of drill steel may be 6, 10, or 15 ft. or more, depending on the length of feed of the particular wagon drill.

**Churn Drills.** A churn drill consists of a steel bit, attached to a heavy steel drill stem, which is lifted by a wire rope and dropped repeatedly in the hole being drilled. Bits are available in diameters varying from approximately 6 to 12 in. A churn drill may be used to drill rock having any degree of hardness.

The hole is filled to the desired depth with water. Then the bit, which may weigh as much as 5,000 lb, is lifted several feet and dropped. This action is repeated until a heavy slurry is formed from the cuttings. The bit is removed from the hole, and a bailer, with a foot valve, is lowered into the hole to remove the cuttings. The bit must be resharpened at intervals in order to bring it back to the required gauge.

Because of the relatively large sizes of the holes drilled, the spacing of holes may be 30 ft or more. Depths may be several hundred feet.

FIG. 11-6. The working parts of a continuous-feed drifter motor. (Gardner-Denver Co.)



In drilling holes in calcite limestone for the production of aggregate for the Bluestone Dam in West Virginia, 9-in. bits were used to drill

holes up to 230 ft deep. The holes were spaced 30 ft apart in a line 20 ft back from the face of the quarry.

Blastholes in limestone for the Bull Shoals Dam were drilled to a depth of 85 ft with 9-in. bits. The holes were spaced 33 ft apart in staggered rows whose spacing was 30 ft. The drilling rate varied from 4 to 5 ft per hour. Drills were resharpened after they had drilled 40 ft of hole.

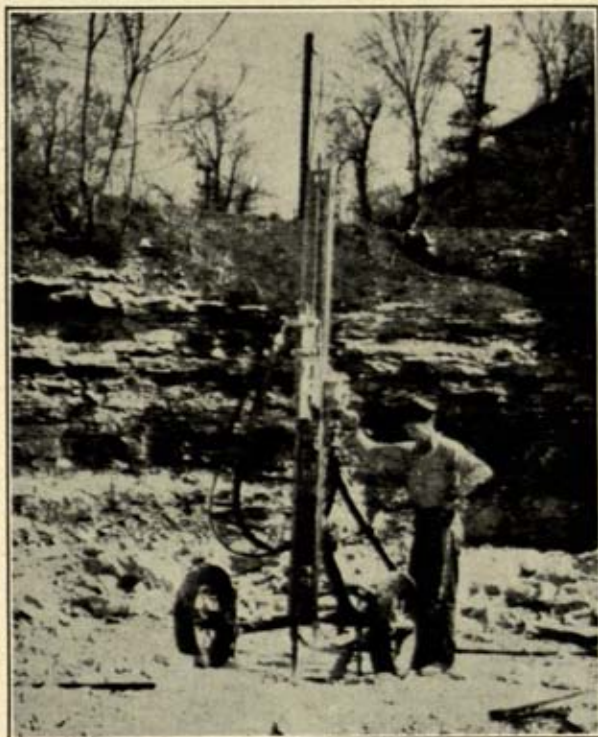


FIG. 11-7. Wagon drill. (Gardner-Denver Co.)

**Piston Drills.** A recent development in drilling equipment is the Ingersoll-Rand piston drill, whose trade name is Quarrymaster. The action of this drill is similar to that of a wagon drill, except that the drill rod, which is a hollow tube with an outside diameter of  $3\frac{3}{4}$  in., is attached to the piston and reciprocates with it. Extension rods are 35 ft long. The height of the mast is approximately 50 ft. The drill strikes approximately 200 blows per minute. The stroke and rotation of the piston are adjustable to give the best performance for the particular type of rock. The diameter of the detachable carbide-insert bit ranges from  $5\frac{1}{4}$  to 6 in. Very little loss in gauge is experienced during drilling. If the bit



is resharpened at the end of a shift, it is possible to operate with a single bit. The cost of a bit is approximately \$250.00. The rock cuttings are removed by compressed air, as for a wagon drill. A 500-cfm air compressor is mounted on the drill rig, which is self-propelled.

This drill has a practical depth limit of approximately 70 ft, which may restrict its use on some projects.

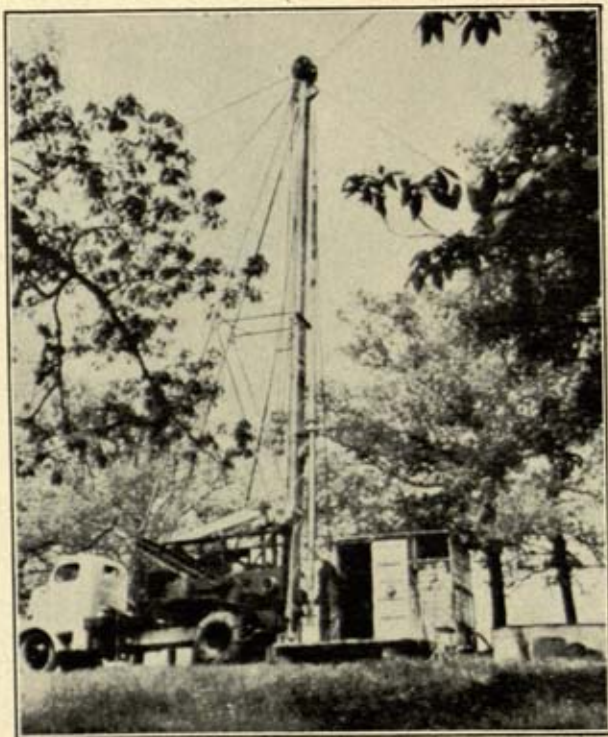


FIG. 11-8. Churn drill. (Bucyrus-Erie Co.)

The drill rig, whose weight is 36,000 lb, is mounted on crawler tracks.

In constructing the Des Joachims Dam in Canada, the contractor used 5½-in. Quarrymasters to drill holes 20 to 40 ft deep in granite-gneiss rock. For this project the performance of a Quarrymaster was equal to that of 2 to 4 churn drills or 4 to 6 wagon drills.

**Blasthole Drills.** The blasthole drill is a self-propelled drill which is mounted on a truck or on crawler tracks. Drilling is accomplished with a tri-cone roller-type bit attached to the lower end of a drill pipe. As the bit is rotated in the hole, a continuous blast of compressed air is forced down through the pipe and the bit to remove the rock cuttings and cool

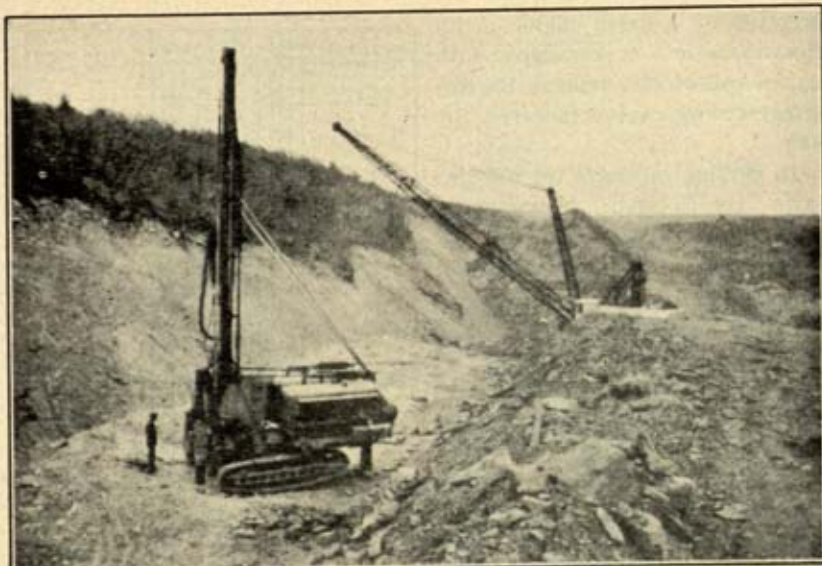


FIG. 11-9. Quarrymaster piston drill. (*Ingersoll-Rand Co.*)

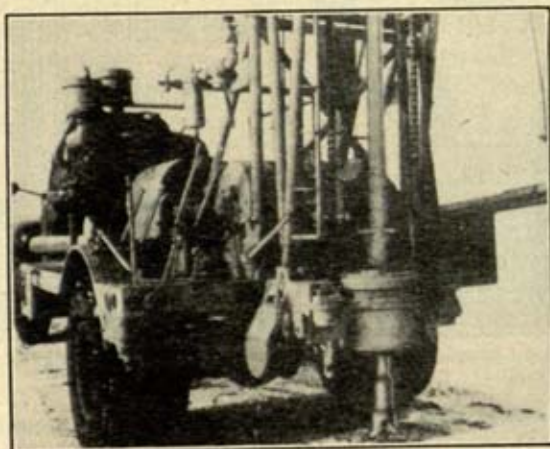


FIG. 11-10. Blasthole drill. (*Davey Compressor Co.*)

the bit. Rigs are available to drill holes to different diameters and to depths up to approximately 300 ft. This drill is suitable for drilling soft to medium rock, such as hard dolomite and limestone, but is not suitable for drilling the harder igneous rocks.

The rig developed by the Joy Manufacturing Company is equipped with leveling jacks. The standard mast is 30 ft long, to handle 20-ft-long pipe. A 40-ft-long mast, to handle 30-ft-long pipe, is optional. It is



powered by a diesel engine or an electric motor. It is equipped with a dust collector to remove the disintegrated rock as it comes from the hole.

In drilling dolomite for the Ontario Hydro-canal, heavyweight drills were used to drill 25- and 50-ft deep holes. The 25-ft holes were

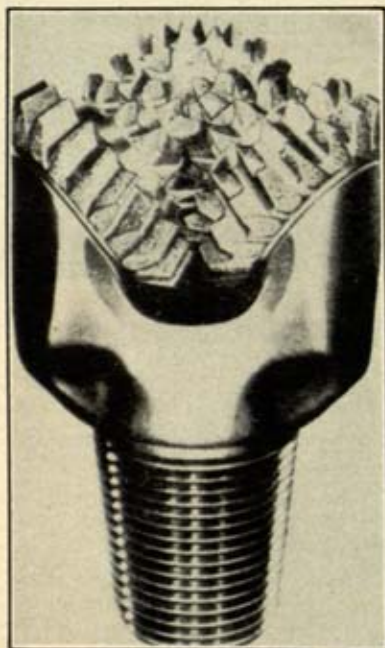


FIG. 11-11. Bit for blasthole drill. (Joy Manufacturing Co.)

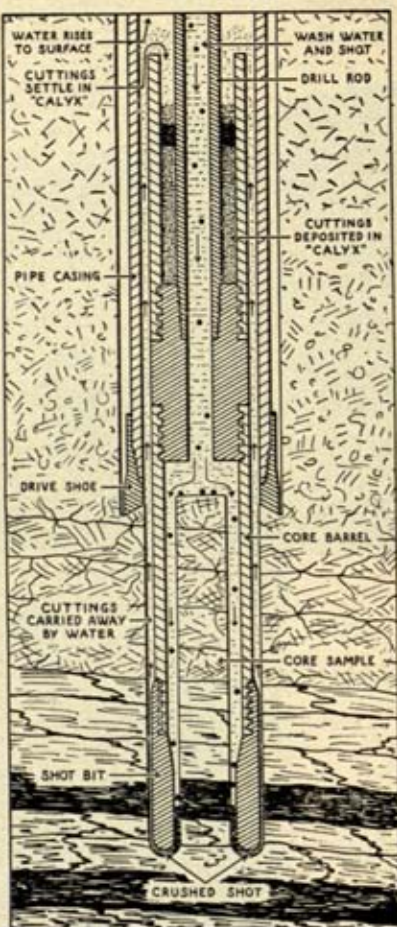


FIG. 11-12. Shot or calyx core drill. (Ingersoll-Rand Co.)

drilled on 10- by 10- and 12- by 12-ft patterns, using  $6\frac{1}{4}$ - and  $6\frac{3}{4}$ -in. bits. The average drilling speed was approximately 30 ft per hr, including moving. The life of the bits was 958 ft for the  $6\frac{1}{4}$ -in. and 1,374 ft for the  $6\frac{3}{4}$ -in. bits.

On other projects, drilling speeds have varied from  $1\frac{1}{2}$  ft per hr in dense, hard dolomite to 50 ft per hr in limestone. The speed of drilling is regulated by pressure delivered through a twin-cylinder hydraulic feed.

**Shot Drills.** A shot drill is a tool which depends on the abrasive effect of chilled steel shot to penetrate the rock. The essential parts include a shot bit, core barrel, sludge barrel, drill rod, water pump, and power-

TABLE 11-4. SPECIFICATIONS FOR JOY BLASTHOLE DRILLS

	Middleweight	Heavyweight
Size hole, in.....	5½-6¼	6¼-7¾
Weight, lb.....	35,000	46,000-50,000
Compressor motor, hp.....	75	75-125
Rotary motor, hp.....	30	50
Dust-collector motor, hp.....	5	5
Compressor capacity, cfm.....	370	370-550
Diameter feed cylinders, in.....	4	6
Length of feed, in.....	30	49
Traction speed, mph.....	1-7	1-5
Height, mast lowered.....	10 ft 9 in.	13 ft 3 in.
Length, mast elevated, ft.....	22	27
Width, jacks and dust collector removed, ft.....	8	8

driven rotation unit. The bit consists of a section of steel pipe, with a serrated lower end. As the bit is rotated, shot are fed to the lower end through the drill rod. Under the pressure of the bit these shot erode the rock to form a kerf around the core. Water, which is supplied through the drill rod, forces the rock cuttings up around the outside of the drill, where they settle in a sludge barrel, to be removed when the entire unit is pulled from the hole. Periodically it is necessary to break the core off and remove it from the hole in order that drilling may proceed.

Figure 11-12 illustrates a type of shot drill that has been used extensively. Figure 11-13 illustrates a drilling unit which is used to rotate the bit. The drive is through the spindle extending below the drill head.

Standard shot drills are capable of drilling holes up to 600 ft or more in depth, with diameters varying from 2½ to 20 in.\* Special equipment has been used to drill holes up to 6 ft in diameter with depths in excess of 1,000 ft. Rock of any hardness may be drilled.

Although large holes are expensive, they permit a man to be lowered into them for a thorough examination of the formation in place. For this purpose holes 30 in. in diameter or larger are sometimes drilled. Smaller holes provide continuous cores for examination for structural information.

The rate of drilling with a shot drill is relatively slow, sometimes less than a foot per hour, depending on the size of the drill and the hardness of the rock.

**Diamond Drills.** Diamond drills are used primarily for exploration drilling, where cores are desired for the purpose of studying the rock structure. The Diamond Core Drill Manufacturer's Association lists four sizes as standard, 1½, 1¾, 2¾ and 3 in. Larger sizes are available, but the investment in diamonds increases so rapidly with an increase in size that shot drills may be more economical for larger diameter holes.



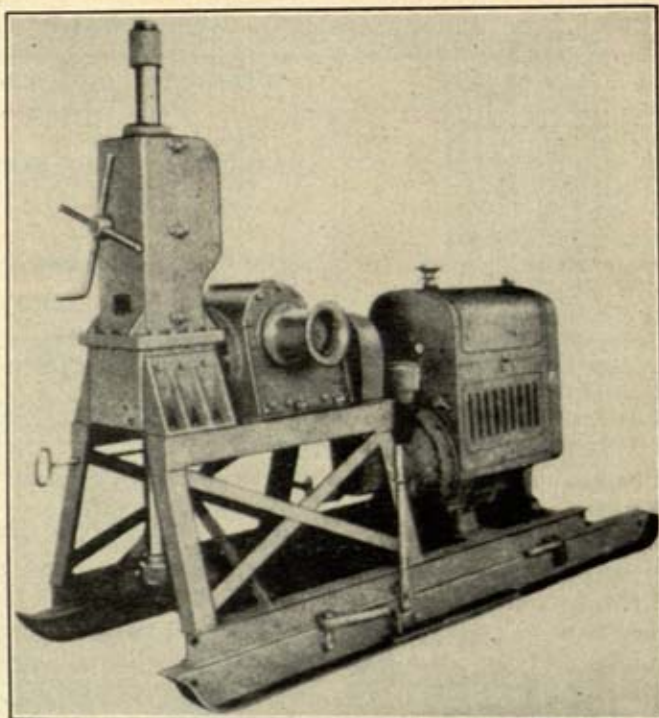


FIG. 11-13. Drilling unit for shot core drill. (Acker Drill Co.)

TABLE 11-5. REPRESENTATIVE CAPACITIES FOR SHOT DRILLS\*

Nominal bit size, in.	Core diameter, in.	Depth, ft
2½	1¾	600
3½	2¾	500
4½	3¾	350
5½	4¾	300
6½	5¾	200
18	17¾	40

\* Courtesy Acker Drill Co.

A drilling rig consists of a diamond bit, a core barrel, a jointed driving tube, and a rotary head to supply the driving torque. Water is pumped through the driving tube to remove the cuttings. The pressure on the bit is regulated through a screw or hydraulic-feed swivel head. Core barrels are available in lengths varying from 5 to 15 ft. When the bit advances to a depth equal to the length of the core barrel, the core is



FIG. 11-14. Diamond-point bits. (Sprague & Henwood, Inc.)

broken off and the drill is removed from the hole. Diamond drills can drill in any desired direction from vertically downward to upward.

The selection of the size of diamonds depends on the nature of the formation to be drilled. Large stones are preferred for the softer formations and small stones for fine-grained solid formations.

Diamond drills are capable of drilling to depths in excess of 1,000 ft. Bit speeds may vary from approximately 200 to 1,200 rpm. The drilling rate will vary from less than a foot to several feet per hour, depending on the type of rock.

Table 11-6 gives information on the dimensions, diamond content, and approximate cost of diamond bits. The cost of diamonds varies with the quality and quantity used with the bit.

**Fusion Piercing.** A recent development in drilling holes for blasting purposes is the fusion-piercing method. Fusion piercing is produced by

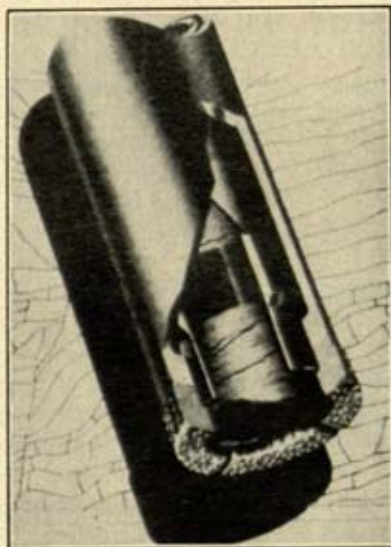


FIG. 11-15. Diamond coring bit and double-tube core barrel. (Sprague & Henwood, Inc.)



TABLE 11-6. REPRESENTATIVE INFORMATION FOR STANDARD DIAMOND CORING BITS

Size of bit, in.	Nominal		Net dimension		Minimum carat content	Approximate cost of diamonds	Approximate cost of bit
	Hole diameter, in.	Core diameter, in.	OD, in.	ID, in.			
EX	1½	¾	1.460	0.845	6.75	\$ 34-68	\$ 40-80
AX	1⅞	1½	1.865	1.185	10.00	50-100	55-115
BX	2⅜	1⅝	2.330	1.655	14.00	70-140	80-160
NX	3	2½	2.945	2.155	18.00	90-180	100-200
2¾ × 3⅞	3⅞	2¾	3.840	2.690	36.00	180-360	210-400
4 × 5½	5½	4	5.435	3.970	60.00	300-600	360-675
6 × 7¾	7¾	6	7.655	5.970	90.00	450-900	550-1,020

burning a mixture of oxygen and a flux-bearing fuel, such as kerosene, at the end of a blowpipe. When the flame is directed against the rock, the high temperature, about 4,000°F, causes some types of rock to spall, or flake off. The flux in the fuel causes other types of rock to melt. A water spray, which is directed on the heated rock, quenches the rock into small fragments, which are blown out of the hole by the expanding steam. This process was developed by the Linde Air Products Company.

In field tests made on taconite at the Mesabi Iron Range 6-in.-diameter holes 30 ft deep were drilled at an average rate of 10 ft per hr, compared with a rate of about 1 ft per hr by other drilling methods.

In drilling 9-in.-diameter holes in coarse granite the drilling rate was 20 ft per hr. The hourly consumption of the drill was 2,800 cu ft of oxygen, 300 gal of water, and 65 lb of fuel.

The drilling rate in tough diabase, for 3½-in. holes, was about 15 ft per hr. The hourly consumption of the drill was 2,600 cu ft of oxygen, 420 gal of water, and 12 gal of kerosene.

**Selecting the Drilling Method and Equipment.** Holes are drilled for various purposes, such as to receive charges of explosives, for exploration, for the injection of grout, etc. Within practical limits the equipment which will produce the greatest over-all economy for the particular project is the most satisfactory. Many factors affect the selection of equipment. Among them are the following:

1. The nature of the terrain. Rough surfaces may dictate jackhammers, regardless of other factors.
2. The required depth of holes.
3. The hardness of the rock.

4. The extent to which the formation is broken or fractured.
5. The size of the project.
6. The extent to which the rock is to be broken for handling or crushing.
7. The availability of water for drilling purposes. Lack of water favors dry drilling.
8. The purpose of the holes, such as blasting, exploration, or grout injection.

9. The size cores required for exploration. Small cores permit the use of diamond drills, while large cores suggest shot drills.

For small-diameter holes, up to  $4\frac{1}{2}$  in. maximum, which are drilled for blasting purposes, the choice is usually between jackhammers and wagon drills. If the job is large enough, and if the terrain is such that wagon drills may be used, they usually will be more economical than jackhammers. They will drill larger and deeper holes more rapidly than jackhammers. The use of larger holes will permit a greater spacing of the holes. However, on rough terrain the use of jackhammers may be necessary, regardless of the cost.

For medium-diameter holes, from 5 to 7 in., which are drilled for blasting purposes, the choice will be between the Quarrymaster, blasthole, or churn drills. For holes up to 70 ft deep in soft to medium rock, either one of the three types may be used. For depths greater than 70 ft, the Quarrymaster is not practical. For rocks harder than limestone and dolomite, the blast-hole drill will not drill satisfactorily. Thus, for drilling medium-size holes, in excess of 70 ft deep, in the harder rocks, the churn drill probably is the most satisfactory equipment. The drilling rates usually are in the order blasthole, Quarrymaster, and churn drill, indicating fastest, intermediate, and slowest, respectively.

If small size cores are desired, up to 3 in. outside hole diameter, the diamond coring drill is the most satisfactory.

If intermediate size cores are desired, 3 to 8 in. outside diameter, the choice will be between a diamond drill and a shot drill. The drilling rate of a diamond drill probably will exceed that of a shot drill. A diamond drill can drill holes in any direction, while a shot drill is limited to drilling holes that are vertical or nearly so. The lower cost of shot bits and shot may give this drill a cost advantage over a diamond drill, especially for drilling the larger sizes of holes.

If larger size cores are desired, a shot drill should be used, as diamond drills are not practical in sizes greater than approximately 8 in. outside diameter.

**Selecting the Drilling Pattern.** The pattern selected for drilling holes to be loaded with explosives will vary with the type and size drill used, the depth of the holes, the kind of rock, the maximum size rocks permissible, and other factors.



If the holes are drilled to produce rock aggregate, the drilling pattern should be planned to produce rock pieces small enough to permit most of them to be handled by the excavator, such as a power shovel, or to pass into the crusher opening without secondary blasting. While this condition is possible, the cost of excess drilling and explosives to produce it may be so high that the production of some oversize rocks is permissible, in the interest of economy.

If small-diameter holes are spaced close together, the better distribution of the explosives will result in a more uniform rock breakage. However, if the added cost of drilling exceeds the value of the benefits resulting from better breakage, the close spacing is not justified.

As large-diameter holes permit greater explosive loading per hole, it is possible to increase the spacing between large holes and thereby reduce the cost of drilling.

In analyzing a job for drilling and blasting operations, there are three factors which should be considered. They are:

1. The cubic yards of rock per linear foot of hole
2. The number of pounds of explosive per cubic yard of rock
3. The number of pounds of explosive per linear foot of hole

The value of each of the three factors may be estimated in advance of drilling and blasting operations, but after experimental drilling operations are conducted, it probably will be desirable to modify the values to give better results.

The relationships between the three factors are illustrated in Table 11-7. The volumes of rock per linear foot of hole are based on the net depth of holes and do not include subdrilling, which frequently is necessary. The pounds of explosive per linear foot of hole are based on filling the holes completely with 60 per cent dynamite. The pounds of explosive per cubic yard of rock are based on filling each hole to 100, 75, and 50 per cent of its total capacity with dynamite. When a hole is not filled completely with dynamite, the surplus volume is filled with stemming.

**Rates for Drilling Rock.** The rates of drilling rock will vary with a number of factors such as the type and size drill, hardness of the rock, depth of holes, drilling pattern, lost time waiting for other operations, etc. While the rates of drilling given in Table 11-8 are based on actual projects, they should be used only as a guide in estimating progress for any given project. The rates are average values, including a reasonable amount of lost time for moving, setting up, changing bits, etc.

**Opening Up a Pit.** When a rock pit or quarry with a level floor is opened up, there is no vertical face to facilitate blasting. Consequently, the first charges of explosive must blow the loosened rock upward, which reduces the effectiveness of the explosive. A face should be established as quickly as practical. Figure 11-16 illustrates a drilling procedure

TABLE 11-7. DRILLING AND BLASTING DATA

Size, hole, in.	Hole pattern, ft	Area per hole, sq ft	Volume of rock per lin ft of hole, cu yd	Lb of explosive per lin ft of hole	Lb of explosive per cu yd of rock		
					Per cent of hole filled		
					100	75	50
1½	4 × 4	16	0.59	0.9	1.52	1.14	0.76
	5 × 5	25	0.93	0.9	0.97	0.73	0.48
	6 × 6	36	1.33	0.9	0.68	0.51	0.34
	7 × 7	49	1.81	0.9	0.50	0.38	0.25
2	5 × 5	25	0.93	1.7	1.83	1.37	0.92
	6 × 6	36	1.33	1.7	1.28	0.96	0.64
	7 × 7	49	1.81	1.7	0.94	0.71	0.47
	8 × 8	64	2.37	1.7	0.72	0.54	0.36
3	7 × 7	49	1.81	3.9	2.15	1.61	1.08
	8 × 8	64	2.37	3.9	1.65	1.24	0.83
	9 × 9	81	3.00	3.9	1.30	0.97	0.65
	10 × 10	100	3.70	3.9	1.05	0.79	0.53
	11 × 11	121	4.48	3.9	0.87	0.65	0.44
4	8 × 8	64	2.37	7.5	3.16	2.37	1.58
	10 × 10	100	3.70	7.5	2.03	1.52	1.02
	12 × 12	144	5.30	7.5	1.42	1.06	0.71
	14 × 14	196	7.25	7.5	1.03	0.77	0.52
	16 × 16	256	9.50	7.5	0.79	0.59	0.40
5	12 × 12	144	5.30	10.9	2.05	1.54	1.02
	14 × 14	196	7.25	10.9	1.50	1.13	0.75
	16 × 16	256	9.50	10.9	1.15	0.86	0.58
	18 × 18	324	12.00	10.9	0.91	0.68	0.46
	20 × 20	400	14.85	10.9	0.73	0.55	0.37
6	12 × 12	144	5.30	15.6	2.94	2.20	1.47
	14 × 14	196	7.25	15.6	2.05	1.54	1.02
	16 × 16	256	9.50	15.6	1.64	1.23	0.82
	18 × 18	324	12.00	15.6	1.30	0.97	0.65
	20 × 20	400	14.85	15.6	1.05	0.89	0.53
	24 × 24	576	21.35	15.6	0.73	0.55	0.37
9	20 × 20	400	14.85	35.0	2.36	1.77	1.18
	24 × 24	576	21.35	35.0	1.64	1.23	0.82
	28 × 28	784	29.00	35.0	1.21	0.91	0.61
	30 × 30	900	33.30	35.0	1.05	0.79	0.53
	32 × 32	1,024	37.90	35.0	0.92	0.69	0.46



TABLE 11-8. REPRESENTATIVE RATES OF DRILLING ROCK WITH VARIOUS TYPES OF DRILLS

Size hole, in.	Class of rock	Range of drilling speeds, ft per hr						
		Jack-hammer	Wagon drill	Churn drill	Piston drill	Blast-hole drill	Diamond drill	Shot drill
1¾	Soft	15-20	30-35	...	.....	.....	5-8	
	Medium	10-15	25-30	...	.....	.....	3-5	
	Hard	5-10	15-25	...	.....	.....	2-4	
2½	Soft	10-15	30-50	...	.....	.....	5-8	
	Medium	7-10	20-30	...	.....	.....	3-5	
	Hard	4-8	12-22	...	.....	.....	2-4	
3	Soft	.....	30-50	...	.....	.....	4-7	2-3
	Medium	.....	15-30	...	.....	.....	3-5	1-2
	Hard	.....	5-15	...	.....	.....	2-4	½-1
4	Soft	.....	10-20	...	.....	.....	3-6	2-3
	Medium	.....	5-10	...	.....	.....	2-4	1-2
	Hard	.....	2-6	...	.....	.....	1-3	½-1
6	Soft	.....	.....	4-7	10-20	30-50	3-5	1½-2½
	Medium	.....	.....	2-5	8-15	10-25	2-4	¾-1½
	Hard	.....	.....	1-2	5-10	6-10	1-3	½-1
9	Soft	.....	.....	3-6	.....	.....	...	1-2
	Medium	.....	.....	2-4	.....	.....	...	¾-1
	Hard	.....	.....	1-2	.....	.....	...	½-¾
12	Soft	.....	.....	...	.....	.....	...	¾-1½
	Medium	.....	.....	...	.....	.....	...	½-1
	Hard	.....	.....	...	.....	.....	...	½-¾
30	Soft	.....	.....	...	.....	.....	...	¾-1
	Medium	.....	.....	...	.....	.....	...	½-¾
	Hard	.....	.....	...	.....	.....	...	¼-½

which will give good results toward establishing a face. The holes in rows 1 and 2 are drilled, loaded, and shot. If the discharge of the explosive in the holes in rows 2 is delayed a short interval, until the rock in the wedge between the holes in rows 1 is loosened, the explosives in the holes in rows 2 will be more effective. As soon as the broken rock between the rows 2 of holes is removed, there will be a face on each side of the pit. These faces may be used for future operations.

**Analyzing a Project for Drilling Purposes.** When an analysis of a project is made for the purpose of selecting drilling equipment, the information available usually includes the kind of rock, the depth of holes, the quantity of rock to be blasted per hour, and the maximum permissible sizes of rock. With this information available the problem will be to select the drilling equipment and hole size and spacing for the most economical production of rock.

**EXAMPLE.** A quarry in hard limestone is to supply 200 tons of rock per hour to a crushing plant. The primary crusher is a 30- by 42-in. jaw crusher, which will handle rocks whose maximum size is approximately 28 in. It is estimated that it will require approximately 0.75 lb of dynamite per cubic yard of rock to produce the desired fracture. The height of the face will vary from 25 to 30 ft. Select the equipment most suitable for drilling the rock.

The weight of limestone in place is 4,500 lb per cu yd. Thus, the required output of the quarry must be  $200 \text{ tons} \div 2.25 \text{ tons per cu yd} = 89 \text{ cu yd per hr}$ .

Experience with this type of project indicates that the boreholes can be filled with dynamite to 75 per cent of their capacity.

From reference to Table 11-7 it is seen that if wagon drills are used the required results can be obtained with 2-, 3-, or 4-in. holes. If a 6-in. drill is used, the results will be as given in Table 11-9 (see Table 11-8 for rates of drilling). The required linear feet of hole per hour are obtained by dividing 89 cu yd of rock per hour by the volume of rock per foot of hole.

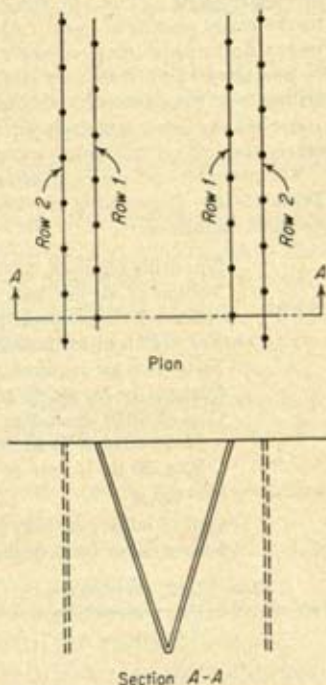


FIG. 11-16. Representative drilling pattern.

TABLE 11-9. ANALYSIS TO SELECT HOLE PATTERN AND DRILL

Type of drill	Size hole, in.	Drilling pattern, ft	Vol. of rock per ft of hole, cu yd	Required lin ft of hole per hr	Rate of drilling, ft per hr per drill	No. drills required
Wagon.....	2	7 × 7	1.81	49.2	20	3
	3	10 × 10	3.70	24.0	15	2
	4	14 × 14	7.25	12.3	8	2
Quarrymaster.....	6	20 × 20	14.85	6.0	10	1
Churn.....	6	20 × 20	14.85	6.0	3	2
Blast-hole.....	6	20 × 20	14.85	6.0	15	1



It appears that either of three sizes of wagon drills can be used. Two drills will be required for 3- or 4-in. holes. Each size will give approximately the same surplus drilling capacity. The 3-in. size probably will give a more uniform rock fracture due to the closer spacing of holes. Also, the 3-in. holes will permit the use of smaller size wagon drills, which may require less compressed air. If 2-in. holes are drilled, it will be necessary to provide three drills, which probably will result in an increase in the drilling cost compared with the drilling of 3-in. holes.

Among the larger size drills the Quarrymaster appears to fit the requirements more nearly than either the churn or the blasthole drill.

The probable cost per hour using the wagon drills and the Quarrymaster is given in Table 11-10. Considering the 2-in. hole size and wagon drills, the cost per hour is obtained as follows:

No. drills required, 3	
Volume of air per drill, 175 cfm	
Assume 2 drills operating at one time	
Volume of air for 2 drills, 350 cfm	
Volume of air required per hr, $350 \times 60 = 21,000$ cu ft	
Cost per hr for air @ \$0.20 per 1,000 cu ft = \$4.20	
Cost of drills, including bits	
Drills only, 3 @ \$1.20	= \$3.60
Bits, 50 lin ft hole @ \$0.06 per ft = 3.00	
Total	= \$6.60
Cost of labor per drill, 2 men @ \$2.00 per hr, average = \$4.00	
Cost of labor for 3 drills, $3 \times \$4.00 = \$12.00$	

TABLE 11-10. APPROXIMATE COST PER HOUR FOR DRILLING BLASTHOLES

Type of drill	Size hole, in.	No. drills required	Volume of air required		Cost per hr			
			Cfm	Cu ft per hr	Air	Drill	Labor	Total
Wagon.....	2	3	350	21,000	\$4.20	\$ 6.60	\$12.00	\$22.80
	3	2	350	16,000	3.20	4.60	8.00	15.80
	4	2	500	15,000	3.00	4.96	8.00	15.96
Quarrymaster	6	1	500	.....	.....	15.25	5.00	20.25

The analysis in Table 11-10 indicates that the 3- and 4-in. holes, drilled by wagon drills, will give the lowest drilling costs. As the difference between the two costs is not significant, either size could be used. However, because of the slightly lower cost and the probability of a more uniform fracture of the rock, the 3-in. holes seem more desirable and should give better results.

This example illustrates a method of analyzing a project for the purpose of selecting drilling equipment. Other projects may be analyzed in a similar manner.

### BLASTING

The operation referred to as blasting is performed to loosen rock in order that it may be excavated or removed from its existing position.

Blasting is accomplished by discharging an explosive that has been placed in a hole specially provided for this purpose. The energy associated with an explosion is the result of the pressure produced in the gases that are formed by the explosive.

There are many types of explosives and methods of using them. A full treatment of each explosive and method is too comprehensive for inclusion in this book. For more complete discussions of this subject the reader is referred to handbooks on blasting, published by manufacturers of explosives.

**Definitions of Terms.** The more common terms which are used in describing blasting operations are given below as a guide for the reader.

*Blasting.* Blasting is the discharging of an explosive to loosen rock.

*Explosive.* An explosive is a compound of chemical elements which, under favorable conditions, will burn or detonate quickly to produce a high pressure.

*Low Explosive.* A low explosive is one that produces pressure by progressive burning. As indicated, the release of energy occurs over a period of time.

*High Explosive.* A high explosive is one that reacts to detonation as an extremely rapid, almost instantaneous process.

*Strength.* This term refers to the energy content of an explosive, which is an indication of the work that it is capable of doing.

*Blasting Powder.* This is a slow-burning low explosive made from salt-peter, sulfur, and charcoal. It is seldom used for blasting rock.

*Dynamite.* This is a high explosive whose primary constituent is nitroglycerin. The strength is indicated according to the per cent by weight of nitroglycerin to the total weight, such as 40 per cent.

*Gelatin Dynamite.* This is a jellylike explosive made by dissolving nitrocotton in nitroglycerin. This explosive is entirely waterproof.

*Nitramon.* This is a high explosive whose primary constituent is ammonium nitrate. It requires a special primer to detonate it, as it is not sensitive to the methods commonly used to detonate dynamite.

*Safety Fuse.* This device is a train of enclosed black powder which is used as a medium to convey a flame to an explosive. The flame will travel along the fuse at a uniform rate.

*Electric Squib.* The electric squib is a metal tube with a charge of deflagrating mixture, such as black powder, that is fired by passing an electric current through a wire bridge within the mixture. A squib is used to ignite blasting powder.

*Blasting Cap.* The blasting cap is a small metal tube loaded with a charge of sensitive explosive. The cap is detonated by a safety fuse, one end of which is firmly crimped into the tube. Blasting caps are used sometimes to detonate dynamite.



*Electric Blasting Cap.* The electric blasting cap is a small metal tube loaded with a charge of sensitive explosive. The cap is detonated by the heat produced by an electric current flowing through a wire bridge inside the cap.

*Delay Electric Blasting Cap.* The delay electric blasting cap is a cap which is designed to delay the detonation for a predetermined time after the electric current is passed through it. The period of delay may be several seconds.

*Millisecond (MS) Delay Electric Blasting Cap.* This is a cap which is designed to delay the detonation for a very short time, a small fraction of a second.

*Primacord.* This device is a high-explosive detonating fuse whose core is contained in a waterproof covering with considerable tensile strength. It is used to detonate such explosives as dynamite. It detonates at a velocity of approximately 20,000 fps.

*Primer.* This is the portion of a charge, loaded with a firing device, which initiates the explosion.

*Blasting Machine.* The blasting machine is used to generate the current for firing an explosive by electricity.

*Leading Wires.* These are wires used to conduct the electric current from its source to the leg wires from electric blasting caps.

*Dynamite Density.* This term refers to the number of 1¼- by 8-in. cartridges per 50-lb case. The number will vary from 80 to 250.

*Borehole.* This is a hole drilled to receive an explosive.

*Stemming.* Stemming is the adding of inert material, such as rock dust, in a borehole on top of an explosive to confine the energy of the explosion.

*Coyote Tunnel.* This is a tunnel, several feet in diameter, into which a large quantity of explosive is placed for detonating purposes.

*Dynamite.* Dynamite is available in many grades and sizes to meet the requirements of a particular job. The approximate strength is specified as a percentage, which is an indication of the ratio of the weight of nitroglycerin to the total weight of a cartridge. Individual cartridges vary in size from approximately 1 to 8 in. in diameter and 8 to 24 in. long.

Dynamite is used extensively for charging boreholes, especially for the smaller sizes. As it is placed in a hole, it is tamped sharply with a wooden pole to expand the cartridges so that they fill the hole. For this purpose it may be desirable to split the sides of a cartridge, or cartridges with perforated shells may be obtained. A charge may be fired by a blasting cap or a Primacord fuse. If a cap is used, it is placed in a hole made in one of the cartridges, which serves as a primer. Electric caps are supplied with two leg wires in lengths varying from 2 to 100 ft. These wires are connected with the wires from other holes to form a closed electric circuit for firing purposes.

**Stemming.** After a hole is filled with an explosive to the required depth, the balance of the hole should be filled with stemming. Stemming, which may consist of rock cuttings or other suitable inert material, confines the energy and increases the effectiveness of an explosion. If a continuous charge of explosive is not required from the bottom to the top of the charge, stemming may be placed between charges at predetermined intervals. When charges are separated with stemming, a separate primer should be provided for each charge. Figure 11-17 illustrates several methods of loading boreholes for firing with electric blasting caps.

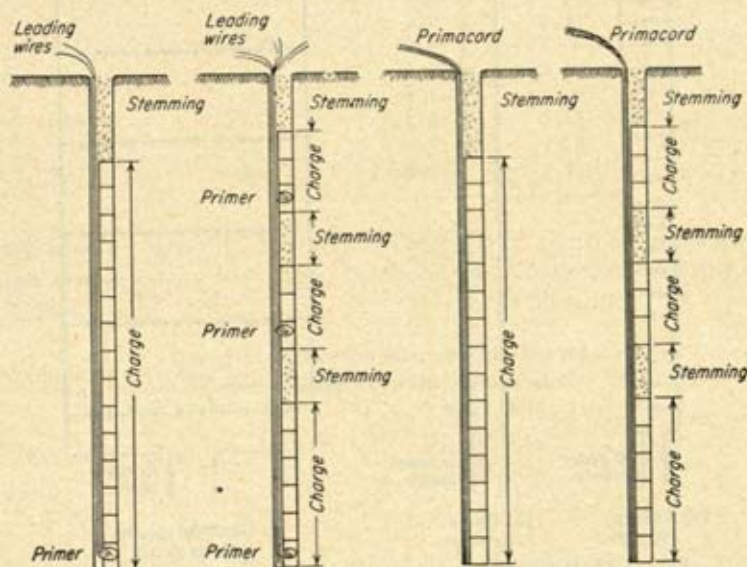


FIG. 11-17. Methods of loading boreholes with explosives.

**Firing Charges.** It is common practice to fire several holes at one time, using either parallel or series circuits or a combination thereof. Prior to making the final connection to the source of electric current, a circuit should be tested with a galvanometer in the line. Each circuit must be tested as a precaution to eliminate open breaks and misfires. Figure 11-18 illustrates three types of circuits.

In order to secure good breakage, with the desired degree of fragmentation, it frequently is necessary to place a higher concentration of explosive near the bottom of a hole than near the top. This may be done by using a strong dynamite near the bottom and a less strong one near the top, or the same effect may be obtained by separating the charges near the top with stemming, provided the total charge in a hole is adequate.



**Ammonia Nitrate Explosives.** Ammonia nitrate explosives are becoming increasingly popular for blasting purposes, especially for use in large-diameter boreholes. These explosives are supplied in cans varying in size from approximately 4 to 12 in. in diameter and from 16 to 24 in. in length. The weights vary from approximately 11 to 75 lb per can.

Ammonia nitrate explosives are much safer than dynamite. They cannot be detonated by blasting caps, Primacord, bullets, or flame. They are

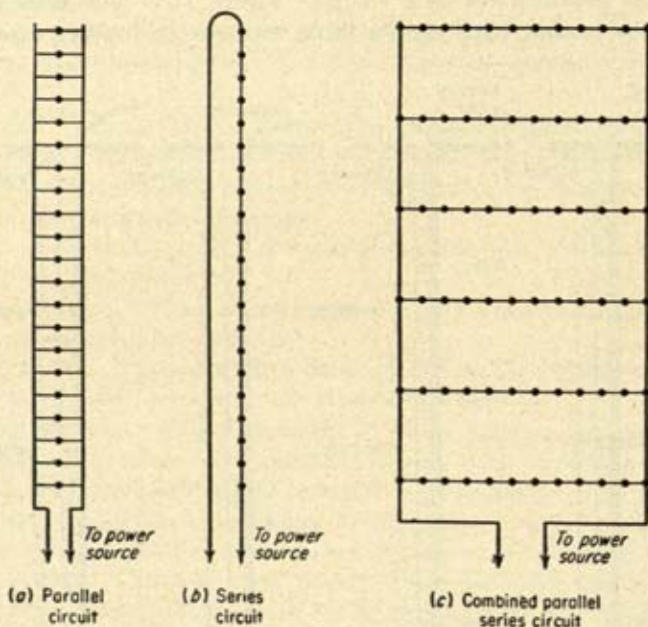


Fig. 11-18. Representative types of circuits for firing explosives.

detonated by special primers. As they are packed in sealed metal cans, they are not affected by water in the boreholes.

**Electric Blasting Caps.** Electric blasting caps are used to detonate charges of dynamite or Primacord fuse. A cap is exploded by passing an electric current through a wire bridge inside the cap. The current, which should be approximately 1.5 amp, heats the bridge, which detonates the explosive in the cap with sufficient violence to fire a charge of dynamite.

Regular-type electric blasting caps are supplied with leg wires whose lengths are indicated in Table 11-11. Number 22 gauge copper wires are used for leg lengths up to 24 ft and No. 20 gauge copper wires for lengths of 30 ft and longer.

In order to analyze an electric circuit used to fire blasting caps, it is necessary to know the resistance of the caps and the leading wires, which

TABLE 11-11. RESISTANCE OF REGULAR AND DELAY ELECTRIC BLASTING CAPS

Length of leg wires, ft	Resistance, ohms per cap	
	Regular	Delay
4	0.94	1.45
6	1.00	1.51
8	1.07	1.58
10	1.13	1.64
12	1.20	1.71
16	1.32	1.84
20	1.45	1.97
24	1.58	2.10
30	1.41	1.93
40	1.62	2.13
50	1.82	2.33
60	2.02	2.53

conduct the current to the caps. Table 11-12 gives the resistance of single-strand copper wire in the sizes most commonly used for firing electric caps.

TABLE 11-12. RESISTANCE OF COPPER WIRE

B. and S. gauge No.	Resistance, ohms per 1,000 ft
8	0.628
10	0.999
12	1.588
14	2.525
16	4.015
18	6.385
20	10.150
22	16.140

**EXAMPLE.** A total of 20 regular electric blasting caps, connected in a single series circuit, are to be fired. Determine the required voltage at the source of supply. The following information is available:

Current required to fire the caps, 1.5 amp  
 Length of leg wires per cap, 40 ft  
 Resistance per cap, 1.62 ohms  
 Distance from source of electricity to blast area, 400 ft  
 Length of leading wires,  $2 \times 400 = 800$  ft  
 Size of leading wires, No. 20 gauge  
 Combined resistance of caps,  $20 \times 1.62 = 32.4$  ohms  
 Resistance of leading wires,  $0.8 \times 10.15 = 8.1$  ohms  
 Total resistance of circuit  $= 40.5$  ohms



From Ohm's law the voltage required is obtained from the equation

$$E = IR$$

where  $E$  = volts

$I$  = current, amp

$R$  = resistance, ohms

$$E = 1.5 \times 40.5 = 60.7 \text{ volts}$$

Thus, any source of electricity that can supply at least 1.5 amp at 60.7 volts will be satisfactory. A 110-volt circuit is adequate.

**EXAMPLE.** If the blasting caps of the previous example are fired in a parallel circuit, with other conditions the same except as noted, the required voltage and current may be determined as follows:

Current required per cap, 0.5 amp

Total current required,  $20 \times 0.5 = 10$  amp

Use No. 14 gauge leading wires

Resistance of leading wires,  $0.8 \times 2.525 = 2.0$  ohms

Resistance of caps,  $1.62 \div 20 = 0.08$  ohm

Total resistance  $= 2.08$  ohms

$$E = IR$$

$$= 10 \times 2.08 = 20.8 \text{ volts}$$

**Delay Blasting Caps.** When the explosive charges in two or more rows of holes parallel to a face are fired at the same time, it is desirable to fire the charges in the holes nearest the face a short time ahead of those in the second row. This procedure will reduce the burden on the holes in the second row and thereby permit the explosive in the second row to break the rock more effectively. If there are more than two rows of holes, the detonations may progress in the order 1, 2, 3, 4, etc., where the numbers indicate the rows, starting with the row nearest the face.

Delay blasting caps are used to obtain this firing sequence. Such caps are available for delay intervals varying from a small fraction of a second to 10 or more seconds. For the shortest delay intervals the caps are called millisecond delay caps and are designated as MS-25, MS-50, MS-200, etc. The number indicates the period of delay in thousandths of a second.

**Primacord.** Primacord is a high-explosive fuse that is used to detonate dynamite or the special primers for ammonia nitrate explosives. The core, which is the explosive, is covered with a sheath for protection, strength, and waterproofing. One or two fuses should be placed to the full depth in a hole as a precaution against a misfire.

When several holes are fired at one time, the fuse from each hole is tied securely to a common fuse, which is fired by a blasting cap that is tied to the fuse. The fuse explodes with such violence that it will detonate dynamite through contact only.

**Handling Misfires.** In shooting charges of explosives, it may be that one or more charges will fail to explode. This is referred to as a misfire. It is necessary to dispose of this explosive before excavating the loosened rock. The most satisfactory method is to shoot it if possible.

If electric blasting caps are used, the leading wires should be disconnected from the source of power prior to investigating the cause of the misfire. If the leg wires to the cap are available, test the cap circuit, and if the circuit is satisfactory, try again to set off the charge.

When it is necessary to remove the stemming to gain access to a charge in a hole, it should be removed with a wooden tool instead of a metal tool. If water or compressed air is available, either one may be used with a rubber hose to wash the stemming out of the hole. A new primer, set on top of or near the original charge, may be used to fire the charge.

**Transporting and Handling Explosives.** Most states and cities have laws regulating the transporting and handling of explosives. Persons responsible for handling explosives on a project should be familiar with such laws.

The Recommendations for Storing, Handling and Transporting Explosives, issued under the Federal Explosives Act of Dec. 26, 1941, revised as of June 1, 1944, suggests safe procedures for storing, handling, and transporting explosives. Among the recommendations for transporting explosives are the following:

1. Any vehicle transporting explosives should be marked or placarded on the front end, both sides and rear with the word "Explosives" in letters not less than 4 in. in height in colors contrasting with the background; or the vehicle should carry in a conspicuous place a red flag not less than 24 in. square with the word "Explosives" in white letters at least 3 in. in height or the word "Danger" in letters 6 in. in height.

2. Vehicles should not carry blasting caps or detonators while carrying other explosives; and no metal, metal tools, oils, matches, firearms, acids, inflammable substances, or similar material should be carried on vehicles transporting explosives.

3. Vehicles transporting explosives should not be overloaded, and in no case should the explosives containers be piled higher than the closed sides of the body. Any vehicle with an open body should have a tarpaulin to cover the explosives containers.

4. All vehicles, when transporting explosives, should be inspected to determine that: The brakes and steering mechanism are in effective working condition; the electric wiring is well insulated and firmly secured; the body and chassis are clean and free from accumulations of oil and greases; the fuel tank and feed line are secure and have no leaks; two suitable fire extinguishers in working order and located near the driver's seat are pro-



vided; and, in general, the vehicle is in proper condition for safe transportation of explosives.

5. The floors of all vehicles should be tight. Any exposed metal on the inside of the body that might come in contact with any package of explosives should be covered or protected with wood or other nonmetallic material.

6. No explosives should be transported in any form of pole-type trailer, nor should any trailer be attached to a vehicle hauling explosives.

7. Passengers or other unauthorized persons should not ride on a vehicle transporting explosives. Smoking or the carrying of matches and smoker's articles should not be permitted on or around a vehicle transporting explosives.

8. Packages or containers of explosives should not be thrown or dropped while being loaded or unloaded or otherwise handled, but they should be carefully deposited and stored or placed in such a manner as to prevent the packages or containers from sliding or falling or being otherwise displaced.

9. Motors of vehicles transporting explosives should be stopped before loading or unloading the explosives.

The recommendations for handling explosives are as follows:

1. Cases or kegs containing explosives should always be lifted and set down carefully and never slid over one another or dropped from one level to another or otherwise roughly handled.

2. Containers of explosives should not be opened inside a magazine nor within 50 feet of a magazine.

3. Tools made of wood or other nonmetallic material should be used in opening boxes or kegs or other containers of explosives. Metallic tools should not be used.

4. Explosives and detonators issued to individual workmen should be placed in separate insulated carriers or containers equipped with lids so constructed and fastened that they can not come open during transportation.

5. No person except the attendant should be permitted to ride with explosives or detonators when they are being transported in a shaft, slope, or other underground working.

**Storing Explosives.** Explosives and detonators should be stored separately in detached, dry, ventilated, bulletproof, and fire-resistant magazines, away from other buildings, railroads, and highways. The American Table of Distances gives the safe distances between magazines and other buildings, railroads, and highways for varying quantities of detonators and explosives.

A magazine for the storage of dynamite should be constructed in such a manner that it will prevent the freezing of the dynamite during extended

periods of cold weather. If the dynamite does freeze, it should be thawed before it is handled or used, as the danger of premature firing is much greater when it is frozen.

### PROBLEMS

**11-1.** In operating a quarry for the production of crushed limestone rock, it is desirable to balance the drilling, blasting, and excavating operations in order that there will always be broken rock available for loading. The rock will be loaded into trucks by a power shovel whose capacity is 195 cu yd per hr.

Three-inch-diameter holes, 20 ft deep, will be drilled with wagon drills to produce an effective depth of 18 ft. Tests indicate that satisfactory results may be obtained by spacing the holes to give 2 cu yd of rock per foot of hole. Tests also indicate that a satisfactory rock break may be obtained with 1 lb of dynamite per cubic yard of rock.

The shovel will operate 22 hr per day, while drilling and blasting will be limited to 8 hr per day.

Determine the following:

- The proper spacing of the holes
- The number of wagon drills required
- The capacity of compressed air required, considering a diversity factor
- The number of pounds of dynamite required per hole

**11-2.** In planning the selection of drilling equipment for a project, it is proposed to use jackhammers and wagon drills. Loss in time resulting from moving to new holes, changing drill steel, etc., will limit the number of drills in operation at any given time.

The table below lists the number of units on the job and the per cent of time that each type will be drilling. If 10 per cent of the compressed air is lost through leakage in the transmission line, determine the amount of air that must be provided for the job, expressed in cfm.

Equipment	No. units at job	Quantity of air required per drill, cfm	Per cent of time in operation	Quantity of air required, cfm
Jackhammers.....	12	90	38	
Wagon drills.....	16	230	50	

**11-3.** The accompanying table gives information on drilling and blasting operations that will produce a satisfactory rock fracture. Determine which plan will give the most economical production.

Size of holes, in.	Spacing of holes, ft	Rate of drilling, ft per hr	Cost of drill per hr	Lb of dynamite required per cu yd of rock
3	6	15	\$ 7.90	0.80
4	9	8	8.20	1.25
5	12	6	16.50	1.40
6	15	10	20.35	1.60

All holes will be drilled 24 ft deep.

The cost of dynamite in place will be \$0.50 per lb.



Prepare your results in a tabular form showing size of the holes, spacing of the holes, rate of drilling, cost of drill per hour, cost of drilling per cubic yard of rock, dynamite required per cubic yard of rock, cost of dynamite per cubic yard of rock, and combined cost of drilling and dynamite per cubic yard of rock.

**11-4.** Determine the minimum voltage required to fire 30 regular electric blasting caps, connected in a single series circuit. The length of leg wires will be 30 ft. The leading wires will be No. 18 gauge, 1,200 ft long.

## CHAPTER 12

### TUNNELING

**Scope of this Subject.** The subject of tunneling is too comprehensive to permit full coverage in this book. Therefore, only the fundamentals will be presented. If more complete coverage is desired, it is suggested that the reader obtain copies of books which are devoted entirely to the subject [1].

**Purposes of Tunnels.** Tunnels are constructed for various purposes such as to provide:

1. Passageways for railroads and automotive vehicles through mountains and under bodies of water
2. Accesses to mines
3. Conduits for water

**Types of Rock.** The rocks which are encountered in tunneling operations can be divided into three major groups, igneous, sedimentary, and metamorphic. Each group can be subdivided according to origin, mineral content, physical condition, etc.

**Igneous Rocks.** Igneous rocks have cooled from molten masses which emerged through fissures from the interior of the earth. If a molten mass cooled prior to reaching the surface of the earth, the rock is defined as intrusive. Examples of intrusive rocks are granite and gabbro. If a molten mass cooled after reaching the surface of the earth, the rock is defined as extrusive. Examples of extrusive rocks are rhyolite and basalt.

**Sedimentary Rocks.** The sedimentary rocks with which the engineer is concerned include those which were deposited by flowing water, such as conglomerates, sandstones, shales, and clays, and those which were deposited by marine organisms, such as limestones and dolomites.

**Metamorphic Rocks.** If igneous or sedimentary rocks are subjected to high temperatures and pressures, they undergo changes in structure and texture. Rocks which have been subjected to such changes are described as metamorphic rocks.

Under the influence of moderate temperatures and pressures, clay and shales are transformed into slates and schists, which are low-grade metamorphic rocks. When subjected to high temperatures and pressures,



slates and schists are metamorphosed into hard and dense gneiss. Limestone metamorphoses into marble and sandstone into quartzite.

**Physical Defects of Rocks.** All rocks, regardless of the type, have physical or structural defects which have considerable effect on tunneling operations. These defects consist of fractures, whose magnitudes and spacings vary considerably. Simple fractures are defined as joints, whereas major fractures, associated with relatively large displacements, are defined as faults.

**Joints.** Joints are surfaces of physical failure or separation with little or no displacement between the rock components on opposite sides of a joint. The joints may exist in two or three planes approximately at right angles with each other.

In driving a tunnel through a rock formation, the existence of joints will affect the extent to which the sides and roof must be supported during the tunneling operation. Also, joints provide passageways through which ground water may flow into a tunnel.

**Faults.** A fault is a zone in a formation where a large displacement has occurred along the plane of failure. The displacement may be horizontal, vertical, or a combination thereof. A fault usually constitutes an undesirable hazard to tunnel driving. Because of the enormous forces that produce a fault the rock formation in the fault zone will be badly broken. The crushed material may vary in size from fine sand to large blocks, which tend to flow into a tunnel as it is driven through a fault zone. If ground water is present in the formation, the broken material within the fault zone will provide excellent passageways for the water to flow into the tunnel unless corrective steps are taken prior to excavating through the zone. It may be necessary to pressure-grout the formation ahead of the tunneling operation in order to eliminate the hazard of ground water.

**Preliminary Explorations.** While the approximate location of a tunnel is dictated by the service it is to provide, the final location should be based on the results of surface and subsurface explorations. Such explorations are made prior to selecting the exact location of a tunnel in order to determine the kinds of formation that exist and the extent to which ground water is present in the formations along the route of a proposed tunnel. The formations may include unconsolidated muck, sand, gravel, or clay, with or without ground water. There may be solid or badly broken rock, or there may be faults and folds to contend with. If a tunnel is driven through solid rock, little or no roof support may be required, whereas if it is driven through badly broken rock, it will be necessary to provide extensive wall and roof supports. If an exploration indicates the presence of significant quantities of ground water, it may be desirable to seek a more favorable location, or if this is not possible, it may be necessary to pressure-grout the formation ahead of excavation as a means of reducing

the flow of water. Plans should be made to have adequate pumps available to remove the water.

Valuable information may be obtained from a surface exploration by a competent geologist who is reasonably familiar with the area. More definite information concerning a formation may be obtained by drilling holes along the proposed route and securing samples of the formation. The holes should be drilled at least to the bottom of the proposed tunnel and should be spaced sufficiently close to give representative samples of the formation. If the formation is free of severe structural irregularities and variations, the spacing of the holes may be greater than for a formation that contains faults, folds, or other structural irregularities.

If a formation is soft enough, the holes may be drilled with earth augers or split tubes, which will permit the recovery of undisturbed samples for examination. If the formation consists of unconsolidated material, such as sand or small gravel, holes may be jetted with water. For this purpose it will be necessary to supply a reasonably large quantity of water, under pressure, and enough pipe to permit holes to be jetted to the desired depth. However, the material recovered from jetted holes may not give true samples of the formation, and the information obtained from such samples may not be sufficiently dependable for selecting the final location of a tunnel.

If a formation is rock, the holes may be drilled with wagon, churn, rotary, or other types of drills that produce cuttings. Since these drills produce cuttings instead of undisturbed samples or cores, the material recovered from the holes will not indicate whether the formation is solid or broken rock. As water must be added to holes drilled with churn drills in order to remove the cuttings, the cuttings will not indicate the extent to which ground water exists in a formation.

When cores from the exploratory holes are desired, they may be obtained with core or shot drills. Cores obtained with diamond bits usually vary in diameter from  $\frac{3}{8}$  to 4 in., while cores obtained with shot drills usually vary in diameter from about 4 to 8 in. However, larger sizes may be obtained with either bit. Large-diameter cores will permit a more intelligent analysis of the structure of the formation. Cores should be assembled in the same order as they come from the hole. In general, the length of the core recovered from a hole will be less than the depth of the hole, the length varying with the kind of rock and the degree of solidity. Typical core recoveries should be about 80 to 90 per cent for igneous rocks, 60 to 70 per cent for limestone, 70 to 80 per cent for sandstone, and 40 to 50 per cent for shale.

After the preliminary explorations have been completed and the results analyzed, the location that will permit the construction of a satisfactory tunnel at the lowest practical cost can be selected.



**Number of Entrances.** If a tunnel is relatively short, not more than a few hundred feet long, it may be driven from one entrance only. However, as the length is increased, conducting all operations from one entrance may result in excessive haul distances and high haulage costs, together with a general congestion between the portal and the head of the tunnel. Such a condition may be eliminated or alleviated by driving a tunnel from both ends. For long tunnels it may be advantageous to provide intermediate openings, such as shafts, to facilitate the removal of muck and water and the delivery of materials, supplies, air, and utilities. Intermediate shafts or openings permit operations at a greater number of headings, thus making possible an increase in the rate of driving a tunnel. This may be especially important for a project when an early completion is desirable.

**Sequence of Operations.** As soon as the construction of a tunnel is under way, the various operations should be carried on in a well-planned sequence. The actual operations will vary with the type and size tunnel, the method of attacking the heading, and the kind of formation encountered. The construction may be on the basis of one, two, or three shifts per day.

For a tunnel driven through rock the following operations might apply:

1. Setting up and drilling
2. Loading holes and shooting the explosives
3. Ventilating and removing the dust following an explosion
4. Loading and hauling muck
5. Removing ground water if necessary
6. Erecting supports for the roof and sides if necessary
7. Placing reinforcing steel
8. Placing the concrete lining

The first four operations are related to the driving of the tunnel and frequently establish the rate of progress in constructing a tunnel. Progress on the other operations should be coordinated with the rate of driving in so far as it is practical to do so.

A representative sequence of operations and the time required for each is given hereafter for a railroad tunnel at the Conemaugh Damsite [2]. The tunnel, whose bore was 36 ft wide and 32 ft high, was driven through sandstone. In driving the main bore, artificial ventilation was not necessary because a pilot tunnel had been driven the full distance prior to starting excavation for the main bore. Each round required 80 holes, 20 ft deep, which were drilled by 9 drifters, mounted on a jumbo. The muck was loaded by 1¼-cu-yd electric power shovel into narrow-gauge cars, whose capacity was 5 cu yd each. The rate of progress was approximately 20 ft per day for 2 shifts.

The time required for several operations in a cycle was as follows:

Shift	Operation	Time, hr	
		Min	Max
1	Drill the holes	5	6
	Load the holes	1	1
	Explode the dynamite		
2	Load and haul muck	9	9
Total time	.....	15	16

The contractor who drove the power tunnel for the Kemano hydroelectric project in British Columbia chose to complete a drilling cycle in each 8-hr shift [3]. The tunnel was a 25-ft horseshoe bore. Each round required 87 to 96 holes, 13 to 15 ft deep, which were drilled with 15 drifters, mounted on a jumbo. The average rate of advance was about 12 ft per shift.

A typical time for each operation and for a cycle was as follows:

Operation	Time, hr
Drill the holes	1 $\frac{3}{4}$
Load the holes	$\frac{3}{4}$
Explode the dynamite	
Ventilate during lunch	$\frac{3}{4}$
Load and haul muck	4 $\frac{3}{4}$
Total time	8

**Driving Tunnels in Rock.** There are several methods of attacking the faces of tunnels driven through rock. The method selected will depend on the size of the bore, the equipment available, the condition of the formation, and the extent to which timbering is required. The more common methods of attack are:

1. Full face
2. Heading and bench
3. Drift
4. Pilot tunnel

Each of these methods is described in the articles which follow.

**Full-face Attack.** When a tunnel is driven by the full-face attack method, the entire bore or face is drilled, the holes are loaded, and the explosives are discharged. Small tunnels whose dimensions do not exceed about 10 ft are always driven by this method. Large-size tunnels in rock frequently are driven by the full-face method. With the development of the jumbo, or drill carriage, the use of this method has become increasingly more popular in driving large tunnels. A number of drills may be



mounted on the front end of a jumbo and operated simultaneously with a high efficiency.

**Heading and Bench Method.** The heading and bench method of driving a tunnel involves the driving of the top portion of the tunnel ahead of the bottom portion, as illustrated in Fig. 12-1. If the rock is firm enough to permit the roof to stand without supports, the top heading usually is advanced one round ahead of the bottom heading. If the rock is badly broken, the top heading may be driven well ahead of the bench and the bench used in installing the timbers to support the roof. The development of the jumbo has reduced the use of the heading and bench method of driving a tunnel.

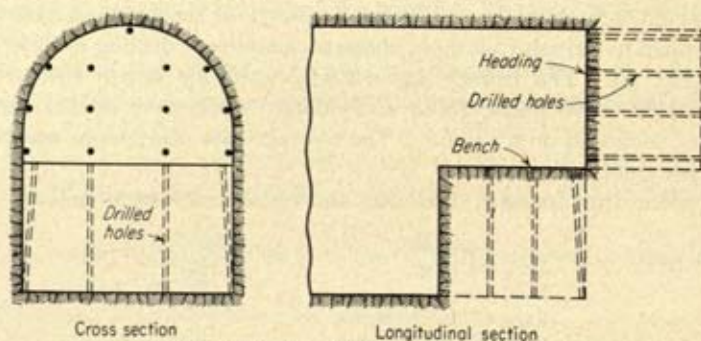


FIG. 12-1. Bench method of driving a tunnel.

**Drift Method.** In driving a large tunnel, it may be advantageous to drive a small tunnel, called a drift, through all or a portion of the length of the tunnel prior to excavating the full bore. A drift may be classified as center, bottom, side, or top, depending on its position relative to the main bore. Figure 12-2 illustrates the position of each type of drift.

The use of the drift method of driving a tunnel has several advantages and disadvantages.

Among the advantages are:

1. Any zone of bad rock or excessive water will be discovered prior to driving the full bore, thus permitting corrective steps to be taken early.
2. The drift will assist in ventilating the tunnel during later operations.
3. The quantity of explosives required may be reduced.
4. Side drifts may facilitate the installation of timbers to support the roof, especially for a tunnel driven through broken rock.

Among the disadvantages are:

1. Driving the main bore must be delayed until the drift is finished.
2. The cost of drilling and handling muck in a small drift will be high because much of the work must be performed by hand instead of by power-operated equipment.

**Drilling Rock.** In driving a tunnel through rock, it is necessary to drill holes for the explosives that loosen the rock. The most commonly used drill is a drifter, equipped with drill steel and detachable bits, either steel or carbide-insert. Drills and bits are of the types described in Chap. 11. Water frequently is used instead of compressed air to remove the cuttings from the holes, as a means of reducing the amount of dust in the air.

For any given project the best depth and spacing of holes over the face of the tunnel should be determined experimentally. The depth of holes

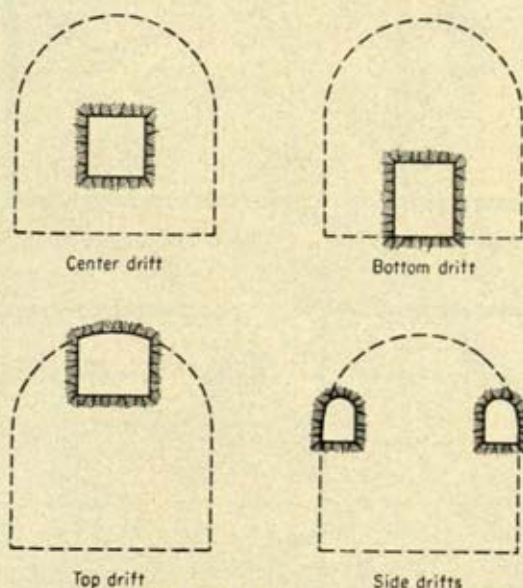


FIG. 12-2. Types of drifts.

will vary with the size and shape of the tunnel, the kind of rock, and the drilling equipment used. The depth advanced during one drilling and shooting operation is called a round. This distance frequently varies from 5 to 20 ft. It will be necessary to drill holes deeper than the advance per round because of loss in depth resulting in blasting. For example, it may be necessary to drill holes 14 ft deep in order to pull 12 ft, the latter value being the effective depth per round.

**Drill Mountings for Small Tunnels.** Drills used in small tunnels and drifts usually are mounted on bars or columns, which are made from sections of steel pipe, equipped with a screw jack at one or both ends. Bars are installed horizontally in a tunnel whose width is less than the height, while columns are installed vertically in a tunnel whose height is less than



the width. Installation consists in placing the bar or column in position and extending the jack until the bar or column is securely wedged in position. Figure 12-3 illustrates the use of bars, while Fig. 12-4 illustrates the use of columns.

The drill may be mounted directly on the bar through an adjustable clamp, which permits movement along the length of the bar. When a column is used, the drill is mounted on an arm, which in turn is mounted

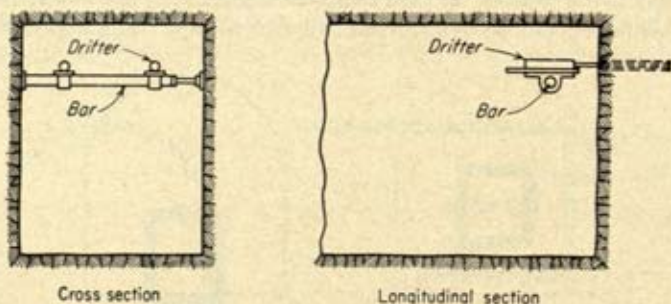


FIG. 12-3. Drifter supported by a bar.

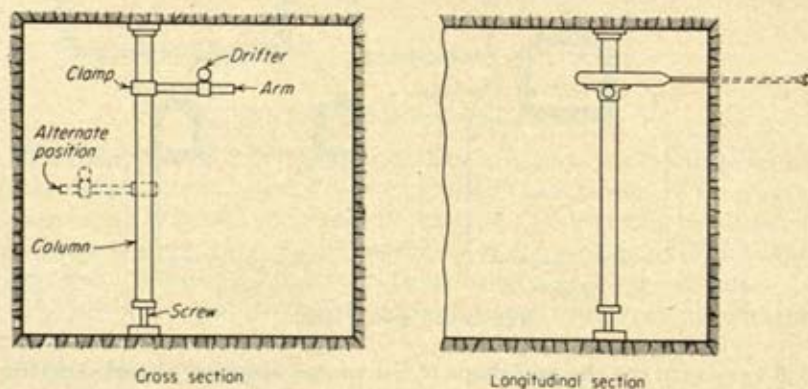


FIG. 12-4. Drill mounted on a column.

on the column through an adjustable clamp. The drill may be moved along the arm or the column.

While bars and columns are satisfactory for use in small tunnels, the excess lengths and weight required for use in large tunnels make them too difficult to handle. In drilling in large tunnels, it is more satisfactory to mount the drills on jumbos.

**Drill Jumbos.** A drill jumbo is a portable carriage with one or more working platforms, equipped with bars, columns, or booms to support the drills. The supports are designed to permit the drills to be spaced to any

desired pattern. The main members of some jumbos have been constructed from welded steel pipe designed to transmit compressed air to the drills.

A recent improvement in drilling equipment is a hydraulic or air-powered boom to support rock drills. This boom, which is mounted on a jumbo, is equipped with controls that permit the operator to spot a drill

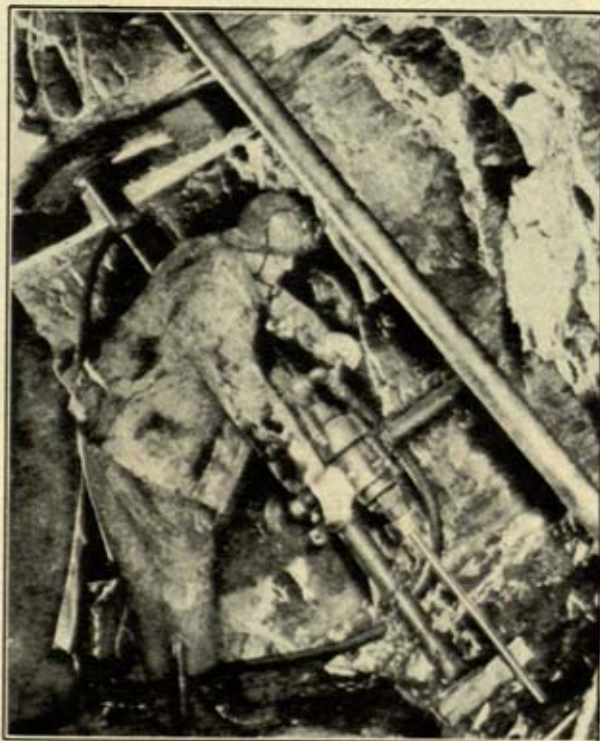


FIG. 12-5. Drill mounted on a column. (*Chicago Pneumatic Tool Co.*)

in any desired position in a few seconds. Figure 12-6 illustrates a powered-boom mounting in operation.

A jumbo may be constructed with one or more working platforms, depending on the size of the tunnel in which it will be used. The platforms may be connected to the jumbo structure with hinges which permit them to be raised or lowered to allow other equipment, such as a mucker or cars, to pass under the jumbo. Several drills may be operated from each platform.

A jumbo may be mounted on skids, on wheels for traveling on rails, or on pneumatic tires. Tire mounting gives a jumbo considerable freedom



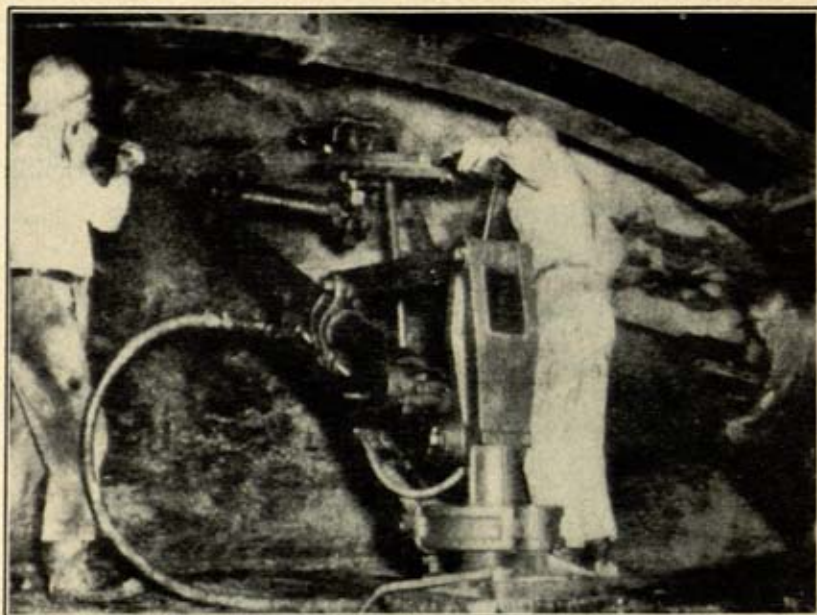


FIG. 12-6. Drill mounted on a powered boom. (*Ingersoll-Rand Co.*)

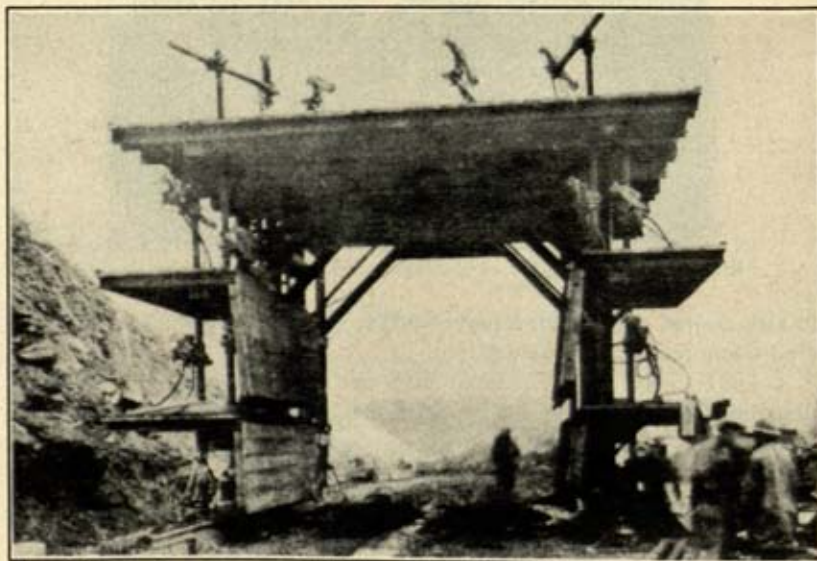


FIG. 12-7. Jumbo mounting 16 drifters, with hinged centers to permit passage of trucks. (*Joy Manufacturing Co.*)

of movement, which facilitates spotting it in position for drilling operations, since it is not restricted to movement on rails.

**Drilling Patterns.** A drilling pattern is the position of the holes drilled into the face of a tunnel in advancing one round. Regardless of the pattern selected for a given tunnel the ultimate purpose will be to break the greatest volume of rock with the least footage of holes and quantity of explosives. The best pattern to produce this result will vary with several factors, such as the size of the tunnel, the depth of holes drilled, the kind

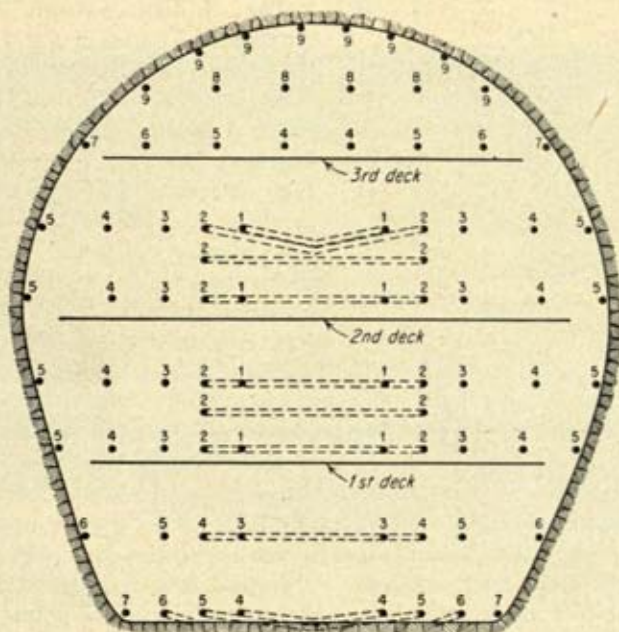


FIG. 12-8. Drilling pattern for the Horetzky-Kemano Tunnel.

of rock, and the method of mounting the drills and should be determined experimentally for each project.

If the explosive in a single hole is discharged, a crater will be formed whose sides make an angle of approximately  $45^\circ$  with the face of the tunnel. When the explosives in holes located around this crater are discharged, the rock breakage per hole will be increased owing to the relieving effect of the crater. In drilling holes for a round of shooting, it is common practice to drill a number of holes which slope toward a common point or a common line near the center of the face to produce an initial cone, or wedge, cut in the rock to the full depth of the round. The explosives in these cut holes are fired first with instantaneous caps; then other holes are fired at progressively later intervals, using delay caps.



A representative drilling pattern for the Horetzky-Kemano heading, a 25-ft horseshoe section, is illustrated in Fig. 12-8 [4]. Charges were fired in nine stages of delay, as indicated by the numbers near the holes. The drilling and shooting pattern was selected to produce a double-V, or wedge, cut. The nominal depth of holes was 13 ft. The positions of the three decks of the jumbo used in drilling the holes are shown.

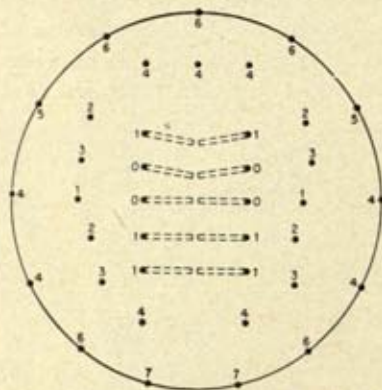


FIG. 12-9. Drilling pattern for the Neversink Tunnel.

The drilling pattern for the Neversink Tunnel of the Delaware Aqueduct is illustrated in Fig. 12-9. This tunnel was drilled to a bore size of 12 ft 6 in., using 5 drifters on a two-deck jumbo. The full-face attack method of drilling was used to produce an average advance of approximately 8 ft per round. As illustrated, the holes drilled near the center of the face were bottomed to form a V, or wedge, cut, while other

holes were drilled perpendicular to the face. The number near each hole indicates the stage of delay in firing the explosive in that hole.

Contracts for driving tunnels usually provide that the contractor shall be paid a given price per cubic yard for all excavation lying within a specified pay line. If a contractor excavates a tunnel beyond this line, he will not be paid for such excess excavation. Also, if the tunnel is lined with concrete, it will be necessary to increase the amount of concrete to replace the excess rock removed. Therefore, it is desirable to keep the overbreak to a minimum. It is good practice to use a template or some other suitable method to locate the pay line and all holes to be drilled. A daub of paint at each hole location will assist in spotting the drills more quickly.

**Loading and Shooting Holes.** In general, the information concerning explosives given in Chap. 11 will apply to the explosives used in driving tunnels. While dynamite is a satisfactory explosive in some respects, the bad fumes which it produces make it less desirable than other explosives for use in tunnel construction, where ventilation may be a major problem.

The explosives may be discharged with electricity, using instantaneous or delay caps, or with Primacord fuse. The cut holes should be fired first to remove the cut, then the relief, and finally the rim holes should be fired to shape the outline of the tunnel.

All equipment should be moved back a safe distance prior to shooting a round of explosives.

**Ventilating Tunnels.** It is necessary to ventilate a tunnel for various reasons, including the following:

1. To furnish fresh air for the workers
2. To remove obnoxious gases and the fumes produced by explosives
3. To remove the dust caused by drilling, blasting, mucking, and other operations

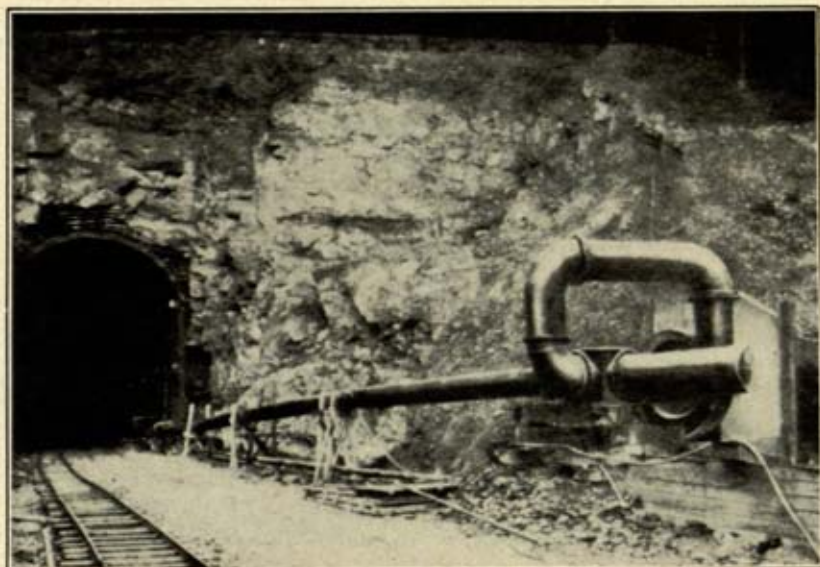


FIG. 12-10. Blower rated ft 12,000 cfm at 2 psi, equipped with reversing valve. (Ingersoll-Rand Co.)

If a drift is driven through a tunnel from portal to portal, it may provide sufficient natural ventilation for the enlarging operations. When natural ventilation is not adequate, as is the case for most tunnels, a positive method of ventilation must be provided.

Mechanical ventilation usually is supplied by one or more electric-motor-driven fans, which may blow fresh air into a tunnel or exhaust the dust and foul air from the tunnel. If air is blown into a tunnel, it may be forced through a lightweight pipe or a fabric duct. If the air is exhausted, it is necessary to use a duct sufficiently rigid to prevent it from collapsing under partial vacuum. Many installations are designed to permit the ventilating system to operate by blowing or exhausting. The reversal of



flow can be accomplished by a valve-and-duct arrangement, as illustrated in Fig. 12-11.

If fresh air is blown into a tunnel, it is released near the working face, and as it flows to the portal through the tunnel, it carries the dust and gases with it. If the exhaust method is used, the foul air and dust are drawn into the duct opening near the working face, thereby causing fresh air to flow into the tunnel from the portal. The latter method has the advantage of more quickly removing objectionable air from the spaces occupied by the workers.

**Volume of Air Required for Ventilation.** The volume of air required to ventilate a tunnel will vary with the number of workers, the frequency of

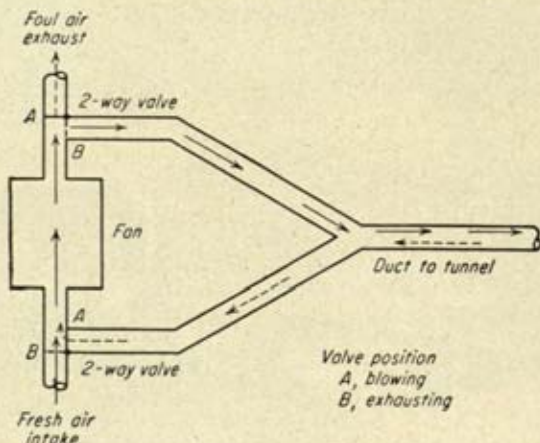


FIG. 12-11. Valve and duct arrangement to reverse the direction of flow of air.

blasting, the method of controlling dust, and the extent to which air-consuming equipment, if any, is used in the tunnel.

Each worker should be supplied with 200 to 500 cfm of fresh air. Compressed air, furnished to the drills, should not be included in computing the volume of air required for a project, as this air is contaminated with oil and dust before it is released by the drilling operations.

The firing of explosives to loosen rock fills the space near the face of the tunnel with gases and dust, which makes the air unfit for breathing. This foul air must be removed and replaced with fresh air before the workers can start mucking out the broken rock. The cycle of operations may be organized so that the workers retire a safe distance from the face prior to firing the explosives and eat lunch during the time required to remove the gases. If a 30-min lunch period is scheduled, the capacity of the ventilating equipment should be sufficient to clear the tunnel in that period of time.

In driving the 22- by 31-ft railroad tunnel near Aspen, Wyo., the contractor supplied 18,000 cfm of air through a 26-in. vent pipe. The first 1,000 ft of each of the 36- by 26½-ft twin bores of the Squirrel Hill Tunnel on the Penn-Lincoln Parkway was ventilated with a vane-axial fan capable of blowing 43,000 cfm of air through a 36-in. vent pipe, as illustrated in Fig. 12-12 [5]. After the tunnels were driven about 1,000 ft, the ventilation system was revised to permit air to flow from one tunnel into the other through passageways excavated between the two tunnels

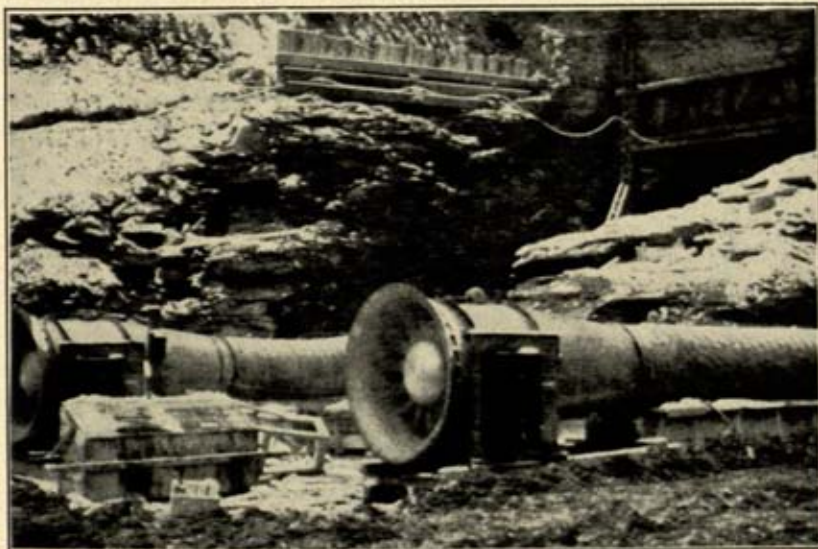


FIG. 12-12. Twin radial fans used to ventilate the Squirrel Hill Tunnel. (*Joy Manufacturing Co.*)

at 500-ft spacings. An air lock at the portal of one tunnel made it possible to control the direction of flow of air into or through either tunnel. The capacity of the ventilation system was increased to 240,000 cfm for both tunnels by installing additional fans.

**Size and Capacity of Vent Pipe.** After the quantity of air required to ventilate a tunnel is determined, the next step is to determine the size pipe and blower or blowers that will give the lowest total cost [6]. The total cost will include the installed costs of the blowers and pipe, with an allowance for salvage value upon completion of the project, plus the cost of electrical energy required to operate the blowers. If a fixed amount of air is to be supplied at the face of a tunnel, the use of a small pipe will require the installation of a larger blower, which will result in a high installation and operating cost. If a large pipe is used, the cost of



the blower and the operating cost will be less but the cost of the pipe will be higher. For every project there is a combination of sizes which will give the lowest total cost. This is the most economical installation.

Consider a tunnel, with a 250- to 300-sq-ft bore, whose maximum length will be 16,000 ft. It is determined that 3,000 cfm of free air will be required for the tunnel. The effect which the size of pipe used has on the pressure lost in the pipe is shown in Fig. 12-13. As the tunnel progresses, the length of the pipe must be increased and the capacity of the blowers also must be increased to overcome the greater back pressure resulting from longer pipe. The cost of energy required to operate the

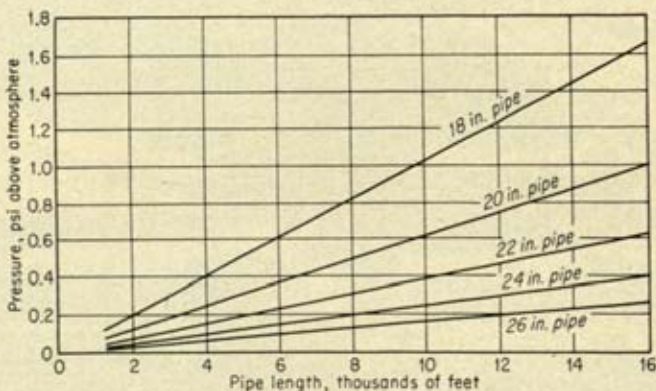


FIG. 12-13. The effect of the size of vent pipe on the loss in pressure.

blowers will increase as the length of the tunnel is increased. However, a reasonably accurate estimate of the total cost of energy can be obtained by computing the cost for each of several convenient equal lengths, such as at the quarter points.

Table 12-1 illustrates a method of determining the most economical size pipe for the tunnel under consideration. For this project the 22-in. pipe is the most economical, considering all costs.

**Dust Control.** The operations such as drilling, blasting, loading, and hauling muck cause dust to accumulate in the air in a tunnel. Unless precautions are taken to limit the concentration, the dust will constitute a serious health hazard to the workers. This is especially true when a tunnel is driven through rock containing a high per cent of silica, as extended exposure to silica dust may cause silicosis, a lung disease, for which there is no positive cure. Most states have laws governing mining and tunneling practices which are designed to protect workers against this disease by limiting the concentration of silica dust particles in the air.

TABLE 12-1. THE COMBINED COST OF COMPRESSORS, PIPE, AND ENERGY FOR VENTILATING A TUNNEL

Size pipe, in.	Cost			
	Compressor	Pipe	Energy	Total
18	\$25,280	\$33,740	\$15,320	\$74,340
20	16,460	38,450	9,790	64,700
22	9,670	42,580	5,520	57,770
24	6,130	49,750	3,480	59,360
26	5,290	58,180	2,360	65,830

Various methods are used to limit the amount of dust in the air in a tunnel, including the following:

1. The use of water instead of air to remove the cuttings from drilled holes
2. The use of a vacuum hood that fits around the drill steel at the rock face to remove the dust that comes from a hole during the drilling operation
3. Complete ventilation of the space near the face, preferably by the exhaust method, following each round of blasting
4. Keeping the muck pile wet during loading operations

A system of dust control that was used successfully on the Delaware Aqueduct was to install on the drill jumbo several suction pipes with openings near the face of the heading. These pipes drew the dust-laden air from the face and passed it through filters, located at the rear of the jumbo, which removed most of the dust.

**Mucking.** The operation of loading broken rock or earth for removal from a tunnel is referred to as mucking. This operation may be performed by hand, power shovels, mucking machines, slushers, or tractor loaders.

Hand mucking is limited to small tunnels and drifts which are not large enough to justify or permit the use of mechanical muckers.

Special power shovels, with short booms and dipper sticks, have been used for mucking in large tunnels. If ventilation is not a serious problem, a gasoline- or diesel-engine-powered unit may be used. If the exhaust fumes are objectionable, a unit powered with an electric motor should be used. Figure 12-14 illustrates a short-boom electric-motor-powered shovel loading a truck.

Several types of mechanical muckers are available for use in tunnels. Figure 12-15 illustrates a popular mucking machine, while Fig. 12-16 illustrates the same machine loading muck in a tunnel. This machine, which operates on rails, moves forward to push the dipper into the pile of muck.





FIG. 12-14. Short-boom electric-motor-powered shovel loading muck. (*Joy Manufacturing Co.*)

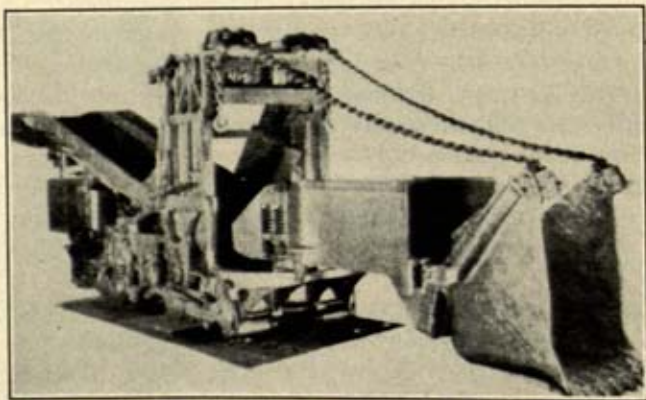


FIG. 12-15. A mucking machine. (*Goodman Manufacturing Co.*)

When the dipper is filled, the machine backs up a short distance and tips the dipper up to discharge the load onto a belt, which conveys it back to a muck car, attached temporarily to the mucking machine. This machine is manufactured in several sizes, with a dipper capacity up to 1 cu yd, and a loading capacity up to 162 cfm loose volume. The side swing of the dippers of the larger models gives them a cleanup range in

excess of 10 ft on each side of the center line of the track. This machine is operated by an electric motor.

The mucking machine illustrated in Fig. 12-17 is designed to discharge onto a conveyor belt, attached to the rear of the machine, or it will discharge directly into a muck car, attached at the rear of the machine.

Several types of tractor-mounted loaders are available for use in tunnels. The bucket of the loader, illustrated in Fig. 12-18, is lowered to a position in front of the tractor, filled by the forward movement of the tractor, then lifted over the tractor to discharge the load into a truck.

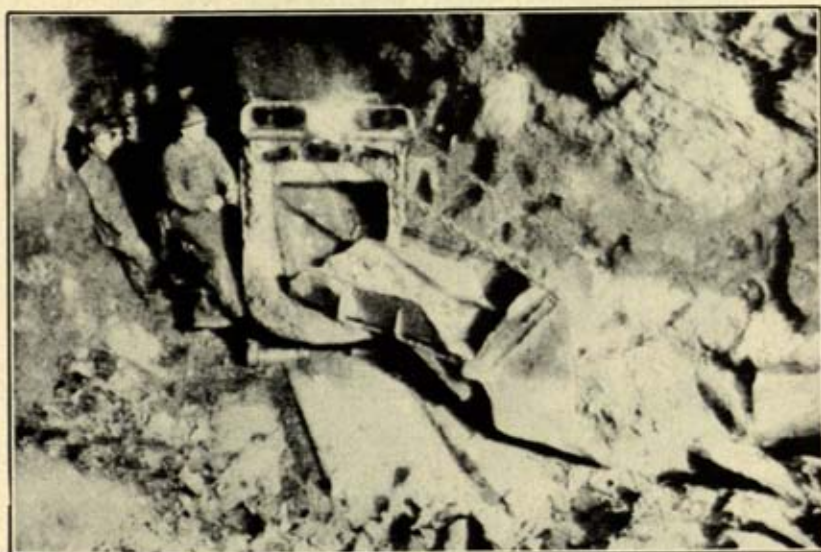


FIG. 12-16. Mucking machine loading muck in a tunnel. (*Goodman Manufacturing Co.*)

Tractors or other loading equipment powered with internal-combustion engines should not be used in a tunnel unless the ventilation system, natural or mechanical, is adequate to remove the exhaust fumes without injury to the workers. As carbon monoxide gas is the most dangerous fume, special care must be exercised if gasoline-engine-powered equipment is used. Because diesel engines do not produce carbon monoxide, they are safer than gasoline engines for use in tunnels.

**Hauling Muck.** Muck is hauled from a tunnel in narrow-gauge muck cars, pulled by locomotives, or in trucks.

In the early days of tunneling the muck was hauled in small cars pulled along the rails by men or mules. These small cars have been replaced with larger cars which are pulled by electric locomotives. Also, trucks are used in increasing numbers to haul muck from tunnels.



**Tracks.** When muck is hauled in cars, steel rails are required. For this use relatively lightweight rails are laid to a narrow gauge, most frequently 24 or 36 in. For a long tunnel it is necessary to provide a double track in order that loaded cars may be moved out while empty cars are moved into the tunnel. As the width of a muck car is usually twice the gauge of the rails, the maximum gauge is limited to slightly less than one-fourth the width of the tunnel. The weight of the rail, expressed in



FIG. 12-17. Rocker shovel loading muck in a tunnel. (*The Eimco Corp.*)

pounds per yard, should be heavy enough to prevent objectionable sag between the supporting ties when the locomotive and loaded cars travel over them. Also, if the same rails will be used for all haulage, including timbering, reinforcing, and concrete for the lining, considerable expense in laying the rails on a good foundation will be justified. Low rolling resistance, plus freedom from excessive maintenance cost and reduced output due to car derailments, depends largely on the use of a good track.

**Muck Cars.** Various types and sizes of cars are used to haul muck from tunnels. The capacity may be expressed in cubic feet or cubic

yards. In general, the largest size that can be used in a tunnel will be the most economical, as large cars reduce the time lost in switching at the loading operation.

The cars commonly used are constructed with sides hinged at the top and fastened with latches at the bottom to permit easy dumping.

**Locomotives.** Three types of electric locomotives are available for tunnel hauling, the trolley, battery, and combination trolley and battery. All three are available in various weights and for operation on different track gauges, as indicated by the manufacturers.

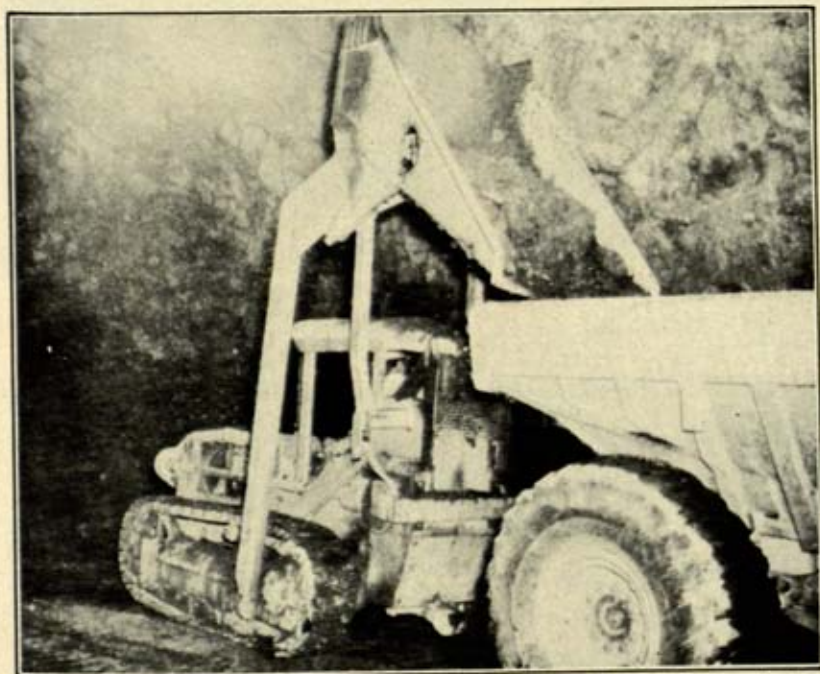


FIG. 12-18. Overshot loader loading muck into a truck. (*Western Construction.*)

The trolley-type locomotive is relatively easy to operate, but it requires a bare trolley wire, which may interfere with other operations in a tunnel and which represents a source of potential danger to workers. Also, it is necessary to ground the rails which serve as a return circuit for the electricity.

The battery-type locomotive is operated from a group of storage batteries mounted directly on the locomotive. These batteries should operate a locomotive for 8 hr, after which they must be recharged, which requires about 8 hr. If a locomotive is to operate more than one 8-hr



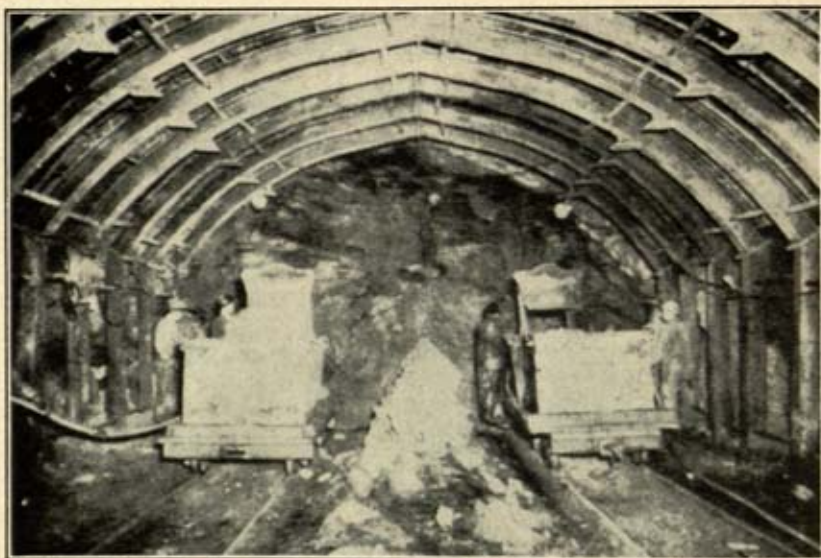


FIG. 12-19. Muck cars being loaded. (*The Eimco Corp.*)

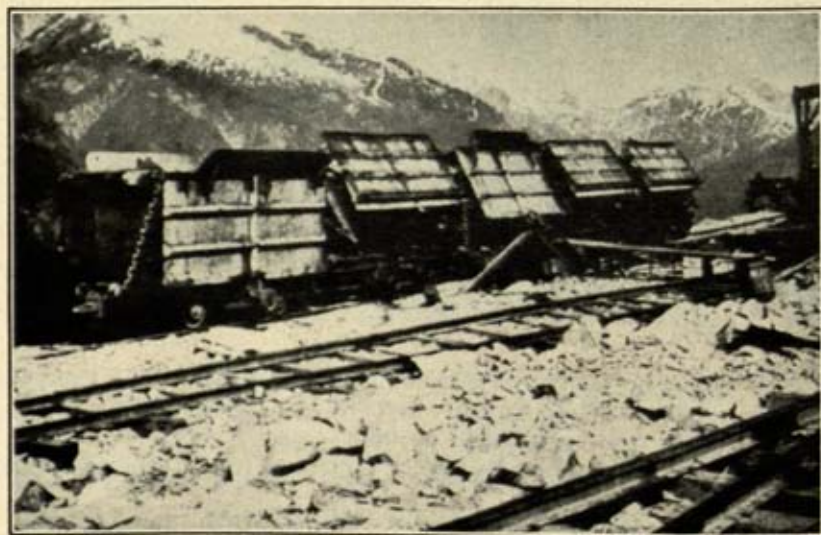


FIG. 12-20. Automatic side-dump muck cars. (*Construction Methods and Equipment.*)

shift per day, it is necessary to provide at least two sets of batteries so that one set may be charged while the other set is in use.

The combination trolley-and-battery-type locomotive is satisfactory for use on a project which requires haulage inside and outside of a tunnel. The batteries are used in the tunnel and the trolley outside the tunnel. If the operation from the trolley is long enough, the batteries may completely recharge each round trip.

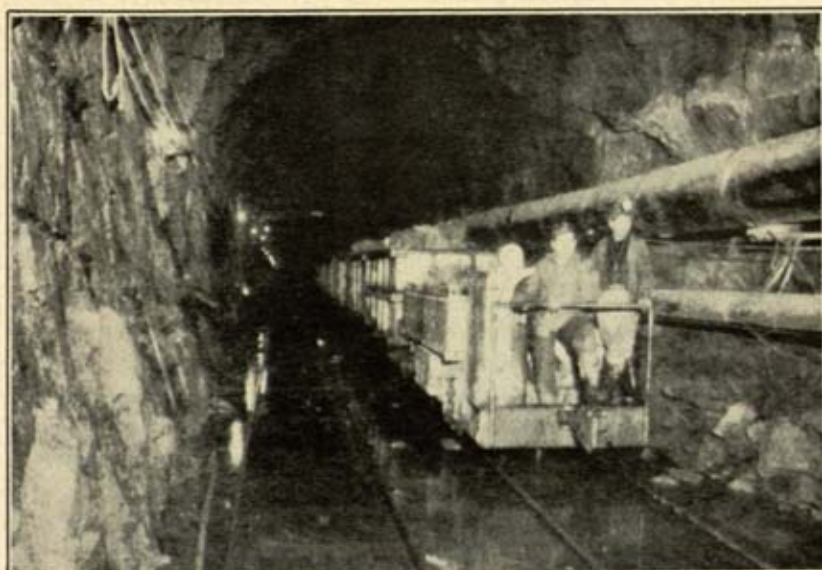


FIG. 12-21. Battery-type locomotive pulling cars of muck out of a tunnel. (Goodman Manufacturing Co.)

The size of a locomotive is indicated by its weight, expressed in tons. If it has sufficient power to slip the driving wheels when standing on dry steel rails, the maximum tractive effort will be equal to the product of the weight times the coefficient of friction between the wheels and the rails. The coefficient of friction will usually be 0.2 to 0.25. Thus, an 8-ton locomotive should provide a tractive effort of at least

$$16,000 \times 0.2 = 3,200 \text{ lb}$$

A method of determining the maximum number of cars that can be hauled is illustrated in the following examples.

**EXAMPLE.** Determine the maximum number of cars that can be hauled on a level track by an 8-ton locomotive. The rolling resistance will be 30 lb per ton and the starting resistance 20 lb per ton of gross load, including the weight of the locomotive.



The cars, which have a capacity of 80 cu ft each, weigh 2,800 lb empty and 10,800 lb loaded.

Available tractive effort,	$16,000 \times 0.2 = 3,200$ lb
Total resistance,	$30 \text{ lb} + 20 \text{ lb} = 50$ lb per ton
Max gross load,	$3,200 \text{ lb} \div 50 \text{ lb per ton} = 64$ tons
Deduct weight of locomotive	$= 8$ tons
Max net load	$= 56$ tons
No. of cars,	$56 \times 2,000 \div 10,800 = 10.4$ , or 10
Volume of muck per trip,	$10 \times 80 \div 27 = 29.6$ cu yd

**EXAMPLE.** Determine the maximum number of loaded cars that can be hauled up a 1 per cent grade with all other conditions the same as for the preceding example.

Grade resistance,	20 lb per ton
Total resistance,	70 lb per ton
Max gross load,	$3,200 \text{ lb} \div 70 \text{ lb per ton} = 45.7$ tons
Deduct weight of locomotive	$= 8$ tons
Max net load	$= 37.7$ tons
No. of cars,	$37.7 \times 2,000 \div 10,800 = 7$
Volume of muck per trip,	$7 \times 80 \div 27 = 20.7$ cu yd

**Ground Support.** When a tunnel is driven, it may be necessary to support the ground adjacent to the tunnel until a permanent concrete lining can be installed. The temporary supports must be strong enough to resist the pressures transmitted to them by the ground. These pressures are caused by faulted, folded, or fractured rock masses or the swelling of the surrounding earth following the removal of the material from a tunnel.

The operation of placing supports in a tunnel to resist the movement of ground is referred to as timbering. The type and extent of timbering is determined to a large degree by the kind and physical condition of ground to be supported. In the early years of tunnel driving heavy wooden timbers were commonly used for supports, but in recent years wood has been replaced by steel H beams. These beams can be fabricated from any desired section to produce ribs that fit the shape of any given tunnel bore. A rib may consist of two or more sections, which are carried into the tunnel, assembled, and bolted together as the driving progresses. Figure 12-22 illustrates the use of wood timbering, while Fig. 12-23 illustrates the use of steel-H-beam timbering.

The advantages of steel sections instead of wood include the following:

1. Because smaller sizes are used it is possible to reduce the size of the bore of a tunnel. This reduces the cost of excavation and permits faster driving.
2. They can be installed more quickly and economically than wood.
3. They supplement the steel reinforcing in the concrete lining.
4. Their smaller dimensions may permit the use of thinner concrete linings.

The safe spacing of ribs may vary from 18 in. to as much as 6 to 8 ft, depending on the physical condition of the ground. Most grounds tend to bridge over at some height above the roof of a tunnel. As the rock above the natural bridge will be self-supporting, only that rock between the roof of a tunnel and the bridge surfaces must be supported by the timbering. Figure 12-24 illustrates how the physical condition of the rock affects the height of the natural bridge formed over the roof of a

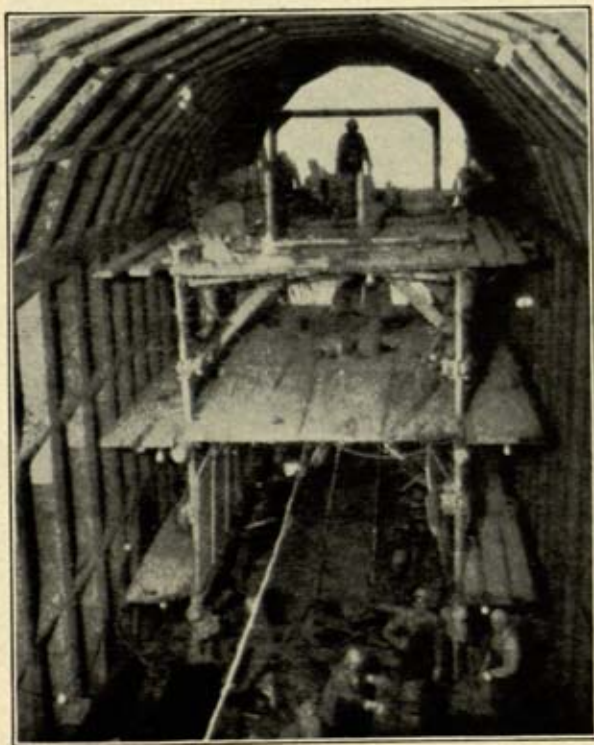


FIG. 12-22. Wood timbering used to support the walls and roof of a tunnel. (*Union Pacific Railroad.*)

tunnel. As shown in the figure, rock having greater solidity or larger pieces will bridge more quickly than rock which is badly broken. The timbering must support the weight of the rock lying below the bridge surfaces *abc*. If the physical condition of the rock is known in advance of driving a tunnel, the spacing of the ribs may be determined with reasonable accuracy. However, because of the changes in the condition of rock encountered as a tunnel is driven, it is found in actual practice that the spacing of ribs should be modified to meet the conditions that exist at any given section in a tunnel.



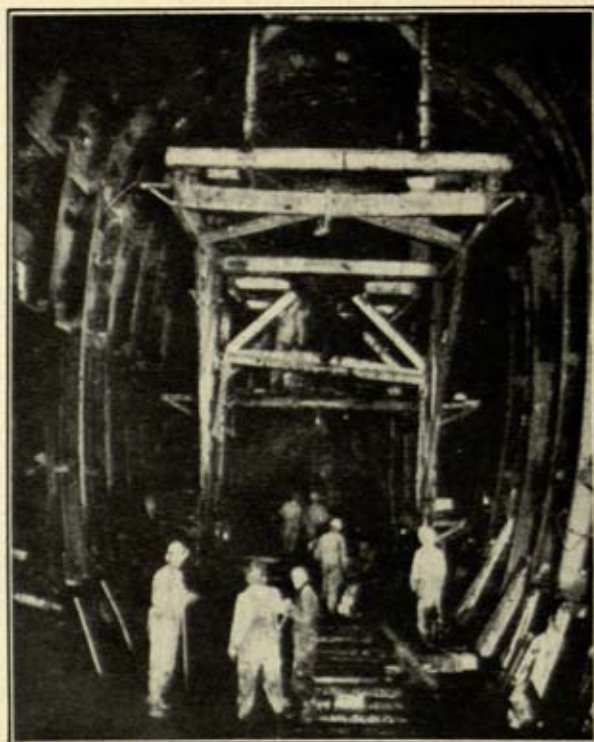
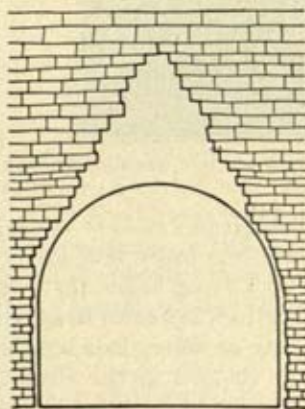
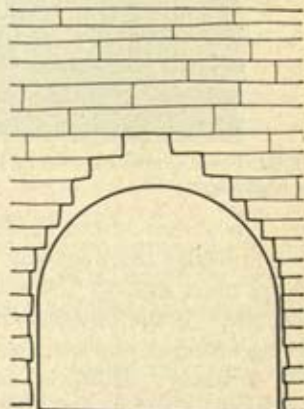


FIG. 12-23. Steel H-beam supports for a tunnel. (*Union Pacific Railroad.*)



(a) Badly broken rock



(b) More solid rock

FIG. 12-24. The effect of the physical condition of rock on the bridging action.

Prior to awarding a contract for driving eight power and outlet tunnels at Garrison Dam, the United States Army Engineers drove a full-size test section 240 ft long in order to determine what forces the timbering and lining must withstand [7]. These circular tunnels, whose excavated diameters varied from 27 to 36½ ft, were driven through a formation of clay and shale. Figure 12-25 shows a section of a steel rib being hoisted to position for installation by a top-mounted jumbo, which rode on rails at the spring line, as shown in Fig. 12-26.

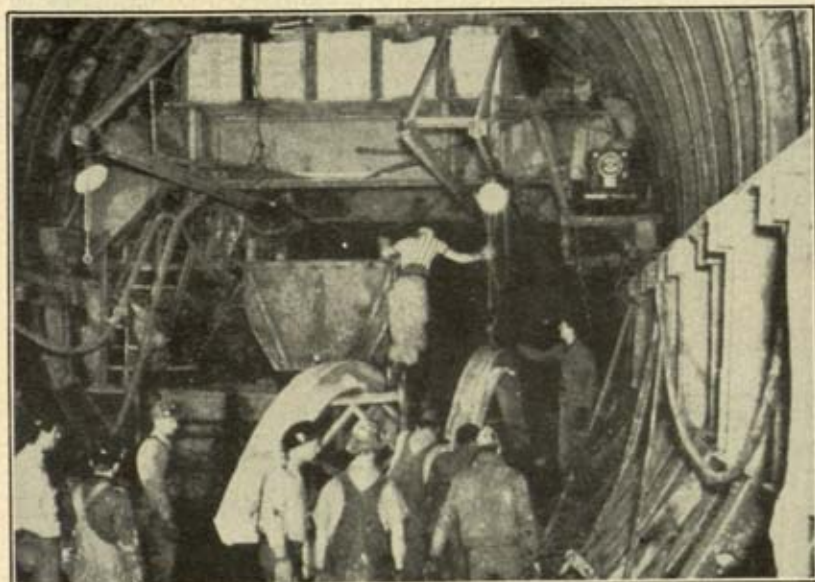


FIG. 12-25. Hoisting section of steel rib into position for tunnel supports. (*Corps of Engineers, U.S. Army.*)

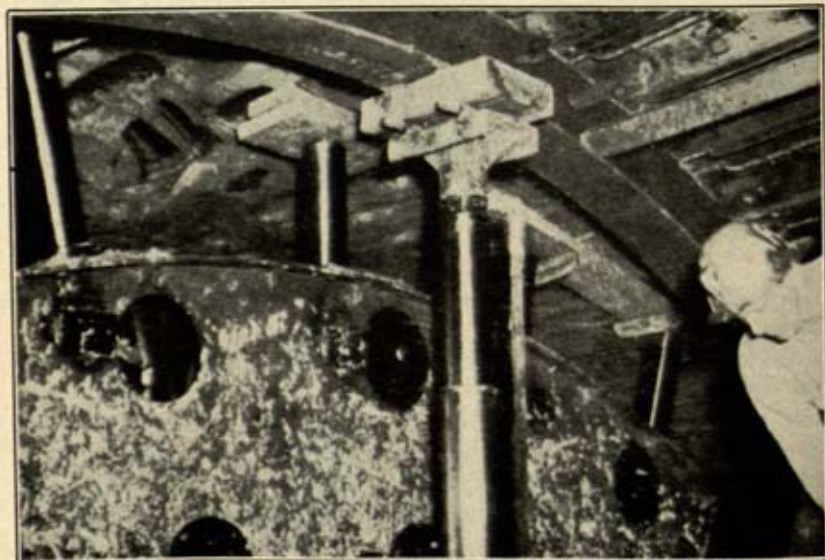
In order that the sets of ribs may resist the forces from the ground and prevent broken pieces of rock from falling into the bore of a tunnel, it is necessary to install some type of cover, called lagging, outside of the ribs. This lagging may consist of heavy pieces of lumber, extending from rib to rib, or it may consist of a series of steel plates. The space between the lagging and the undisturbed ground or rock should be filled with timbers, large pieces of rock, or gravel packing to prevent the ground from shifting toward the timbering. The use of lagging and rock packing is illustrated in Fig. 12-23.

**Controlling Water.** In driving a tunnel, the control of water will consist of one or two operations, namely, preventing excess quantities of water from entering the tunnel and removing the water that does enter.





(a)



(b)

FIG. 12-26. Forcing rib sections into place with jacks mounted on a jumbo. (*Corps of Engineers, U.S. Army.*)

Most of the water in a tunnel comes from two sources, that used to wash the cuttings from the drill holes and that which flows in from the ground through which the tunnel is driven. The former may be estimated with reasonable accuracy, but the latter is subject to great variation. For example, shooting a charge of explosives may open fissures into a ground-water reservoir, thus permitting an unexpectedly large quantity of water to flow into a tunnel. It is good practice to drill exploratory holes ahead of and deeper than those drilled for explosives in order to determine whether there is badly broken rock or ground water



FIG. 12-27. Air-driven sump pump removing water from a tunnel. (*Ingersoll-Rand Co.*)

ahead. If the exploratory holes indicate that such a condition exists, it is possible to grout off heavy flows of water and solidify the formation before the tunnel reaches the trouble zone.

Many types and sizes of pumps are available for removing water from a tunnel, as described in Chap. 15. The two types most commonly used are the air-driven centrifugal and the electric-motor-driven centrifugal. Both are compact units, with high capacities, which will operate satisfactorily under varying heads.

The water that accumulates near the face of a tunnel is collected in a sump. This water is picked up with air-driven centrifugal pumps and pumped back to another sump located nearer the portal, where most of



the solid materials may settle out, after which it is pumped out of the tunnel with semipermanently installed electric-motor-driven centrifugal pumps.

An air-driven sump pump is satisfactory for use under the adverse conditions that usually exist near the face of a tunnel, but an electric-motor-driven pump usually is preferred for use in the main pumping operation. A switch control operated by a float set in the sump will make a pumping unit practically automatic.

Because of the possibility of encountering excessive flows of water into a tunnel it is good practice to provide stand-by pumps, which may be placed in operation very quickly.

See Chap. 15 for more complete information on various types of pumps.

**The Cross Sections of Tunnels.** The shape of the cross section of a concrete-lined tunnel will depend on the pressure of the ground which the lining must resist and the purpose for which the tunnel is constructed. If the ground is solid rock, any desired shape may be selected. For an aqueduct the section may be circular, while for a vehicular tunnel the section may consist of a flat invert, vertical walls, and an arched roof. If the ground is broken rock, subject to horizontal pressure, the vertical wall section of a vehicular tunnel should be replaced with horseshoe curves to resist such pressure. If the ground is highly unstable, such as soft clay or sand, it may be necessary to use a circular section, because of its greater resistance to external pressures, regardless of the purpose for which the tunnel will be used.

The most common cross sections are illustrated in Fig. 12-28. They include the circular, elliptical, horseshoe, and vertical wall with arch-roof types. The circular and elliptical sections are popular for water and sewage conduits, while the horseshoe and vertical sections are popular for vehicular tunnels where the ground conditions permit such sections to be used.

**Thickness of Concrete Linings.** In the interest of economy it is desirable for a concrete tunnel wall to be as thin as practical. The thickness may be determined by the condition of the ground surrounding the tunnel, the size and shape of the cross section, the requirements of construction conditions, or the internal pressure in the event it is a water conduit.

The geological survey made prior to designing a tunnel lining should indicate whether the ground is solid or broken rock or unconsolidated soil, subject to horizontal and vertical pressures. If steel ribs and reinforcing are used in a concrete wall, the thickness may be reduced but the use of steel for the sole purpose of reducing the thickness of the lining is not economical. A rule which has been used to some extent as a guide only is to allow 1 in. of wall thickness for each foot of diameter.

As solid rock does not impose any load on a concrete lining, the thick-

ness may be the minimum that can be placed behind the forms and cover any utility pipes, ducts, or appurtenances.

The designed thickness of a concrete lining which must resist the pressure from bad ground may include the concrete surrounding steel ribs and sections but should not include any concrete that is encroached on by wood timbers or any kind of lagging.

The 8-ft-diameter Carter Lake pressure tunnel of the Colorado-Big Thompson project, which was designed for a maximum dynamic head of

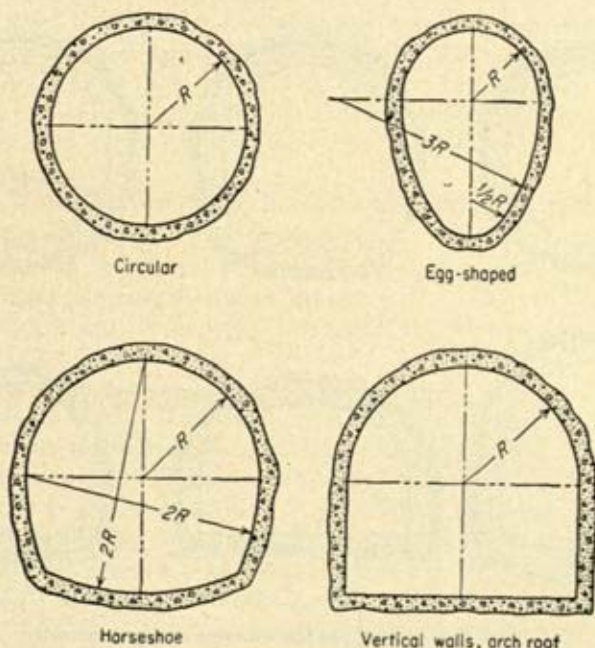


FIG. 12-28. Typical cross sections of tunnels.

325 ft of water, has a minimum concrete thickness of  $5\frac{1}{2}$  in. but is heavily reinforced with steel bars. The eight circular power and outlet tunnels at Garrison Dam, ranging from 27 to  $36\frac{1}{2}$  ft in excavated diameter, have wall thicknesses varying from 30 to 42 in. [7]. The Gaviota Gorge Tunnel, a highway tunnel in California, whose inside dimensions are 35 ft 3 in. wide and 22 ft high, has three wall thicknesses, 18 in. in the cut and cover section, 24 in. in the section supported with steel ribs, and 36 in. in the section supported with timbers [8]. All these are minimum thicknesses, which are subject to increases due to overbreak in the sizes of the bores during blasting operations.



**Sequence of Lining a Tunnel.** If a tunnel is driven through solid rock, or if the supports will prevent objectionable movements of the rock until the entire bore is holed through, it is desirable to delay starting the lining until the excavation is finished. If this plan of construction is followed, the mucking and lining operations will not interfere with each other and a greater operating efficiency should be possible. However, if the ground is so unstable that it is difficult or impossible to restrain its movements, it will be necessary to install the lining as quickly as possible after blasting and mucking each round.

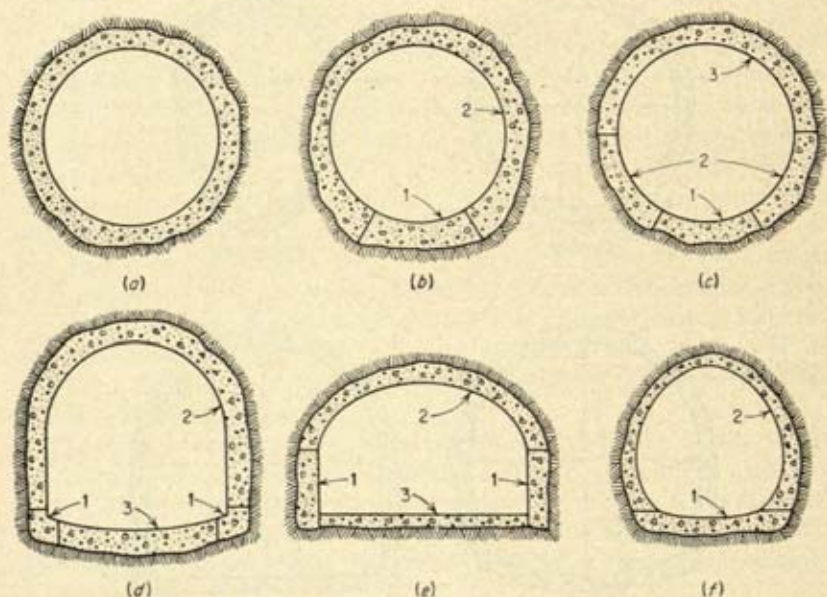


FIG. 12-29. Sequence of placing concrete lining in tunnels.

The sequence of installing the lining around the perimeter of a tunnel will depend on a number of factors. As illustrated in Fig. 12-29, any one of several sequences may be selected. The numbers 1, 2, and 3 indicate the sequence of placing the sections of the lining.

Plan *a* shows the entire wall placed in one operation. This method is limited to circular tunnels which are poured in relatively short sections. The top of the form must be rigidly blocked to prevent it from floating to the roof of the bore.

Plan *b* provides a rigid base on which the forms for the side walls and roof may be supported. If this plan is adopted, construction on the invert should be started at the most distant point and proceed toward the

portal in order to eliminate the need of hauling materials over the concrete before it has set sufficiently.

Plan *c* is limited to large tunnels where it is desirable to separate the pours into the indicated sections.

Plan *d* offers several desirable advantages. The two curbs can be installed at the sides of a tunnel with little or no interference to the haulage tracks, located near the center of the tunnel. These tracks need not be removed or disturbed. After the curbs have cured sufficiently, they may be used to support wide-gauge rails on which the main form jumbos travel. Also, they may be used as guides, supports, and anchors for the bottoms of the wall forms. After the curbs, walls, and roof are completed, the original rails, which were used for hauling muck, supports, concrete, etc., can be removed just ahead of placing the invert. Thus, these rails need not be replaced, as would be necessary if the invert were placed first.

Plan *e* is used in placing the lining for large tunnels. Either the side walls or the invert may be placed first.

In plan *f* the invert is placed entirely across the tunnel floor; then the walls and roof are placed later in one operation. The chief objection to this method is that the haulage tracks must be removed prior to placing the invert, then relaid on top of the invert, if they are to be used to haul concrete or other materials for the walls and roof.

**Reinforcing Steel.** If reinforcing steel is required in a concrete lining, it may consist of steel ribs, bars, or both. The design for a thick concrete lining may specify two layers of reinforcing bars, one near the inner and the other near the outer surface of the lining. The space between the two layers, especially near the top of the roof, must be great enough to permit the insertion of a pipe through which the concrete will be placed. Figure 12-30 shows the reinforcing steel in place for the Queen Creek Tunnel in Arizona. A jumbo was constructed for use by the ironworkers in placing the reinforcing.

**Forms for Concrete Linings.** Forms used for lining tunnels are, with few exceptions, of the traveling type, constructed of steel or a combination of steel and wood. While the initial cost of steel forms will exceed the cost of wood forms, the additional uses obtained from steel compared with wood, together with the savings in time and labor required to move and set up in using steel forms, usually will make steel forms cheaper than wood for any tunnels other than short ones.

The traveling-type form is constructed of steel members, lined with steel plate or wood to give a surface which conforms with the shape of the inside surface of the portion of the tunnel for which it will be used. Thus, a form may be used for constructing the invert, the side walls, the roof, or any desired combination thereof. Each form is mounted on a traveler



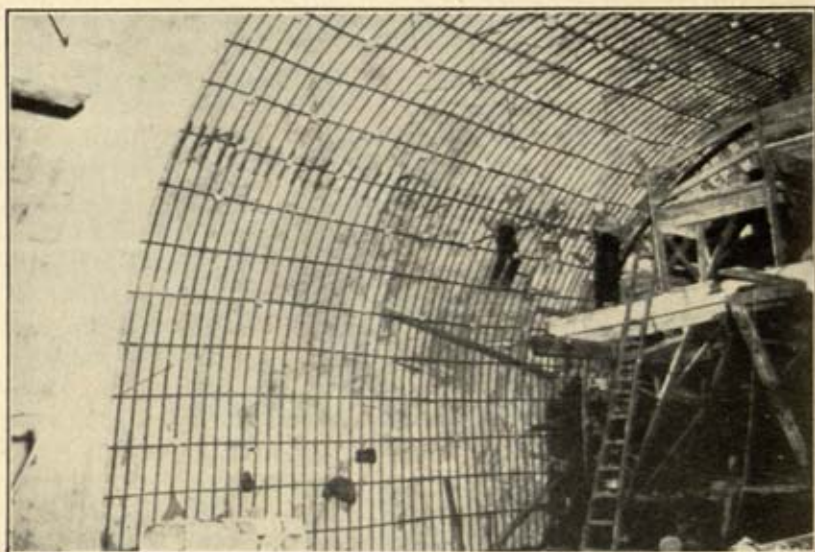


FIG. 12-30. Placing reinforcing steel for the lining of a tunnel. (*Western Construction.*)

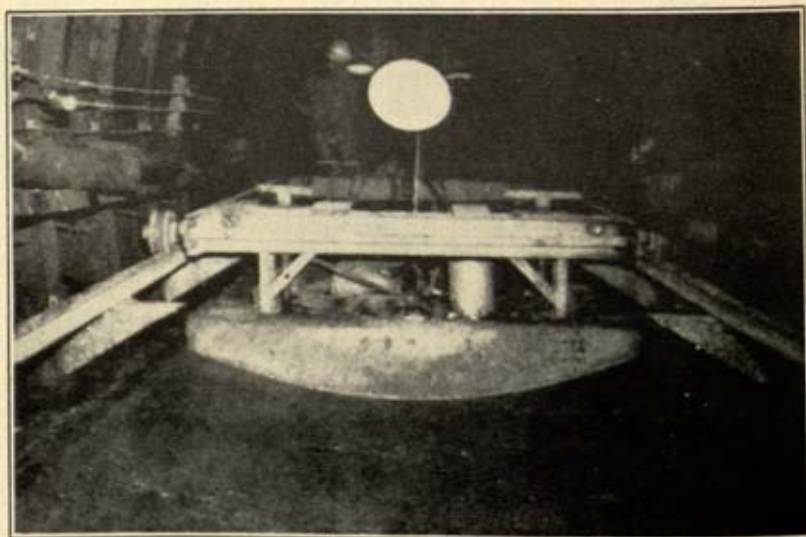


FIG. 12-31. Steel screed used to line the invert of a tunnel. (*Chicago Bridge & Iron Co.*)

or a jumbo, which in turn is mounted on wheels that permit it to be moved along rails. A traveler is equipped with adjustable jacks or screw ratchets, which permit the form to be expanded into position for a concrete pour, then collapsed slightly to pull it away from the concrete in order that it may be moved into a new location.

Figure 12-31 shows a steel-screed form, mounted on wheels that roll on wide-gauge rails, used to line the invert of a tunnel.

Figure 12-32 shows a steel form 24 ft 6 in. in diameter by 32 ft long used to line the walls and roof of a circular tunnel. The form, which is hinged

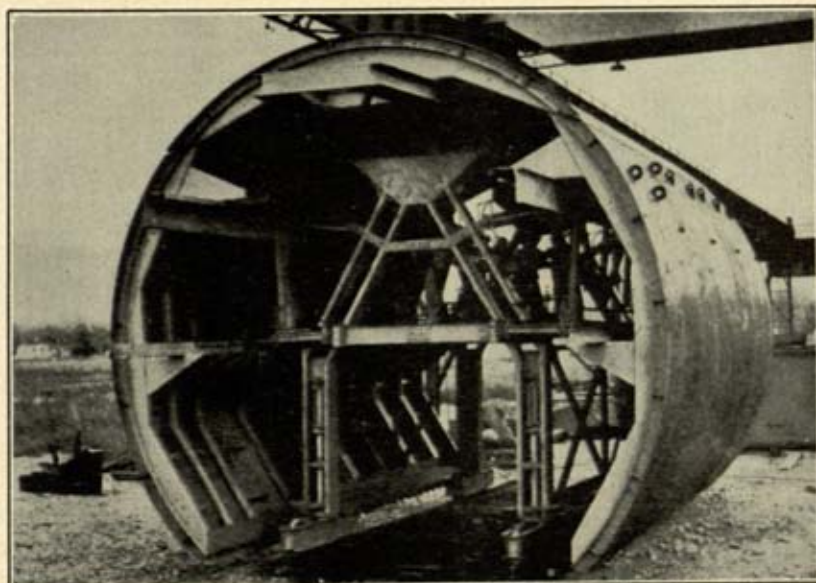


FIG. 12-32. Steel forms used to line tunnels. (Chicago Bridge & Iron Co.)

near the upper quarter points, is supported by a prefabricated steel jumbo mounted on eight wheels. Through a series of jacks, attached between the form and the jumbo, it is possible to expand the form to full size or to retract the side walls and lower the roof to permit movement to a new location. Forms of this type may be fabricated with hinged doors along the side walls or roof to permit inspection behind the form or the placing of concrete through the doors.

In lining a highway tunnel 42 ft wide by 22 ft 10 in. high in Arizona, the contractor used two sets of steel forms, each 30 ft long. The first form was set in position, and the concrete lining for the side walls and roof was installed. The form was left in place for 72 hr after the pour, then moved ahead far enough to leave a 30-ft-long gap of unlined tunnel.



The second form was used later in lining this gap. This procedure was used throughout the tunnel. Figure 12-33 shows the first set of forms in place, supported by a jumbo. Note the pipe line used in pumping concrete for the lining.

Figure 12-34 shows the three 50-ft-long sections of steel forms used to line the twin-tube Squirrel Hill Tunnel. Each form was constructed with a continuous hinge along each side just above the spring line. A set of forms could be collapsed sufficiently to permit it to pass through a form set in position for a concrete pour. After the first section was anchored

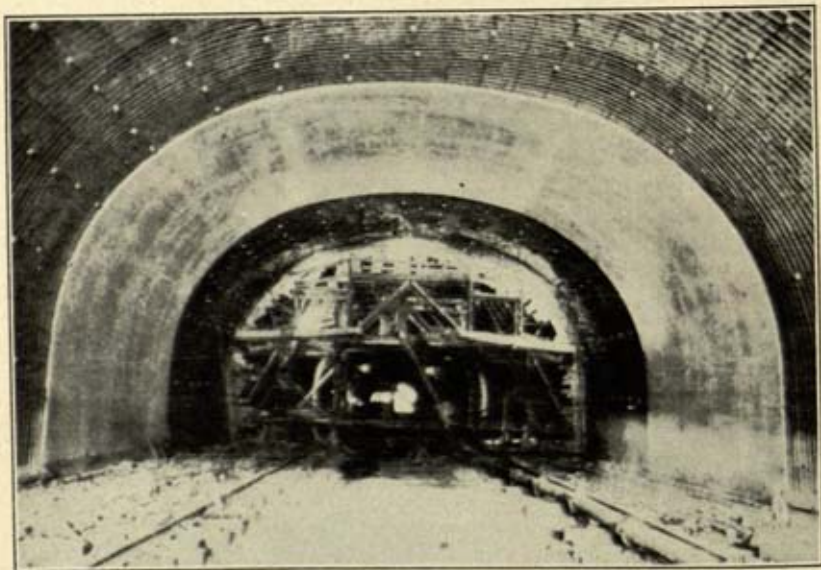


FIG. 12-33. Two sets of steel forms used to place concrete lining in a tunnel. (Arizona Highway Department.)

in position and the concrete was poured, the second section was moved ahead and anchored and the concrete was poured. This operation was repeated for the third section. By the time the concrete was poured for the third section, the first section was ready to be collapsed and moved through the other two sections into a new position. This procedure was repeated until the lining was completed. One 50-ft section of lining was poured each day.

**Tunnel Lining by the Pumpcrete Method.** The most common method of placing concrete lining for a tunnel is by a Pumpcrete machine. The machine includes an agitator, or remixer hopper, a single- or double-cylinder piston pump, and a discharge pipe through which the concrete is pumped to the form.

Concrete that has been mixed by any convenient method is fed to the remixer hopper. This hopper, which serves as a storage reservoir ahead of the pump, is equipped with an agitator to ensure that a concrete of uniform quality will flow into the pump. The pump, which may consist of one or two horizontal cylinders, each with a single-acting piston, is located beneath the remixer hopper. When the piston is pulled back in a cylinder, concrete will flow by gravity from the remixing hopper through a valve-controlled opening into the cylinder. When the piston is pushed forward to eject the concrete from the cylinder, the inlet valve is closed

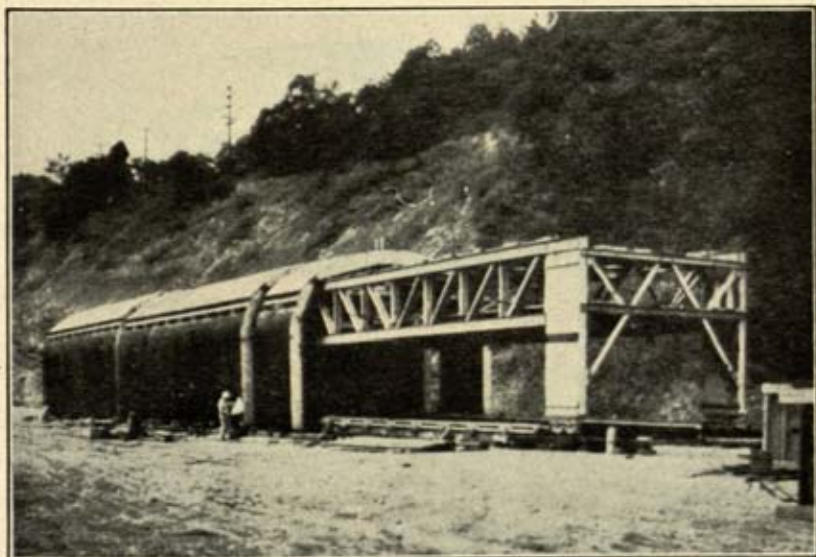


FIG. 12-34. Three sections of steel forms used to place concrete lining in a tunnel. (Blaw-Knox Co.)

and an outlet valve is opened by a system of mechanically operated levers and the concrete is forced the entire length of the discharge pipe and into the forms. Pumperete machines are available with capacities varying from about 10 to 60 cu yd of concrete per hour. A machine may be driven by a gasoline engine or an electric motor.

Figure 12-35 shows a complete mixing and pumping unit, consisting of a concrete mixer, a cylindrical remixer hopper, and a pump. The transmission pipe is connected to the pump at the left of the figure. The equipment at the right is a special aggregate car, which is lifted off the truck by a skip in order to dump the aggregate into the mixer. The aggregate may be delivered by truck. This machine is mounted on rails to permit it to operate inside a tunnel.



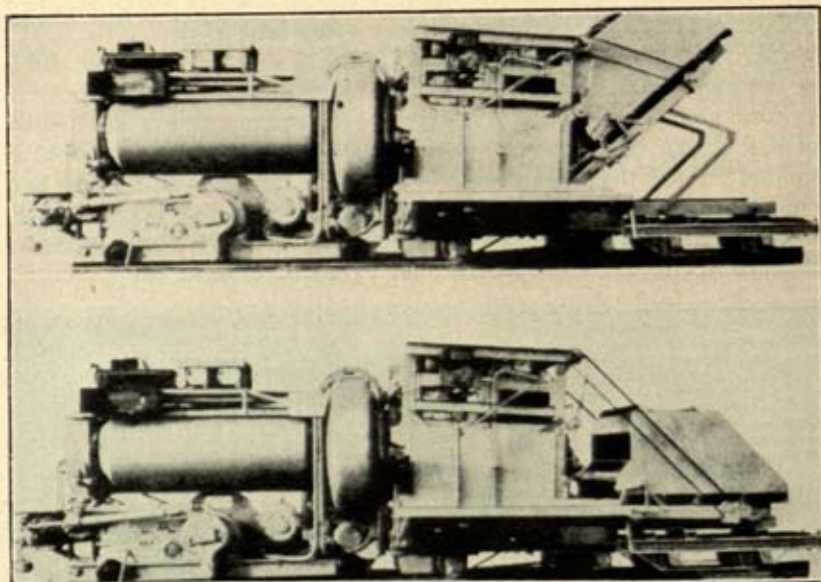


FIG. 12-35. Pumperete tunnel liner, consisting of a concrete mixer, agitator hopper, and pump. (*Chain Belt Co.*)

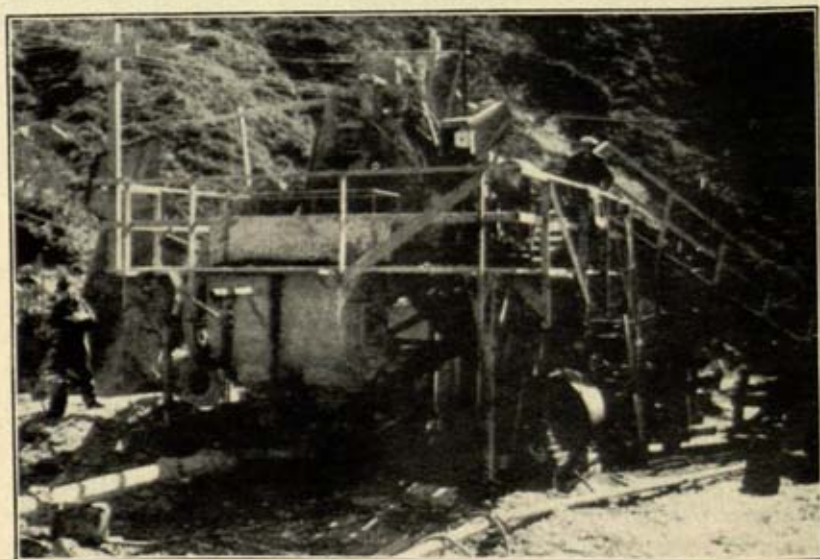


FIG. 12-36. Double Pumperete machine used to line the Gaviota Tunnel. (*Western Construction.*)

Figure 12-36 shows a double Pumpcrete machine in operation placing the concrete lining for the Gaviota Tunnel. The concrete mixer is mounted above the remixer hopper of the Pumpcrete machine.

A Pumpcrete machine may be set up outside a tunnel, near a portal, or near a vertical shaft which provides passage into the tunnel. When such an installation is used, a pipe is laid and the concrete is pumped to the forms. As the maximum horizontal distance that concrete can be pumped is approximately 1,000 ft, it will be necessary, in installing lining more than 1,000 ft from a portal, either to take the Pumpcrete machine

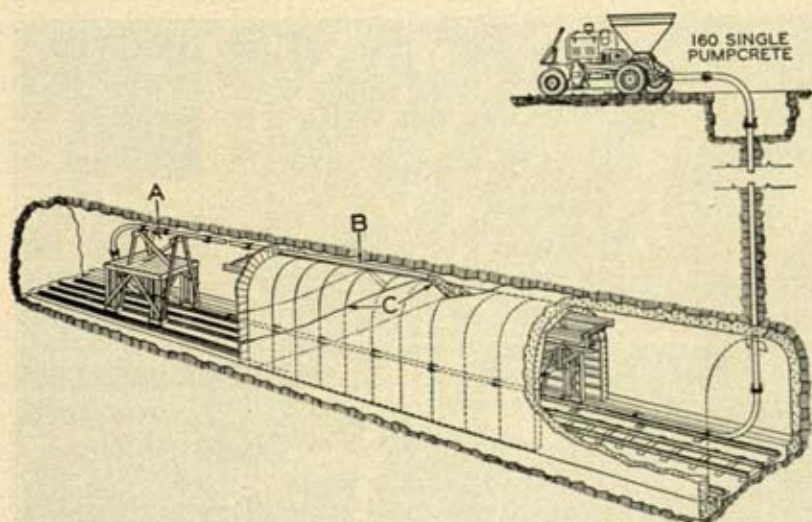


FIG. 12-37. Placing concrete lining in a tunnel with a Pumpcrete machine and a drop pipe. (Chain Belt Co.)

into the tunnel or to use more than one machine, with the machine on the outside pumping concrete into the remixing hopper of a machine set up inside the tunnel.

Figure 12-37 illustrates a method of providing access to a tunnel by installing a drop pipe from a Pumpcrete located at the surface of the ground above the tunnel. The drop pipe may be installed in a drilled hole or in a shaft.

If the bore is large enough, it may be desirable to set up the Pumpcrete machine on rails inside the tunnel near the lining operations, as illustrated in Fig. 12-38. It will be noted that the pipe, which is raised to near the roof in order that it may extend above the arch of the liner form, is supported on a jumbo, mounted on wheels, so that it may be moved along with the pumping equipment.



**Placing the Concrete Lining.** The concrete walls may be placed by directing the flow of concrete from the Pumperete pipe through temporary openings in the form. As soon as the walls are poured to the desired height, the discharge pipe can be connected to an arch pipe, which is already installed over the top of the form. The arch pipe, frequently referred to as a slick pipe, usually extends to within a few feet of the opposite end of the form. As the concrete fills the space behind and above the form, the arch pipe is withdrawn until the entire space is filled. An alternate method is to place all the concrete from the arch pipe by

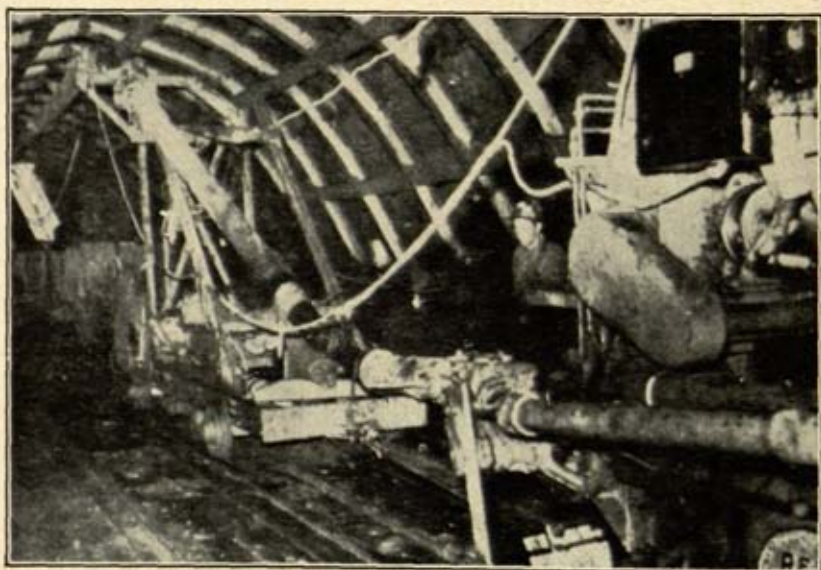


FIG. 12-38. Pumperete machine mounted on wheels to facilitate placing concrete in a tunnel. (Chain Belt Co.)

letting the concrete for the side walls flow down the back sides of the form. Care should be exercised to keep the depth of concrete on each side wall nearly the same in order to eliminate unbalanced forces on the form.

The injection of a quantity of compressed air into the arch pipe through a quick-opening valve is referred to as air slugging. The slugger is operated at intervals by opening the air valve and permitting a large volume of compressed air to flow into the slick line. This has the effect of ejecting the concrete with sufficient velocity to push it away from the discharge end of the pipe into the most remote spaces. For the slugging action to be effective, at least 75 to 150 cu ft of free air at 100 psi should be injected through a 1½- to 2-in. connection. The air usually is injected 20 to 60 ft behind the discharge end of the pipe.

As the space behind a set of forms is filled with concrete, the operation of the pump may be continued to build up considerable pressure between the form and the ground. Such a pressure is possible if the ends of the form are heavily bulkheaded to prevent the escape of concrete. This pressure forces concrete into all voids and spaces and produces a more solid lining.

The concrete should be vibrated as it is placed to eliminate voids and honeycombing. Internal vibrators may be used on the side walls if access to the concrete is possible, such as through doors in the walls of forms. If internal vibration is not practical, form vibrators may be used.

Additional information describing the operation of Pumpcrete equipment is given in Chap. 19.

**Tunnel Lining with Pneumatic Placers.** When a tunnel is so small that a Pumpcrete machine cannot be set up in it and the length is so great that concrete cannot be pumped through a pipe, a pneumatic placer may be used to place the concrete lining.

Pneumatic placing concrete for a tunnel lining involves using compressed air to force the concrete out of an airtight hopper through a discharge pipe. Concrete is mixed outside the tunnel, loaded into cars, and hauled by a locomotive to the placing equipment. The concrete is transferred from the cars to the pneumatic placer or hopper, the charging door is closed tightly, and compressed air is injected into the placer to force the concrete through the discharge pipe into the forms. As the discharge pipe is generally placed above the roof of a form, it is necessary to provide a pipe support whose height can be adjusted. Air compressors, set up outside the tunnel, supply compressed air through a pipe to a portable receiver located near the placer. The purpose of the air receiver is to provide an adequate supply of compressed air during the placing operation. All the equipment used in the tunnel should be mounted on wheels to permit easy movement along the rails.

A modification of the pneumatic placer described in the previous paragraph is mounted on wheels and used as a car to haul the concrete from the mixer. The hopper of each car has a large door at the top, which can be sealed and made airtight. A number of these placer cars, loaded with concrete, are pushed by a small locomotive to the section of a tunnel to be lined. The front car is connected to the discharge pipe; then compressed air is injected to force the concrete into the discharge pipe. This operation is repeated until all the cars are emptied. Placers of this type were used in lining four aqueducts, 6 ft in diameter, for the city of San Diego. Each placer had a capacity of  $1\frac{1}{4}$  cu yd of concrete.

When concrete is placed by the pneumatic method, care must be exercised to keep the velocity of the concrete low until the discharge end of the pipe is submerged in concrete. If this precaution is not observed, the



concrete leaving the pipe will be segregated. Each pneumatic placer should be equipped with a throttling valve to regulate the flow of air into it.

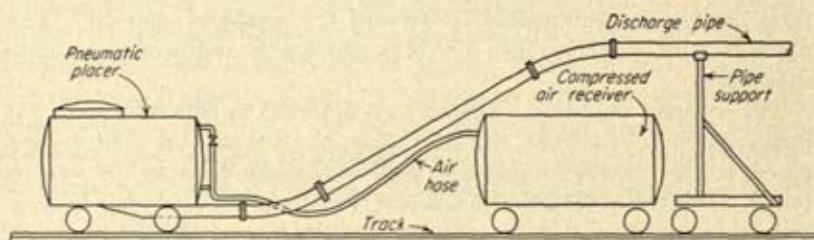


FIG. 12-39. Equipment used to place concrete lining by pneumatic method.

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## CHAPTER 13

### FOUNDATION GROUTING

**Need for Grouting.** Although large deposits of rock frequently are referred to as solid rock, in many instances they are not solid. These deposits may contain fissures, cavities, slips, faults, seams, or breaks, which make the deposits unsuitable for dams, reservoirs, buildings, bridge piers, locks, tunnels, etc. When subsurface investigations disclose the existence of such structural defects, it is necessary to adopt corrective steps if a foundation is to be made suitable for the intended use. If correction is impossible, or if it is unreasonably expensive, it may be necessary to abandon the site.

An operation to correct the foundation conditions is described as pressure grouting. The foundation under or adjacent to a structure is grouted for several reasons such as:

1. To solidify and strengthen the formation in order to increase its capacity to support a load
2. To reduce or eliminate the flow of water through a formation, such as under a dam or into a tunnel
3. To reduce the hydrostatic uplift under a dam

**Exploring to Determine the Need for Grouting.** The most satisfactory method of determining whether a foundation should be grouted is to obtain core samples from representative locations within the foundation area. Cores may be obtained with diamond or shot drills, usually diamond for the smaller sizes and shot for the larger sizes. Several shot-drilled holes, 30 in. in diameter or larger, may be desirable in order that a man may be lowered into them for visual inspection of the formation. The size, number, depth, and spacing of the exploratory holes should be planned to provide the greatest amount of information for the lowest practical cost. Increasing the number of holes will provide more dependable information, but it will increase the cost of exploration. For each project there must be a weighted balance between the need for foundation information and the cost of obtaining this information. The decision should be made by a competent foundation engineer.

An accurate record should be kept for each exploratory hole. The record should show the location, size, and depth of the hole and, with the core recovered, should show the physical nature of the formation. If a



core is recovered in long, continuous pieces, with little loss in length compared with the depth of the hole, this indicates a reasonably solid formation, which may require little or no grouting. However, if the core is badly broken, and if the recovered length is small in proportion to the depth of the hole, this indicates a bad foundation condition, which probably will require a large quantity of grout.

The approximate rate at which a hole will take grout may be determined by forcing water, under pressure, into the hole. For this purpose a section of pipe,  $1\frac{1}{2}$  to 2 in. in diameter, and 2 to 4 ft long, is sealed into the top of a hole with a threaded end projecting from the hole. As water is forced into the hole, the rate of flow and pressure should be recorded. If the rate of flow drops quickly, with a corresponding increase in pressure, this indicates the presence of only a few thin seams or fissures, which can be closed easily with grout. If the rate of flow remains high, with little or no increase in pressure, this indicates a highly porous formation, with extensive fissures, for which a large amount of grout will be required.

**Material Used for Grout.** The materials commonly used for grout include

1. Cement and water
2. Cement, rock flour, and water
3. Cement, clay, and water
4. Cement, clay, sand, and water
5. Asphalt
6. Clay and water

When a grout of cement and water only is to be injected into fine seams, it may be necessary to use as much as 10 parts of water to 1 part of cement, by volume, in order to obtain penetration. When the seams are large, the grout may be as dry as  $\frac{3}{4}$  part water, or less, to 1 part cement. For most grouting operations the ratio will vary from 1 to 2 parts water to 1 part cement. Usually, the most satisfactory grout is the stiffest mix that can be injected effectively. This should be determined by testing the rate of injection, using varying mix ratios.

Rock flour and clay may be added to cement grout in the interest of economy if the seams are small, while sand may be added if the seams are large enough to permit the sand to penetrate. Grout made of neat cement will give a higher strength than grout containing clay or sand. In grouting the foundation for the Norris Dam, the grout was mixed in the ratio 1 part cement, 1 part rock flour, by volume, with 3 lb calcium chloride per 100 lb of cement added to speed setting. For the Chickamauga Dam the mixture was 2 parts cement,  $\frac{1}{2}$  part bentonite, and 4 parts sand, by volume. As bentonite has the property of increasing up to several times its original volume when mixed with water, it is necessary to mix it thoroughly with water prior to adding cement and sand.

Table 13-1 gives the properties of certain admixtures when they are used with cement grout.

TABLE 13-1. PROPERTIES OF ADMIXTURES USED WITH CEMENT GROUTS

Admixture	Property
Calcium chloride Sodium hydroxide Sodium silicate	Accelerates setting time
Gypsum Lime sugar Sodium tannate	Retards setting time
Finely ground bentonite	Increases plasticity Reduces grout shrinkage
Clay Ground shale Rock flour	Reduces cost of grout Reduces strength of grout

The use of asphalt and clay grouts will be discussed later in the chapter.

**Drilling Patterns.** After a foundation has been explored and tested to determine the extent of grouting required, a drilling pattern should be adopted. The size, depth, and spacing of injection holes should give the best results at the lowest cost. It may be necessary to change the drilling pattern from time to time if the grouting operations encounter differences in foundation conditions.

In general, the smallest holes which will permit the injection of grout are the most desirable, as the size of a hole seems to be secondary so long as grout can be injected through it. Because of their lower cost, small holes permit a greater number to be drilled for a limited expenditure and, thus, increase the probability of obtaining a successful grouting operation.

A simple pattern, such as a hole spacing of 20 by 20 ft, with all holes  $2\frac{3}{4}$  in. in diameter and 40 ft deep, may be entirely satisfactory for one project but unsatisfactory for another. Figure 13-1 is a section through the Norris Dam showing shallow grout holes for consolidating the foundation under the dam and deeper holes for producing the impervious curtain to prevent the flow of water under the dam.

**Drilling Injection Holes.** Holes for the injection of grout may be drilled with jackhammers, wagon drills, diamond drills, or shot drills, depending on the terrain, class of foundation material, and size and depth of holes.

Diamond drills usually give holes that are uniform in shape and size, which are more satisfactory than holes drilled by other equipment when



packers must be installed for washing or grouting individual seams. Wagon drills are satisfactory for holes whose depths do not exceed 30 to 40 ft.

**Preparations for Grouting.** The preparations for washing or grouting seams by the full-length method consist in installing a section of pipe, usually  $1\frac{1}{2}$  to 2 in. in diameter, 18 to 36 in. long, in the grout hole, with the top end projecting out a short distance for connection to an air line or a pump. The space around the bottom of the pipe is closed with

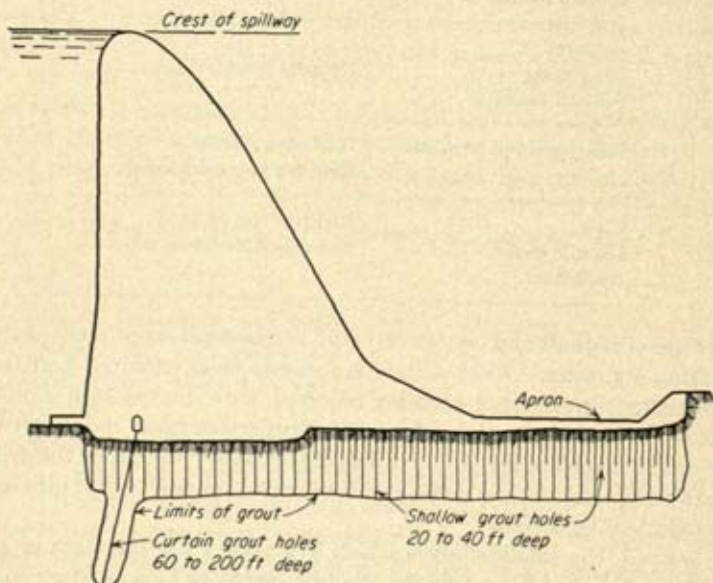


FIG. 13-1. Method of grouting under the Norris Dam.

oakum or other suitable material; then the balance of the space is filled with cement mortar, melted sulfur, or it may be calked with lead wool.

In order to reduce the danger of weakening the formation through fractures resulting from the application of excessive pressure, uplift gauges should be installed at several locations over the area to detect any lifting of the surface during the grouting operations.

**Washing the Seams.** When a formation is grouted with neat cement for consolidation purposes, it is desirable to deposit the cement in clean seams from which any clay or unconsolidated materials have been removed. The most effective method of removing such materials is to force a mixture of air and water through the seams. The removal of materials may be made more effective by alternately reversing the direction of flow of the air and water.

In washing a formation, a pattern of holes is selected. Some of the holes are capped for water, some for compressed air, and others are left open to permit the outflow of the washed materials. The direction of flow may be reversed by interchanging the pipe caps. When the water flowing from the uncapped holes clears up, indicating the removal of the unconsolidated materials, the caps are moved to another pattern of holes.

If the grout holes are deep and pass through several seams of unconsolidated materials, it may be desirable to isolate each seam in order that it may be washed individually. This is done by using an injection pipe, with the lower end closed, sufficiently long to extend below the lowest seam, with a perforated section long enough to extend completely through the seam. The pipe is equipped with an expandable packer above and below the perforated section, which is set opposite the seam to be washed. When the injection pipe is lowered into a hole and the packers are expanded, any air or water delivered to the pipe will be confined to a single seam.

It is possible to determine whether a seam is open from one hole to others by injecting into the seam water containing a coloring agent, such as fluorescein dye. If the colored water appears in other holes, this indicates open passages through the seam.

**Grouting Pressures.** The most suitable pressure for grouting operations is difficult to determine in advance. Some engineers follow a general practice of using a pressure of 1 psi for each foot of depth of hole. There is no logical proof or demonstration that this is the most satisfactory pressure.

In the interest of economy and effectiveness it is desirable to use the highest pressure that is safe. However, when grout is forced into a seam under pressure, it is possible that the total upward force on the formation above the seam may exceed the combined weight and resisting strength of the formation. If this condition is permitted to occur, the entire formation may be lifted upward, with a resulting fracture that is more serious than the original condition that grouting is supposed to correct. Thus, it is possible for grouting to do more harm than good unless it is injected under careful supervision.

If the weight of a rock formation is 150 lb per cu ft, the unit pressure on a horizontal plane, 1 ft below the surface of the rock, will be  $150 \text{ lb} \div 144 \text{ sq in.} = 1.04 \text{ psi}$ . At a depth of 100 ft the pressure resulting from the weight only will be 104 psi. As most rocks weigh 150 lb or more per cubic foot, it is improbable that a grouting pressure equal to 1 psi for each foot of depth will endanger a foundation formation provided the pressure is confined to the intended depth.

When grout is injected by the full-length-hole method, care must be exercised to prevent the pressure of the grout near the surface of the



ground from exceeding the maximum safe pressure. If the pressure at the bottom of a hole 40 ft deep is 40 psi, the pressure at a depth of 20 ft will be equal to 40 psi minus the hydrostatic pressure of 20 ft of grout. The hydrostatic pressure of cement grout may be determined from the curve in Fig. 13-2. If the grout is mixed in the ratio  $1\frac{1}{2}$  parts of water to 1 part cement, the pressure change per foot of depth will be 0.66 psi. In 20 ft the reduction in pressure will be  $20 \times 0.66 = 13.2$  psi. Thus, for the case cited above the pressure at a depth of 20 ft will be  $40 - 13.2 = 26.8$  psi. The pressure at the surface of the ground will be

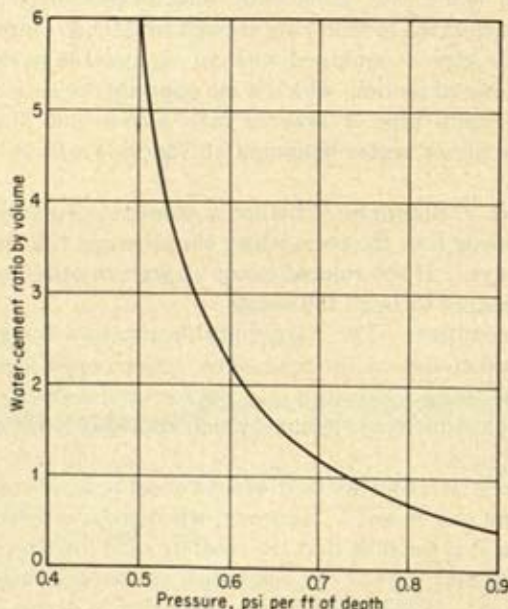


FIG. 13-2. Hydrostatic pressure produced by cement grout.

$40 - 40 \times 0.66 = 13.6$  psi. If the injection of grout by the full-length-hole method produces objectionably high pressures near the top of a hole, this objection may be overcome by injecting grout in limited depth zones.

The pressures at which grout is injected frequently are varied with the depth of injection and the stage at which the grout is injected. For example, when the foundation under a concrete dam is grouted for consolidation purposes, the grout may be injected in two or three stages. The first stage might consist in drilling holes 20 to 40 ft deep, with a spacing of 20 ft each way. The pressure at the bottom of these holes might be limited to a maximum of 40 to 50 psi. The second stage might consist in drilling holes 40 to 60 ft deep, with a spacing of 10 ft each way. If the grout is injected after the dam is partly constructed, the pressure

at the bottom of the holes might safely be increased to 80 to 100 psi. The third and last stage might consist in drilling holes 60 to 100 ft or more deep for final high-pressure grouting under the dam. These holes might be spaced as close as 5 ft apart in a single row along the axis of the dam, to permit the grouting of a solid cutoff curtain, to prevent the flow of water under the dam. If these holes are not grouted until the dam is completed, the pressure may be as high as several hundred pounds per square inch. Figure 13-1 illustrates a method of grouting the foundation for a dam.

**Equipment for Cement Grouting.** The most common method of injecting cement grout is to use one or more piston-type pumps to produce

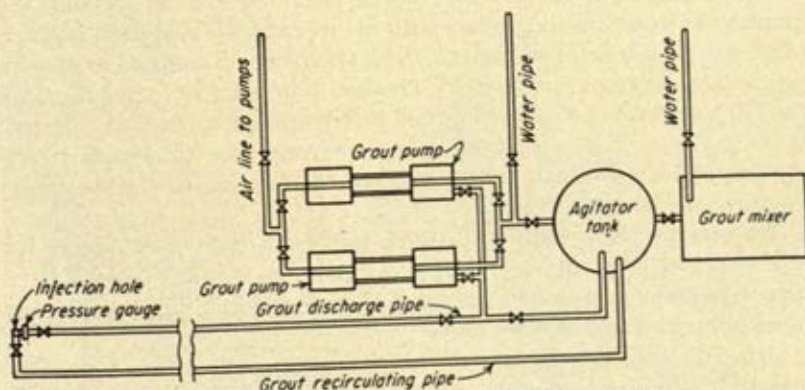


FIG. 13-3. Boulder-type equipment used to inject cement grout.

the necessary pressure. The pumps usually are air-driven duplex-double-acting types so constructed that the number of strokes per minute and the pressure on the grout may be varied by regulating the quantity of compressed air supplied to the pump.

The equipment will include:

1. One or more air compressors
2. One or two grout mixers
3. One agitator-type reservoir tank
4. One or more grout pumps
5. Grout discharge pipe or hose, valves, pressure gauges, etc.

The grout mixer contains a shaft with paddles, operated by a motor. After the grout is mixed, it is discharged into a tank, with an agitator to prevent separation of the solids from the water. The pumps draw their charges directly from the agitator tank.

The essential parts of the Boulder-type grout unit are illustrated in Fig. 13-3. The location of the component parts may be modified to fit



any particular injection conditions. The grout discharge line may be a pipe, a rubber hose, or a combination thereof. The use of a hose will facilitate moving from one grout hole to another.

Figure 13-3 shows a grout recirculating pipe or line whose primary function is to permit grout to flow through the pumps and discharge pipe at a uniform rate even though the rate of injection into a hole is reduced as the cavities are filled.

It is good practice to install two grout pumps, even though one pump can supply all the grout that will be needed. In the event of a pump failure, the auxiliary pump can be placed in operation immediately, thereby reducing the danger of losing a partly filled hole or group of holes.

**Injecting Cement Grout.** The records that were kept at the time the grout holes were drilled, together with the records obtained from washing the holes, if such an operation was performed, should serve as a guide in estimating the grout mix to use. The best results are obtained by using the thickest grout that can be injected without plugging the hole. It may be necessary to start with a batch of thin grout, then thicken each succeeding batch by reducing the water-cement ratio, until the maximum practical thickness is determined.

The specifications covering the grouting of a project may require the injection of grout at a given pressure until the rate of injection for a given hole shall diminish to a specified amount or until a hole will not take any more grout at a specified pressure.

Grout may be injected into the full length of a hole at one time, or it may be injected into a portion of the length only. If the latter method is used, it is possible to apply a high pressure in injecting at the bottom of a hole. As the depth of injection is reduced, the pressure may be reduced accordingly. This is referred to as the zone method of grouting. In order to inject grout by the zone method, it is necessary to use an injection pipe that is long enough to reach the lowest zone of injection. The zone to be grouted at a given time is isolated from the rest of the hole by means of a packer, which is set near the bottom of the injection pipe and just above the top of the zone. Most packers are of removable types, which permit them to be reused many times.

**Pressure Grouting with Asphalt.** If a formation contains fissures with water flowing through them, it will be difficult or even impossible to consolidate the formation with cement grout. The velocity of the water tends to sweep the grout through the openings without giving it an opportunity to solidify.

In numerous instances the injection of asphalt grout into fissures containing flowing water has sealed the fissures and stopped or reduced the flow of water. After the flow of water is stopped, it is possible to inject cement grout to complete the consolidation operation. Thus, the pri-

mary function of asphalt grout is to seal off the flow of water in order that cement grout may be retained in the fissures.

The heated asphalt is injected through a perforated pipe, which may have a steam line running through it, as a means of keeping the asphalt at the desired temperature until it flows into the formation. An alternate method of heating the asphalt in the injection pipe is to install an electric wire inside the pipe, with the pipe completing the electrical circuit. An electric current through the wire will heat the asphalt. When hot asphalt flows from an injection pipe into the openings in a formation, the outside surface of the asphalt tends to solidify, but, because of the low

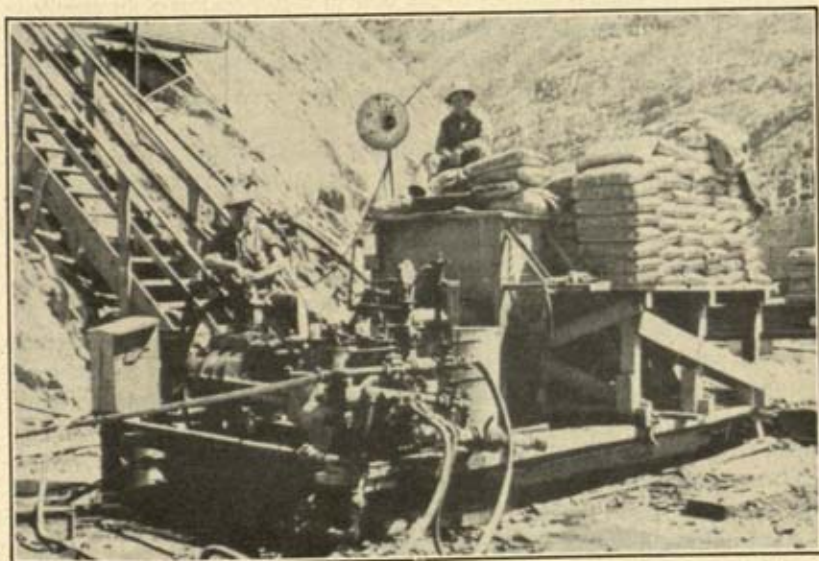


FIG. 13-4. Air-driven grout pumps on the Davis Dam. (Gardner-Denver Co.)

heat conductivity, the inside tends to remain a liquid for some time. The pressure from the injection pipe will keep the interior part of the asphalt flowing for a considerable distance, several hundred feet in some instances. As the grout solidifies under pressure, it conforms with the shape of the fissures and seals them against the flow of water.

The equipment required to inject asphalt grout consists of a heating kettle, a piston-type pump, an air compressor or an electric motor to operate the pump, a source of electrical current, plus a supply of pipe, hose, valves, and pressure gauges.

An interesting example of the use of asphalt grout to stop the flow of water through a fissurized formation was developed in correcting the leakage from the reservoir at Great Falls Dam [1]. The limestone abut-



ments to the dam had developed extensive fissures below the level of the water in the reservoir, through which a large quantity of water flowed from the reservoir into the river channel below the dam. Injection holes, which were drilled through the fissurized formations, were grouted first with asphalt to stop the flow of water, after which the consolidation was completed using cement grout. Asphalt grout, at temperatures varying from 300 to 350°F, was injected at rates varying from 40 to 60 cu ft per hr.

**Clay Grouting.** Grout mixtures of clay and water or cement, clay, and water have been used successfully to fill large seams and cavities subjected to low hydrostatic heads. While clay adds little, if any, strength to a foundation, its high resistance to the flow of water makes it an excellent barrier to water seepage. It can be mixed with water to any desired consistency and injected with equipment similar to that used for cement grouting. Pressures in excess of 100 psi have been used to inject clay grout into formations.

Clay is not a satisfactory grout for use in fissures which contain flowing water. The primary advantage of clay as a grouting material is its low cost compared with cement and asphalt, especially when a deposit of clay is available near the site to be grouted.

In sealing large seams and solution channels in the limestone along the ridges of the reservoir at Madden Dam in the Canal Zone, 70,000 cu yd of clay grout was used [2]. The water content of the mixture varied from 43 to 55 per cent by weight, depending on the back pressure of the fissures.

Seams in the abutments of the Chickamauga Dam were sealed with a cement-clay grout [3]. For this project grout was mixed in the proportions 1½ parts of cement, 7 parts of clay, and 6 parts of water, by volume. Prior to mixing, the clay was screened to remove any particles or objects that would not mix readily in the grout.

**Determining the Effectiveness of Grouting.** A question that usually arises in connection with a grouting operation is how to determine whether the operation has been successful. Several methods have been used with varying degrees of success.

To determine the extent of flow of grout from the injection holes through a formation, several holes are left open to see whether grout will appear in them. The appearance of grout in these holes serves as a guide to indicate the extent of flow.

Prior to concluding a grouting operation, additional exploratory holes may be drilled at various locations within the area that has been grouted in order to obtain cores from the formation. If these cores show the existence of sufficient grout to produce good consolidation where voids originally existed, this indicates that the grouting operation has been successful. The effectiveness of the grouting operation also may be tested by attempting to inject water or grout into the holes from which

the cores were obtained. If these holes refuse to take grout, the test indicates that the formation has been consolidated adequately by previous injections.

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## CHAPTER 14

### PILES AND PILE-DRIVING EQUIPMENT

**Introduction.** This chapter deals with the selection of load-bearing piles and the equipment required to drive the piles.

Load-bearing piles, as the name implies, are used primarily to transmit loads through soil formations with poor supporting properties into or onto formations that are capable of supporting the loads. If the load is transmitted to the soil through skin friction between the surface of the pile and the soil, the pile is called a *friction pile*. If the load is transmitted to the soil through the lower tip, the pile is called an *end-bearing pile*. Many piles depend on a combination of friction and end bearing for their supporting strengths.

**Types of Piles.** Piles may be classified on the basis of their use or the materials from which they are made. On the basis of use there are two major classifications, *sheet* and *load-bearing*.

Sheet piling is used primarily to resist the flow of water and loose soil. Typical uses include cutoff walls under dams, cofferdams, bulkheads, trench sheeting, etc. On the basis of the materials from which they are made sheet piling may be classified as *steel*, *wood*, and *concrete*. The use of sheet piling will be discussed in Chap. 16.

On the basis of the material from which they are made and the method of constructing and driving them, load-bearing piles may be classified as follows:

1. Timber
  - a. Untreated
  - b. Treated with a preservative
2. Concrete
  - a. Precast
  - b. Cast-in-place
3. Steel
  - a. H-section
  - b. Steel-pipe
4. Composite

Each type of load-bearing pile has a place in the field of construction, and for some projects more than one type may seem satisfactory. It is

the duty of the engineer to select that type of pile which is most satisfactory for a given project, considering all the factors that affect the selection. Among the factors that will influence his decision are the following:

1. Type, size, and weight of the structure to be supported
2. Physical properties of the soil at the site
3. Depth to a stratum capable of supporting the piles
4. Possibility of variations in the depth to a supporting stratum
5. Availability of materials for piles
6. Number of piles required
7. Facilities for driving piles
8. Comparative costs in place
9. Durability required
10. Types of structures adjacent to the project
11. Depth and kind of water, if any, above the ground into which the piles will be driven

To illustrate the effect which these factors have on the selection of types of piles, consider factor 4. If soil borings at the site of a project indicate that the depth to a stratum capable of supporting piles varies considerably, precast concrete piles should not be selected. Regardless of other desirable factors the difficulty and expense of increasing or decreasing the length of such piles should eliminate them from consideration. If concrete piles are desired, one of the cast-in-place types should be selected.

**Timber Piles.** Timber piles are made from the trunks of trees. While such piles are available in most sections of the nation and the world, it is becoming more difficult to obtain long, straight timber piles. Pine piles are reasonably available in lengths up to 60 ft, while Douglas fir piles are available in lengths in excess of 100 ft from the Pacific Northwest.

Among the advantages of timber piles are the following:

1. The more popular lengths and sizes are available on short notice.
2. They are economical in cost.
3. They are handled easily, with little danger of breakage.
4. They can be cut off to any desired length after they are driven.
5. They can be pulled easily in the event removal is necessary.

Among the disadvantage of timber piles are the following:

1. It may be difficult to obtain piles sufficiently long and straight for some projects.
2. It may be difficult or impossible to drive them into hard formations.
3. It is difficult to splice them to increase their lengths.
4. While they are satisfactory when used as friction piles, they are not suitable for use as end-bearing piles under heavy loads.
5. The length of life may be short unless the piles are treated with a preservative.



**Precast Concrete Piles.** Square and octagonal piles are cast in horizontal forms, while round piles are cast in vertical forms. After the piles are cast, they should be cured under damp sand, straw, or mats for the period required by the specifications, frequently 21 days.

With the exception of short lengths, precast concrete piles must be reinforced with sufficient steel to prevent damage or breakage while they are being handled from the casting beds to the driving positions. The Foundation Code of the City of New York, adopted in 1948, specifies that precast concrete piles shall contain longitudinal reinforcing steel in an amount not less than 2 per cent of the volume of a pile [1]. Lateral steel shall be at least  $\frac{1}{4}$ -in.-diameter round bars, spaced not more than 12 in.

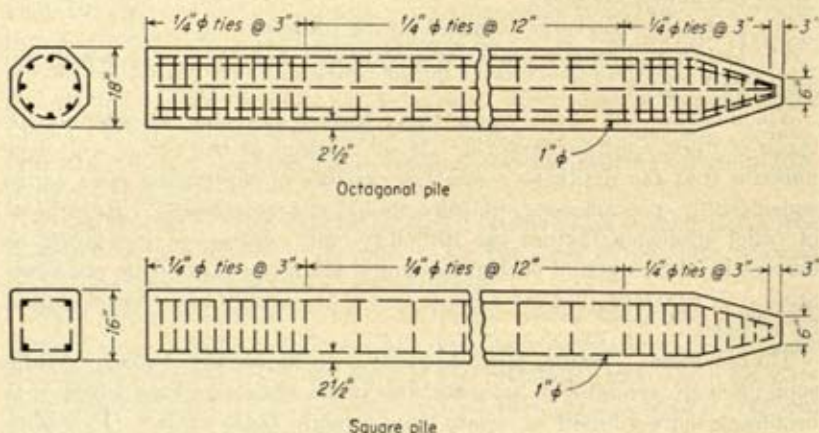


FIG. 14-1. Typical details of precast concrete piles.

apart, except at the top and bottom 3 ft of a pile, where the spacing shall not exceed 3 in. The concrete cover over the reinforcing steel shall be at least 2 in. Figure 14-1 shows typical details of precast concrete piles.

Concrete piles should be cast as near the site of use as possible in order to reduce the cost of handling them from the casting beds to the pile driver. In the event it is necessary to transport them to a driver, this may be done by a truck, as illustrated in Fig. 14-3. These piles, which were used in constructing a highway bridge in North Carolina, varied in length from 36 to 60 ft. For handling concrete piles, care must be exercised to prevent breakage or damage due to flexural stresses. Long piles should be picked up at several points to reduce the unsupported lengths. Figure 14-4 shows a barge-mounted pile driver installing precast concrete piles for a pier.

Concrete piles may be cast in any desired sizes and lengths. Those used in constructing the Morganza Floodway on the Mississippi River

were square and octagonal in cross section, 20 in. wide, and varied in length to more than 100 ft. Almost 360,000 lin ft of piles was driven on this project. The piles were cast in prefabricated steel forms, loaded on railroad cars by a gantry crane with a 135-ft span, and hauled to the

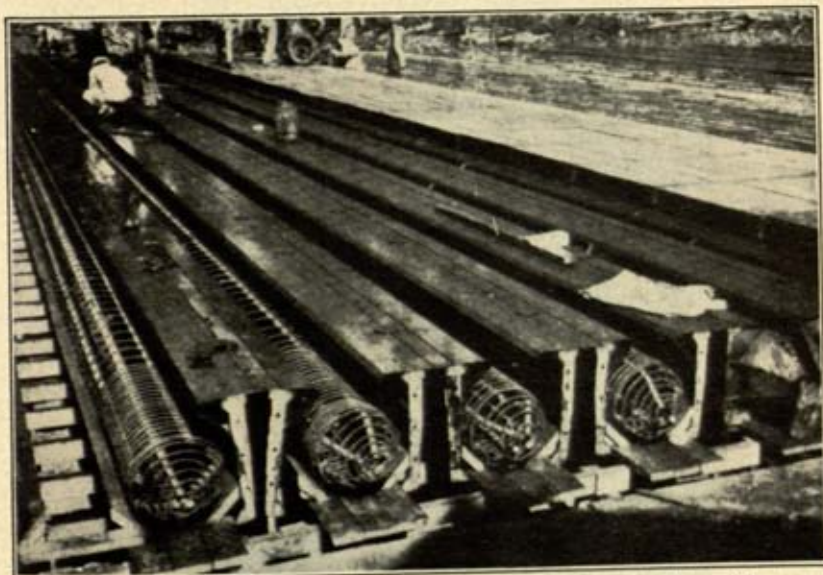


FIG. 14-2. Preparing forms and reinforcing for precast concrete piles. (*Raymond Concrete Pile Co.*)



FIG. 14-3. Hauling precast concrete piles. (*Construction Methods and Equipment.*)

driving site, where they were driven by special rigs [2]. For a project as big as this the large investment in special equipment is justified, but for a small project the required investment in equipment probably would make the cost prohibitive. Thus, the maximum-size concrete piles that can be used on a project may be determined by economy.



One of the disadvantages of using precast concrete piles, especially for a project where different lengths are required, is the difficulty of reducing or increasing the lengths of the piles.

If a pile proves to be too long, it is necessary to cut off the excess length. This is done, after a pile is driven to its maximum penetration, by chipping the concrete away from the reinforcing steel, cutting the reinforcing with

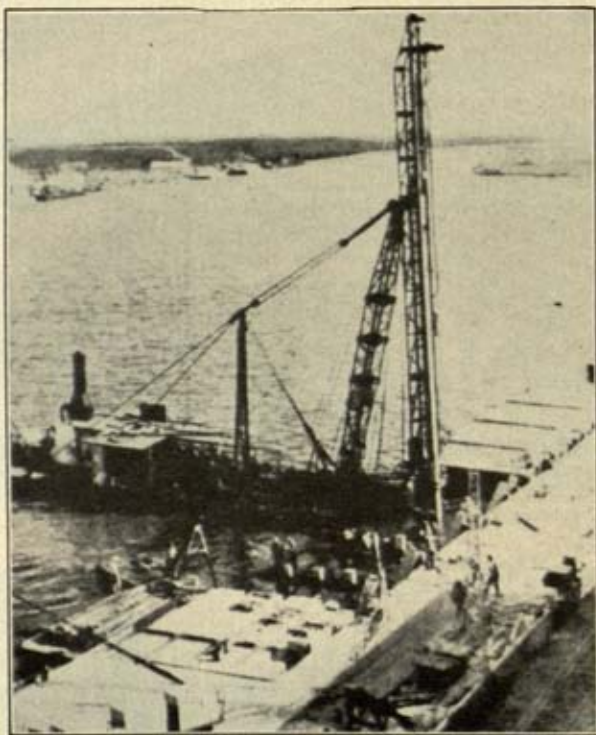


FIG. 14-4. Barge-mounted rig driving precast concrete piles. (*Raymond Concrete Pile Co.*)

a gas torch, then removing the surplus length of the concrete core. This operation represents a waste of material and time which can be very expensive.

When a precast concrete pile does not develop sufficient driving resistance to support the design load, it may be necessary to increase the length and drive the pile to a greater depth. Unless the reinforcing bars extend above the top of a pile, it will be necessary to chip the concrete back far enough to permit additional reinforcing to be welded to the original longitudinal bars. Then the concrete is placed for the added length.

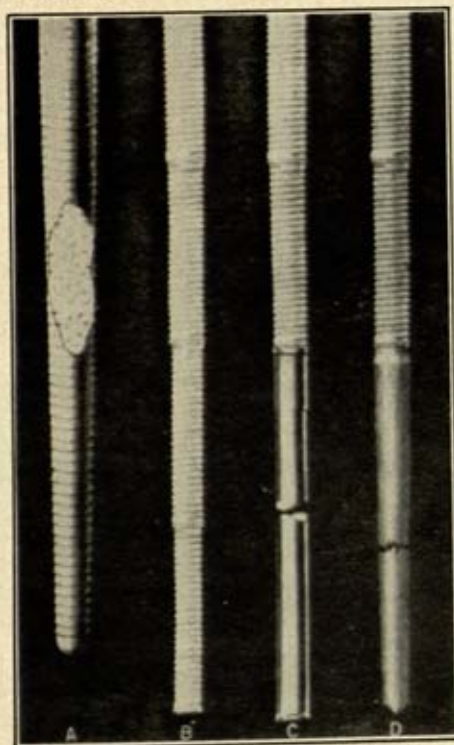


FIG. 14-5. Raymond cast-in-place concrete piles: (A) standard pile; (B) step-taper pile; (C) pipe step-taper pile; (D) composite pile. (Raymond Concrete Pile Co.)

TABLE 14-1. DATA ON RAYMOND STANDARD CONCRETE PILES

Length of pile, ft	Diameter, in.		Volume of concrete, cu ft
	Tip	Top	
15	8	14	10.5
20	8	16	17.0
25	8	18	25.0
30	8	20	35.0
35	8	22	47.0
37	8	22.8	52.5

Among the advantages of precast concrete piles are the following:

1. High resistance to chemical and biological attacks.
2. High strength.
3. A pipe may be installed along the center of a pile to facilitate jetting.



Among the disadvantages of precast concrete piles are the following:

1. It is difficult to reduce or increase the length.
2. Large sizes require heavy and expensive handling and driving equipment.
3. Inability to obtain piles by purchase may delay the starting of a project.
4. Possible breakage of piles during handling or driving produces a delay hazard.

**Cast-in-place Concrete Piles.** As the name implies, cast-in-place concrete piles are constructed by depositing the freshly mixed concrete in

place in the ground and letting it cure there. The two principal methods of constructing such piles are:

1. Driving a metallic shell, leaving it in the ground, and filling it with concrete

2. Driving a metallic shell and filling it with concrete as the shell is pulled from the ground

There are several modifications for each of the two methods.

The more commonly used piles constructed by these two methods are described in the following articles.

**Raymond Standard Concrete Piles.** A thin corrugated-steel shell, closed at the bottom with a steel driving boot, is placed on the outside of a collapsible steel mandrel or core; then the core and the shell are driven into the ground to the desired depth. When sufficient driving resistance is developed, the core is collapsed and withdrawn from the shell. The shell is inspected by using the light from a mirror, flashlight, or droplight and, if found in good condition, is filled with concrete. In the event a

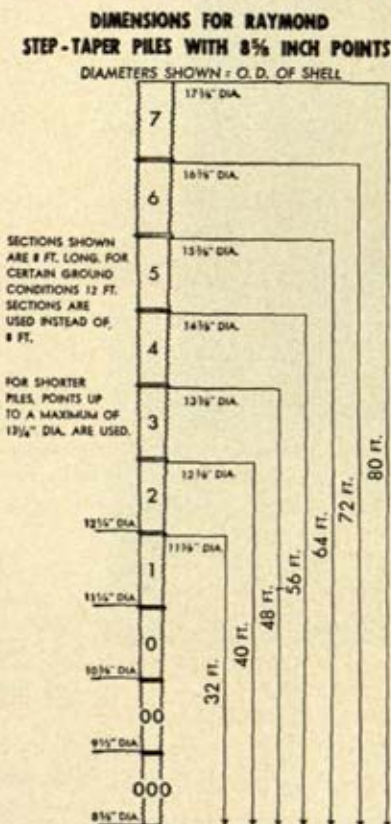


FIG. 14-6. Dimensions for Raymond step-taper piles. (Raymond Concrete Pile Co.)

shell is damaged during driving, it may be withdrawn and replaced with a good shell.

The shells are available in lengths up to 37 ft. These piles are illustrated in Fig. 14-5. Table 14-1 gives data on these piles.

**Raymond Step-taper Concrete Piles.** The step-taper pile is installed by driving a spirally corrugated steel shell, made up of sections 4 or 8 ft long, with successive increases in diameter of 1 in. for each 8-ft section. A corrugated sleeve at the bottom of each section is screwed into the top

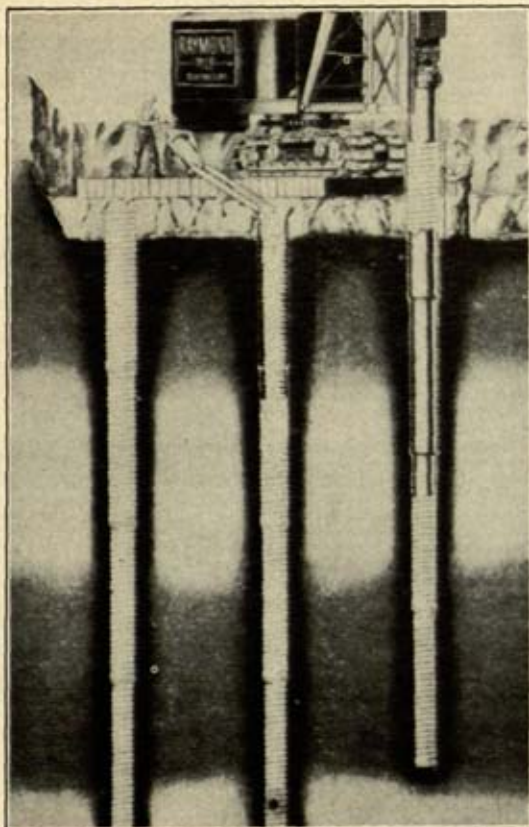


FIG. 14-7. Driving Raymond step-taper piles. (Raymond Concrete Pile Co.)

of the section immediately below it. Piles of the necessary length, up to a maximum of 80 ft, are obtained by joining the proper number of sections at the job. The shells are available in various gauges of metal to fit different job conditions. The bottom of the shell, whose diameter can be varied from  $8\frac{1}{2}$  to  $13\frac{1}{8}$  in., is closed prior to driving by a flat steel plate or a hemispherical steel boot.

After a shell is assembled in the desired length, a step-tapered rigid-steel core is inserted and the shell is driven to the desired penetration.



The core is removed, and the shell is filled with concrete. Figure 14-6 gives the dimensions of a step-taper pile shell. Figure 14-7 illustrates steps in driving these piles. Table 14-2 gives data on step-taper piles. This table may be used to determine the volume of concrete required to fill the shell of any pile section and length. For example, the volume of concrete required to fill a shell 32 ft long, with a tip diameter of  $12\frac{1}{4}$  in. and a top diameter of  $16\frac{1}{4}$  in., will be  $58.00 - 22.10 = 35.90$  cu ft.

TABLE 14-2. DATA ON RAYMOND STEP-TAPER CONCRETE PILES

Length of pile, ft	Diameter, in.		Volume of concrete, cu ft
	Tip	Top	
24	$8\frac{1}{2}$	$10\frac{1}{4}$	10.83
32	$8\frac{1}{2}$	$11\frac{1}{4}$	15.93
40	$8\frac{1}{2}$	$12\frac{1}{4}$	22.10
48	$8\frac{1}{2}$	$13\frac{1}{4}$	29.20
56	$8\frac{1}{2}$	$14\frac{1}{4}$	37.50
64	$8\frac{1}{2}$	$15\frac{1}{4}$	47.15
72	$8\frac{1}{2}$	$16\frac{1}{4}$	58.00
80	$8\frac{1}{2}$	$17\frac{1}{4}$	70.40

**Monotube Piles.** The monotube pile is obtained by driving a fluted, tapered steel shell, closed at the tip with an 8-in.-diameter driving point,

TABLE 14-3. DATA ON MONOTUBE PILES

Length, ft	Type F, taper 1 in. in 7 ft, tip diam 8 in.				
	Butt diam, in.	Weight of shell, lb			Volume of concrete per pile, cu ft
		No. 11 gauge	No. 9 gauge	No. 7 gauge	
10	9.4	137	166	195	3.5
15	10.1	202	248	293	5.8
20	10.8	271	334	396	8.4
25	11.5	348	430	511	11.4
30	12.2	425	526	627	14.3
35	12.9	512	635	757	18.1
40	13.6	600	744	988	22.1
45	13.9	700	865	1,030	26.6
50	14.5	794	983	1,172	31.0
55	15.3	894	1,111	1,328	36.5
60	16.0	1,004	1,244	1,438	42.2
70	17.4	1,231	1,529	1,826	55.5
75	18.1	1,356	1,685	2,013	62.8

to the desired penetration. The shell is driven without a mandrel, inspected, and filled with concrete. Any desired length of shell, up to approximately 125 ft, may be obtained by welding extensions to a standard-length shell.

Table 14-3 gives dimensions and other information on one type of tube.

**Button-bottom Piles.** This pile is installed by driving a 17-in.-diameter precast concrete button to the desired depth. The button is tapered

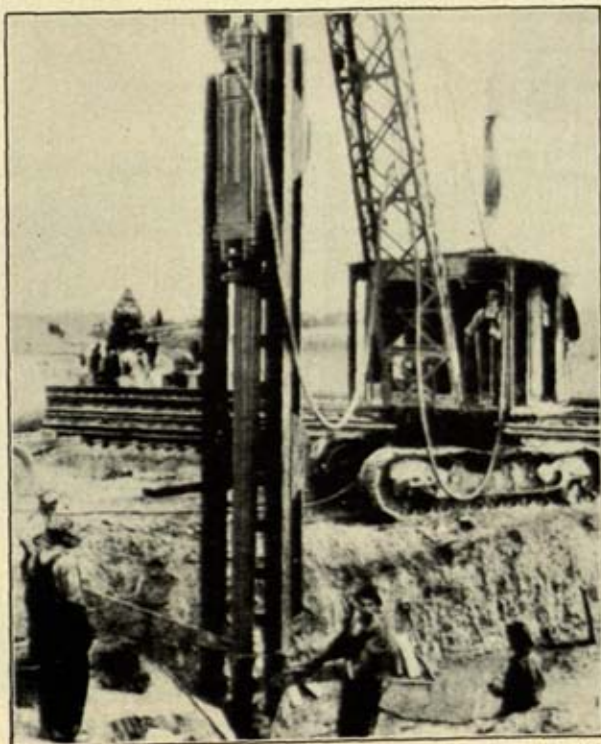


FIG. 14-8. Double-acting steam hammer driving monotube pile. (McKiernan-Terry Corp.)

on the bottom to facilitate its penetration through the soil and on the top to permit a steel pipe to fit over it. A steel pipe, usually 14 in. in diameter, with  $\frac{1}{2}$ -in.-thick walls, is placed on top of the button. After the pipe and the button are driven to the desired depth, a corrugated steel shell is lowered inside the pipe until it rests on the button, where a positive connection between the shell and the button is effected. The pipe is withdrawn, and the shell is filled with concrete. Figure 14-9 shows steps in driving button-bottom piles.



The developer of this type pile, the Western Foundation Corporation, reports that piles can be installed economically to depths in excess of 100 ft and to carry loads of 60 tons or more.

Figure 14-10 shows the driving casing being lowered onto a button in preparation for driving.

One of the chief advantages of this type pile is the ability of the button to pierce obstacles in a formation. On the Stuyvesant Town project in New York City, button-bottom piles were driven through old concrete

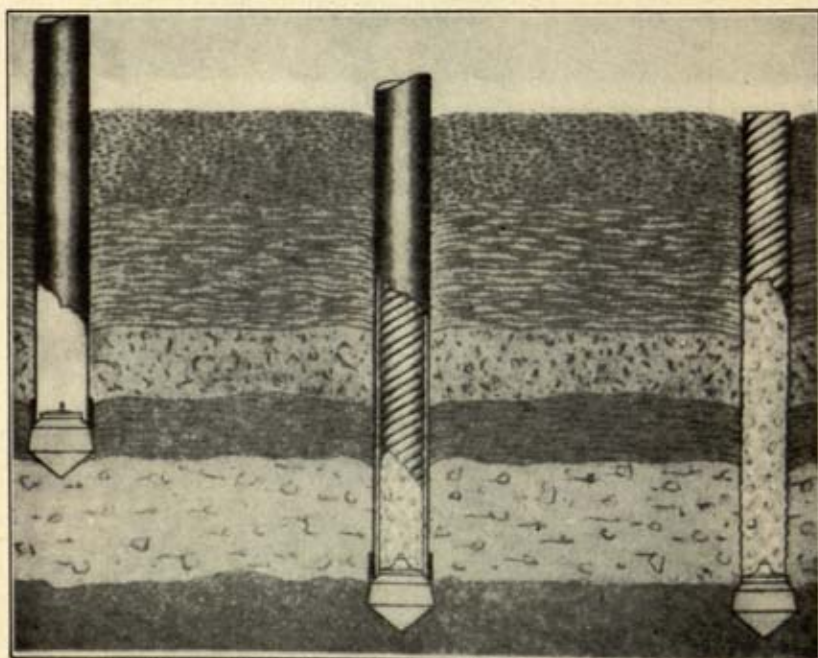


FIG. 14-9. Steps in driving button-bottom piles. (Western Foundation Corp.)

foundations without serious difficulty [3]. The projecting shoulder of the button beyond the outside diameter of the driving casing eliminates skin friction as a driving factor. Thus, practically all the energy of the pile hammer is concentrated on driving the button.

**Pedestal-type Concrete Piles.** A pedestal-type concrete pile is installed by driving a heavy-wall steel pipe, with the bottom temporarily closed, to the desired depth. The lower part of the pipe is filled with concrete, on which a steel core is placed. With the pipe lifted slightly, usually 2 or 3 ft, the energy from a pile hammer is transmitted by the core to the concrete, to force the concrete out into the form of a pedestal. As



FIG. 14-10. Lowering the driving casing onto a button in preparation for driving. (Western Foundation Corp.)

the pipe is pulled from the hole, concrete is added. The weight of the core, which is returned to the pipe after each batch of concrete is added, is sufficient to force the concrete to completely fill the hole left by the pipe.

Figure 14-11a illustrates an uncased and Fig. 14-11b a cased pedestal pile.

**Advantages and Disadvantages of Cast-in-place Concrete Piles.** Among the advantages of cast-in-place concrete piles are the following:

1. The lightweight shells may be handled and driven easily.
2. Variations in length do not present a serious problem. The length of a shell may be increased or decreased easily.
3. The shells may be shipped in short lengths and assembled at the job.
4. Excess reinforcing, to resist stresses caused by handling only, is eliminated.

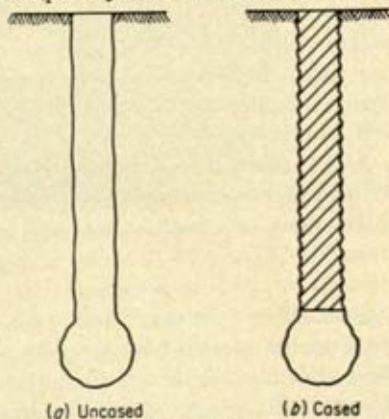


FIG. 14-11. Pedestal-type concrete piles.



5. The danger of breaking a pile while driving is eliminated.
6. Additional piles may be provided quickly if they are needed.

Among the disadvantages of cast-in-place concrete piles are the following:

1. A slight movement of the earth around an unreinforced pile may break it.
2. An uplifting force, acting on the shaft of an uncased and unreinforced pile, may cause it to fail in tension.
3. The bottom of a pedestal pile may not be symmetrical.

**Steel-pipe Piles.** These piles are installed by driving pipes to the desired depth and filling them with concrete. A pipe may be driven with the lower end closed with a plate or a steel driving point, or the pipe may be driven with the lower end open. Pipes varying in diameter from 6 to 30 in. or more have been driven in lengths varying from a few feet to more than 200 ft [4, 5].

A closed-end pipe pile is driven in any conventional manner, usually with a pile hammer [6]. If it is necessary to increase the length of a pile, two or more sections may be welded together or sections may be connected by using an inside sleeve for each joint. This type of pile is particularly advantageous for use on jobs when the headroom for driving is limited and short sections must be added to obtain the desired total length.

An open-end pipe pile is installed by driving the pipe to the required depth, removing the material from inside, and filling the space with concrete. Because open-end pipe piles offer less driving resistance than closed-end piles, a smaller pile hammer may be used. The use of a light hammer is desirable when piles are driven near a structure whose foundation might be damaged by the impact of the blows from a large hammer. Open-end piles may be driven to depths which could never be reached with closed-end piles.

After a pile is driven to the desired depth, the material inside is removed by bursts of compressed air, a mixture of water and compressed air, an earth auger, or a small orange-peel bucket; then the pipe is filled with concrete [7]. Figure 14-12 shows a barge-mounted driver driving 24-in. pipe piles up to 220 ft long for a dry dock in Portland, Oreg. [4].

**Steel Piles.** In constructing foundations that require piles driven to great depths, steel H piles probably are more suitable than any other type. Steel piles may be driven through hard materials to a specified depth to eliminate the danger of failure due to scouring, such as under a pier in a river. Also, steel piles may be driven to great depths through poor soils to bear on a solid rock stratum. The great strength of steel combined with the small displacement of soil permits a large portion of the energy from a pile hammer to be transmitted to the bottom of a pile. As a result, it is possible to drive steel piles into soils which could not be pene-

trated by any other type pile. However, in spite of the great strength of these piles the author has seen jobs where it was necessary to drill pilot holes into compacted sand ahead of steel H piles in order to obtain the specified penetration. By weld splicing sections together, lengths in excess of 200 ft have been driven [8].

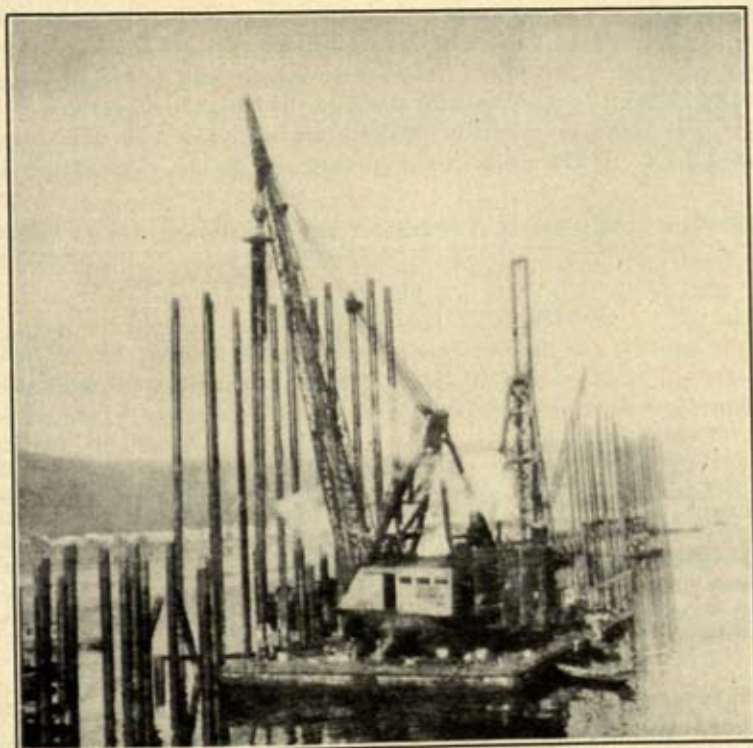


FIG. 14-12. Barge-mounted rig driving 24-in. pipe piles up to 220 ft long. (*Construction Methods and Equipment*.)

**The Resistance of Piles to Penetration.** In general, the forces which enable a pile to support a load also cause the pile to resist the efforts made to drive it. The total resistance of a pile to penetration will equal the sum of the forces produced by skin friction and end bearing. The portion of the resistance supplied by either skin friction or end bearing may vary from almost 0 to 100 per cent, depending on the soil more than on the type of pile. A steel H pile driven to refusal in stiff clay should be classified as a skin-friction pile, while the same pile driven through a mud deposit to rest on solid rock should be classified as an end-bearing pile.



Numerous tests have been conducted to determine values for skin friction for various types of piles and soils. A representative value for skin friction can be obtained by determining the total force required to pull a pile up slightly, using hydraulic jacks with calibrated pressure gauges.

The value of the skin friction is a function of the coefficient of friction between the pile and the soil and the pressure of the soil normal to the surface of the pile, or for a soil such as some types of clay the value of the skin friction may be limited to the shearing strength of the soil immediately adjacent to the pile. Consider a concrete pile driven into a soil that produces a normal pressure of 100 psi on the vertical surface of the pile. This is not an unusually high pressure for certain soils such as compacted sand. If the coefficient of friction is 0.25, the value of the skin

TABLE 14-4. APPROXIMATE ALLOWABLE VALUE OF SKIN FRICTION ON PILES\*

Material	Skin friction, psf		
	Approximate depth 20 ft	Approximate depth 60 ft	Approximate depth 100 ft
Soft silt and dense muck .....	50-100	50-120	60-150
Silt (wet but confined) .....	100-200	125-250	150-300
Soft clay .....	200-300	250-350	300-400
Stiff clay .....	300-500	350-550	400-600
Clay and sand mixed .....	300-500	400-600	500-700
Fine sand (wet but confined) .....	300-400	350-500	400-600
Medium sand and small gravel .....	500-700	600-800	600-800

\* Some allowance is made for the effect of using piles in small groups.

friction will be  $0.25 \times 100 \times 144 = 3,600$  psf. Table 14-4 gives representative values of skin friction on piles. The author of the table states that the information is intended as a qualitative guide, not as correct information to be used in any and all cases.

The magnitude of end-bearing pressure can be determined by driving a button-bottom-type pile and leaving the driving casing in place. A second steel pipe, slightly smaller than the driving casing, is lowered onto the concrete button. The force, applied through the second pipe, required to drive the button into the soil is a direct measure of the supporting strength of the soil. This is true because there is no skin friction on the inside pipe.

**Pile Hammers.** The function of a pile hammer is to furnish the energy required to drive a pile. Pile-driving hammers are designated by type and size.

The types commonly used include the following:

1. Drop
2. Single-acting steam
3. Double-acting steam
4. Differential-acting steam
5. Diesel

The size of a drop hammer is designated by its weight, while the size of each of the other hammers is designated by the theoretical energy per blow, expressed in foot-pounds.

For each type of hammer listed the driving energy is supplied by a falling mass, which strikes the top of a pile. The various types are described in the following articles.

**Drop Hammers.** A drop hammer is a heavy metal weight that is lifted by a rope, then released and allowed to fall on top of the pile. The hammer may be released by a trip and fall freely, or it may be released by loosening the friction band on the hoisting drum and permitting the weight of the hammer to unwind the rope from the drum. The latter type of release reduces the effective energy of a hammer because of the friction loss in the drum and rope. Leads are used to hold the pile in position and to guide the movement of the hammer so that it will strike the pile with a solid blow.

Standard drop hammers are made in sizes which vary from about 500 to 3,000 lb. The height of drop or fall most frequently used varies from 5 to 20 ft. When a large energy per blow is required to drive a pile, it is better to use a heavy hammer with a small drop than a light hammer with a large drop.

Drop hammers are suitable for driving piles on remote projects which require only a few piles and for which the time of completion is not an important factor.

Among the advantages of drop hammers are the following:

1. Small investment in equipment
2. Simplicity of operation
3. Ability to vary the energy per blow by varying the height of fall

Among the disadvantages of drop hammers are the following:

1. Slow rate of driving piles
2. Danger of damaging piles by lifting a hammer too high
3. Danger of damaging adjacent buildings as a result of the heavy vibration caused by a hammer
4. Cannot be used directly for underwater driving

**Single-acting Steam Hammers.** A single-acting steam hammer is a freely falling weight, called a ram, which is lifted by steam or compressed air, whose pressure is applied to the underside of a piston that is connected to the ram through a piston rod. When the piston reaches the top of the



stroke, the steam pressure is released and the ram falls freely to strike the top of a pile. The energy supplied by this type hammer is delivered by a heavy weight striking with a low velocity, due to the relatively low fall.

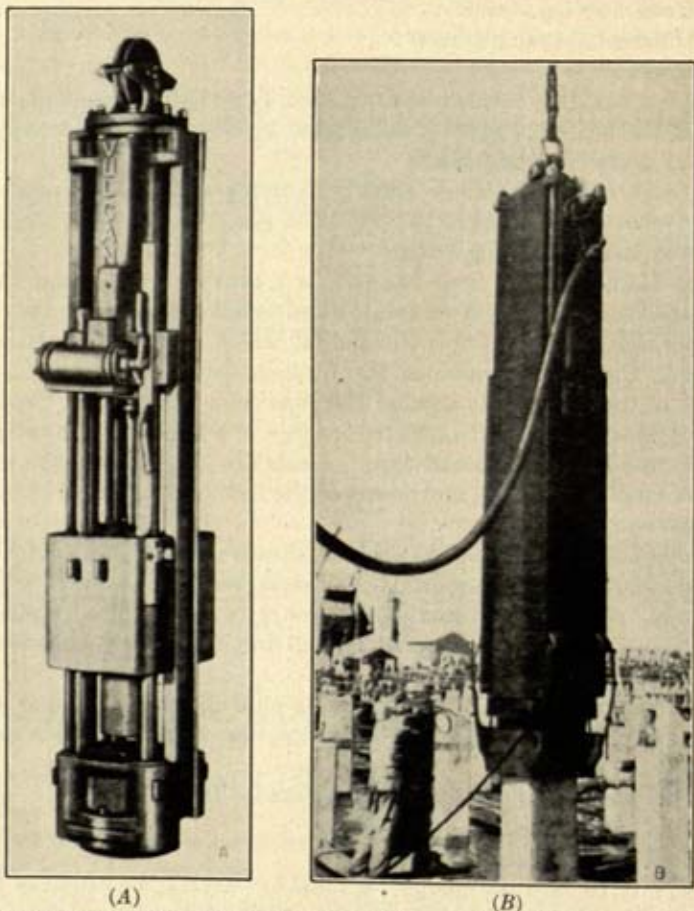


FIG. 14-13. Single-acting steam hammers: (A) open type (Vulcan Iron Works); (B) enclosed type (McKiernan-Terry Corp.).

Whereas a drop hammer may strike 4 to 8 blows per minute, a single-acting steam hammer will strike 50 or more blows per minute when delivering the same energy per blow.

Single-acting steam hammers may be open or enclosed. Figure 14-13 shows one of each type. This hammer is available in sizes varying from a few hundred to more than 30,000 ft-lb of energy per blow. Table 14-5 gives data on several of the more popular sizes of single-acting steam

hammers. The length of the stroke and the energy per blow for this type of hammer may be decreased slightly by reducing the steam pressure below that recommended by the manufacturer. The reduced pressure has the effect of decreasing the height to which the piston will rise before it begins its free fall.

TABLE 14-5. DIMENSIONS AND DATA FOR SINGLE-ACTING STEAM HAMMERS

	Size of hammer					
	3*	2*	1*	0*	OR*	S14†
Rated energy, ft-lb per blow	3,600	7,260	15,000	24,375	30,225	37,500
Blows per min.	80	70	60	50	50	60
Weight of ram, lb.	1,800	3,000	5,000	7,500	9,300	14,000
Normal stroke, ft.	2.0	2.42	3.0	3.25	3.25	2.67
Boiler hp.	18	25	40	60	72	90
Compressed air, cfm.	220	336	565	841		
Steam pressure, psi.	80	80	80	80	100	100
Weight of hammer, lb.	3,700	6,700	9,700	16,250	18,050	31,700

\* Vulcan Iron Works hammers.

† McKiernan-Terry hammer.

Among the advantages of single-acting steam compared with drop hammers are the following:

1. Greater number of blows per minute permits faster driving.
2. Greater frequency of blows reduces the increase in skin friction between blows.
3. Heavier ram falling at lower velocity transmits a greater portion of the energy to driving piles.
4. Reduction in the velocity of the ram decreases the danger of damage to piles during driving.
5. The enclosed types may be used for underwater driving.

Among the disadvantages of single-acting steam compared with drop hammers are the following:

1. Require more investment in equipment such as a steam boiler or an air compressor
2. More complicated, with higher maintenance cost
3. Require more time to set up and take down
4. Require a larger operating crew

**Double-acting Steam Hammers.** In the double-acting steam hammer, steam pressure is applied to the underside of the piston to raise the ram; then during the downward stroke steam is applied to the top side of the piston to increase the energy per blow. Thus, with a given weight ram, it is possible to attain a desired amount of energy per blow with a



shorter stroke than with a single-acting hammer. The number of blows per minute will be approximately twice as great as for a single-acting hammer with the same energy rating.

The lighter ram and higher striking velocity of the double-acting hammer may be advantageous when driving light- to medium-weight piles

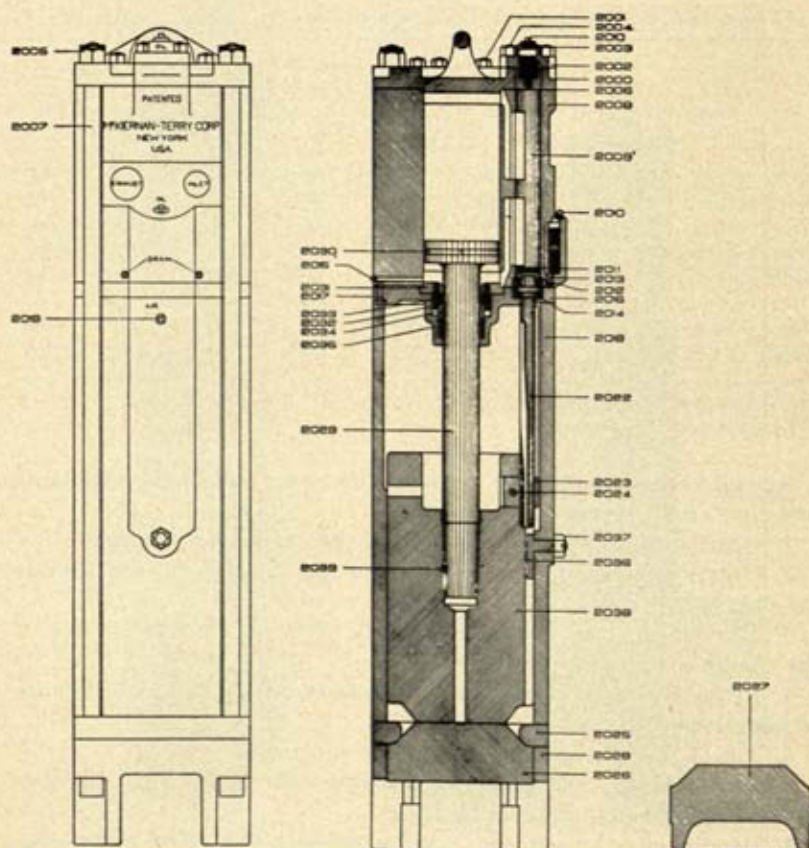


FIG. 14-14. Section through a double-acting pile hammer. (McKiernan-Terry Corp.)

into soils having normal frictional resistance. It is claimed that the high frequency of blows will keep a pile moving downward continuously, thus preventing static skin friction from developing between blows. However, when heavy piles are driven, especially into soils having high frictional resistance, the heavier weight and slower velocity of a single-acting hammer will transmit a greater portion of the rated energy into driving the piles. Figure 14-14 shows the essential parts of a double-acting hammer.

This hammer is fully enclosed by a steel case. Figure 14-15 shows a double-acting hammer driving a steel H pile without leads.

Table 14-6 gives dimensions and other data for several of the more popular sizes of double-acting hammers. As indicated in this table, the energy per blow as well as the number of blows per minute can be modified by varying the steam pressure.



FIG. 14-15. Double-acting steam hammer driving a steel H pile without leads. (McKiernan-Terry Corp.)

Among the advantages of double-acting compared with single-acting hammers are the following.

1. The greater number of blows per minute reduces the time required to drive piles.
2. The greater number of blows per minute reduces the development of static skin friction between blows.
3. Piles can be driven more easily without leads.

Among the disadvantages of double-acting compared with single-acting hammers are the following:



1. The relatively light weight and high velocity of the ram make this type hammer less suitable for use in driving heavy piles into soils having high frictional resistance.

2. The hammer is more complicated.

TABLE 14-6. DIMENSIONS AND DATA FOR MCKIERNAN-TERRY DOUBLE-ACTING STEAM HAMMERS

Size of hammer.....	5	6	7	9-B-3	10-B-3	11-B-3
Weight of ram, lb.....	200	400	800	1,600	3,000	5,000
Normal stroke, in.....	7	8¾	9½	17	19	19
Equivalent stroke, ft.....				5.46	4.36	3.82
Normal blows per min.....	300	275	225	145	105	95
Blows per min:	Rated energy per blow, ft-lb					
275.....		2,500				
230.....		2,160				
200.....		1,680				
225.....			4,150			
195.....			3,720			
170.....			3,280			
145.....				8,750		
140.....				8,100		
135.....				7,500		
130.....				6,800		
105.....					13,100	
100.....					12,000	
95.....					10,900	
90.....					9,550	
95.....						19,150
90.....						18,300
85.....						17,500
80.....						16,700
Boiler hp.....	20	25	35	45	50	60
Compressed air, cfm.....	250	400	450	600	750	900
Steam pressure, psi.....	100	100	100	100	100	100
Weight of hammer, lb.....	1,500	2,900	5,000	7,000	10,850	14,000

**Differential-acting Hammers.** A differential-acting steam hammer is a modified double-acting hammer in that steam pressure is used to lift the ram and to accelerate the ram on the downstroke. As shown in Fig. 14-16, the ram has a large piston which operates in an upper cylinder and a small piston which operates in a lower cylinder. The lifting of the ram is effected by the difference in the pressure forces acting on the two pis-

tons. The number of blows per minute is comparable with that for a double-acting hammer, while the weight and the equivalent free fall of the ram are comparable with those of a single-acting hammer. Thus, it is claimed that this type of hammer has the advantages of the single- and double-acting hammers. It is reported that this hammer will drive a pile in one-half the time required by the same size single-acting hammer and

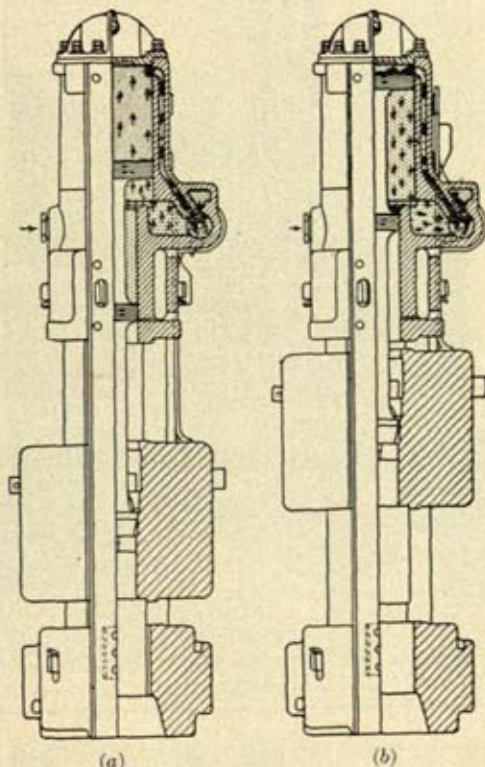


FIG. 14-16. Section through a differential-acting steam hammer: (a) piston in lower position; (b) piston in upper position. (Vulcan Iron Works.)

in doing so will use 25 to 35 per cent less steam. This hammer is available in open or closed types. Figure 14-17 shows an open-type differential-acting hammer operating in hanging leads to drive a timber pile. Table 14-7 gives dimensions and data for these hammers. The values given in the table for rated energy per blow are correct provided the steam pressure is sufficient to produce the indicated normal blows per minute.

**Diesel Hammers.** A diesel pile hammer is a self-contained driving unit which does not require a steam boiler or an air compressor. In this



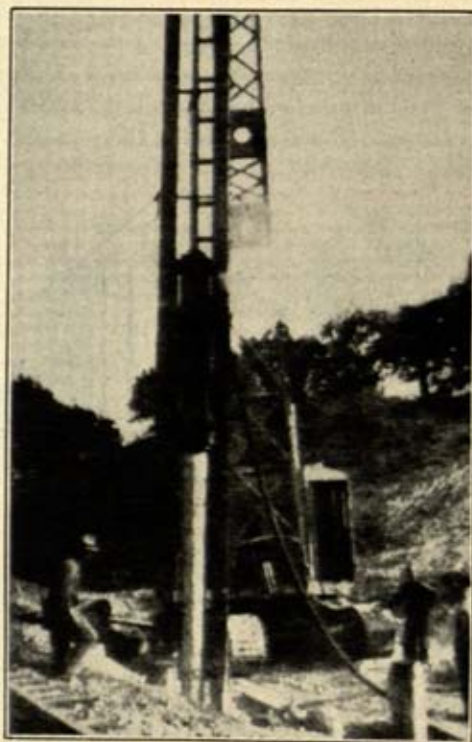


FIG. 14-17. Open-type differential steam hammer driving wood pile. (*Vulcan Iron Works.*)

TABLE 14-7. DIMENSIONS AND DATA FOR SUPER-VULCAN DIFFERENTIAL-ACTING STEAM HAMMERS

Size of hammer:						
Closed type.....	1800	3000	5000	8000	14000	20000
Open type.....	18C	30C	50C	80C	140C	200C
Rated energy per blow, ft-lb.....	3,600*	7,260*	15,100*	24,450*	36,000*	50,200*
Weight of ram, lb.....	1,800	3,000	5,000	8,000	14,000	20,000
Normal stroke, in.....	10½	12½	15½	16½	15½	15½
Equivalent stroke, ft.....	2.0	2.42	3.01	3.05	2.57	2.51
Normal blows per min.....	150	133	120	111	103	98
Boiler hp.....	25	40	60	80	100	120
Compressed air, cfm.....	308	488	880	1,245	1,425	1,746
Steam pressure, psi.....	120	120	120	120	140	142
Weight of hammer, lb.....	4,139	7,036	11,782	17,885	27,984	39,050

\*These values are correct only when the hammer is operating at the specified normal blows per minute.

respect it is simpler than a steam hammer. A complete hammer unit consists of a vertical cylinder, a piston or ram, an anvil, fuel and lubricating-oil tanks, fuel pump, injectors, and mechanical lubricator. Several sizes are available.

After a hammer is placed on the top of a pile, the combined piston and ram are lifted and dropped to start the unit operating. As the ram nears the end of its downward stroke, it engages a cam-roller-operated fuel pump that injects the fuel into the combustion space between the ram and

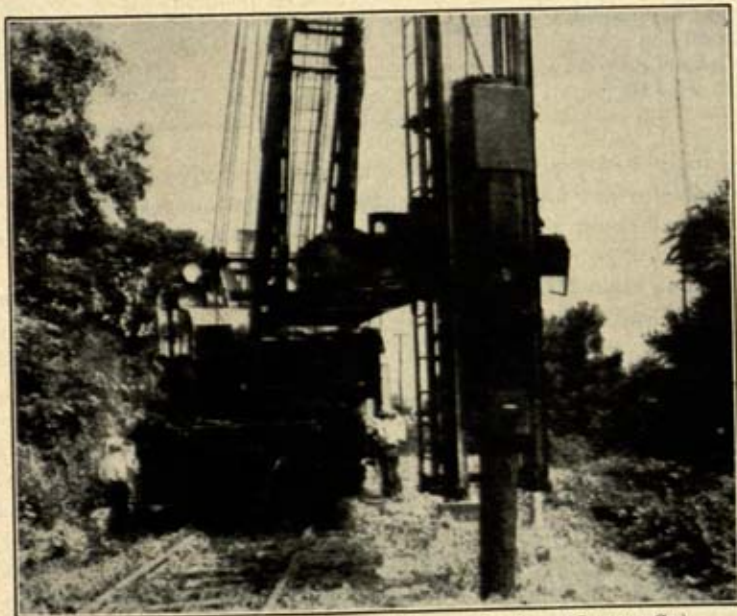


FIG. 14-18. Diesel pile hammer driving wood pile. (Syntron Co.)

the anvil. The continued downstroke of the heavy ram compresses the air and fuel to ignition heat. The resultant explosion drives the pile downward and the ram upward. The energy per blow, which is controlled by the operator, may be varied from zero to full power. Table 14-8 gives dimensions and other data for Syntron diesel pile hammers.

**Jetting Piles.** The use of a water jet to assist in driving piles into sand or fine gravel frequently will speed the driving operation. The water, which is discharged through a nozzle at the lower end of a jet pipe, keeps the soil around a pile in agitation, thereby reducing the resistance due to skin friction. Successful jetting requires a plentiful supply of water at a pressure high enough to loosen the soil and remove it from the hole ahead of the penetration of the pile. Jet pipes commonly used vary in size from



TABLE 14-8. DIMENSIONS AND DATA FOR SYNTRON DIESEL PILE HAMMERS

	Hammer model		
	4300 DPH	9000 DPH	16000 DPH
Total net weight, lb.....	3,600	10,500	12,500
Weight of ram, lb.....	1,460	3,800	5,000
Length of idling stroke, in.....	14	16	22-24
Max length of stroke, in.....	38½	34	48
Output energy per blow, ft-lb.....	4,300	9,000	16,000
Blows per min.....	90-98	100-105	80-84
Fuel consumption, gal per hr.....	0.8	1.75	2.0
Length, over-all, ft.....	10.17	10.5	13.5

2 to 4 in. in diameter, with nozzles varying from ½ to 1½ in. in diameter. The water pressure at the nozzle may vary from approximately 100 to more than 300 psi, with the quantity commonly varying from 300 to 500 gpm, but as high as 1,000 gpm in some instances.

Although some piles have been jetted to final penetration, this is not considered good practice, primarily because it is impossible to determine the safe supporting capacity of a pile so driven. Most specifications require that piles shall be driven the last few feet without the benefit of jetting. The Foundation Code of the City of New York requires a contractor to obtain special permission prior to jetting piles and specifies that piles shall be driven the last 3 ft with a pile hammer.

**Driving Piles below Water.** If it is necessary to drive piles below water, either of two methods may be used. When the driving unit is a drop hammer, an open-type steam hammer, or a diesel hammer, the pile is driven until the top is just above the surface of the water. Then a follower is placed on top of the pile, and the driving is continued through the follower. The follower may be made of wood or steel and must be strong enough to transmit the energy from the hammer to the pile.

When the driving unit is an enclosed steam hammer, the driving may be continued below the surface of the water, without a follower. It is necessary to install an exhaust hose to the surface of the water for the steam. Also, it is necessary to supply about 60 cfm of compressed air to the lower part of the hammer housing to prevent water from flowing into the casing and around the ram. An air pressure of ½ psi for each foot of depth below the surface of water will be satisfactory.

**Pile-driving Formulas.** There are many pile-driving formulas, each of which is intended to give the supporting strength of a pile. The formulas are empirical, with coefficients that have been determined for certain existing or assumed conditions. While each formula may give

dependable values for the conditions under which it was developed, there is no formula that will give dependable values for the supporting strength of piles for all the varying conditions that exist on foundation jobs.

It is not within the scope of this book to analyze the various pile-driving formulas or the theory related to them. For a more comprehensive study of this subject it is suggested that the reader consult the books listed at the end of this chapter [9, 10, 11]. Perhaps the most popular formula in the United States is the *Engineering News* formula. Its popularity seems due primarily to its simplicity rather than its accuracy. For the three types of pile-driving hammers in current use it has the following forms:

For a drop hammer

$$R = \frac{2WH}{S + 1.0} \quad (14-1)$$

For a single-acting steam hammer

$$R = \frac{2WH}{S + 0.1} \quad (14-2)$$

For a double- and differential-acting steam hammer

$$R = \frac{2E}{S + 0.1} \quad (14-3)$$

where  $R$  = safe load on a pile, lb

$W$  = weight of falling mass, lb

$H$  = height of free fall for mass  $W$ , ft

$E$  = total energy of ram at bottom of its downward stroke, ft-lb

$S$  = average penetration per blow for last 5 or 10 blows, in.

The *Engineering News* formula is based on a factor of safety of 6.

Another formula which is useful in analyzing the performance of a hammer in driving a pile is the *Hiley* formula. The author of the formula conducted numerous tests to verify it and to determine the values of coefficients used with it [12]. The formula has two forms, based on the type of hammer with which it is used:

For drop, single-acting steam, and diesel hammers

$$U = \frac{eWh}{S + \frac{1}{2}(C_1 + C_2 + C_3)} \cdot \frac{W + k^2P}{W + P} \quad (14-4)$$

For double-acting and differential-acting steam hammers

$$U = \frac{12eE}{S + \frac{1}{2}(C_1 + C_2 + C_3)} \cdot \frac{W + k^2P}{W + P} \quad (14-5)$$



where  $U$  = ultimate supporting capacity of a pile, lb

$W$  = weight of falling mass, lb

$P$  = weight of pile, including driving cap, head, anvil, or other driving accessories resting directly on pile, lb

$E$  = total theoretical energy of ram at bottom of its downward stroke, ft-lb

$h$  = height of free fall for mass  $W$ , in.

$S$  = average set or penetration per blow for last 5 or 10 blows, in.

$e$  = efficiency of hammer, equal to actual energy divided by rated energy per blow. The values of  $e$  for different hammers are as follows:

= 1.00 for drop hammers released by triggers

= 0.50 to 0.75 for drop hammers lifted by ropes and winches

= 0.75 to 0.90 for single-acting steam hammers

= 0.65 to 0.90 for double-acting steam hammers

= 0.75 to 0.85 for differential-acting steam hammers

= 0.90 to 1.00 for diesel hammers

$k$  = coefficient of restitution

= 0.55 for steel hammer on steel pile, with no cushion

= 0.50 for well-compacted cushion in driving pipe piles

= 0.50 for double-acting steam hammer striking on steel anvil and driving steel piles or precast concrete piles

= 0.40 for ram of double-acting steam hammer striking steel anvil and driving wood piles

= 0.40 for medium-compacted wood cushion in driving steel or pipe piles

= 0.40 for ram of single-acting steam hammer or drop hammer striking directly on head of precast concrete pile

= 0.25 for ram of single-acting steam hammer or drop hammer striking on well-conditioned wood cap in driving precast concrete piles or directly on wood-pile heads

= 0.0 for badly broomed wood piles

$C_1$  = temporary compression of pile head and cap, in.

$C_2$  = temporary compression of pile, in.

$C_3$  = temporary compression, or quake of ground, for average cases where pile is driven into penetrable ground, in.

The unit stresses associated with a pile are as follows:

$$p_1 = \frac{U}{\text{area of pile head}} \quad (14-6)$$

where  $p_1$  = stress in driving cushion or on pile head if no cushion is used, psi

$$p_2 = \frac{U}{\text{area of pile}} \quad (14-7)$$

where  $p_2$  = stress on average cross section area of a pile, including the mandrel if one is used, psi

$$p_3 = \frac{U}{\text{area of tip}} \quad (14-8)$$

or 
$$p_3 = \frac{U}{\text{gross area at ground surface}} \quad (14-9)$$

where  $p_3$  = stress under tip of constant cross section, and on gross area of a pile at ground surface for a tapered friction pile, psi

Representative values of  $C_1$ ,  $C_2$ , and  $C_3$  are given in Tables 14-9, 14-10, and 14-11, respectively.

TABLE 14-9. TEMPORARY COMPRESSION ALLOWANCE  $C_1$  FOR PILE AND CAP

Type of head and cap	Resistance to driving			
	Low, $p_1 = 500$ psi, in.	Medium, $p_1 = 1,000$ psi, in.	High, $p_1 = 1,500$ psi, in.	Very high, $p_1 = 2,000$ psi, in.
Head for timber pile.....	0.05	0.10	0.15	0.20
For precast concrete pile:				
Cap on head.....	0.05*	0.10*	0.15*	0.20*
3-4-in. packing inside cap.....	0.07*	0.15*	0.22*	0.30*
½-1-in. mat pad only on head of precast concrete pile.....	0.025	0.05	0.075	0.10
Steel-covered cap, containing wood packing for steel pile or pipe....	0.04	0.08	0.12	0.16
Head of steel pile or pipe.....	0.0	0.0	0.0	0.0

\* These values should be added if wood packing is used inside the head and cap.

TABLE 14-10. TEMPORARY COMPRESSION VALUES OF  $C_2$  FOR PILES

Resistance to driving.....	Low	Medium	High	Very high
$p_2$ for wood or concrete.....	500 psi	1,000 psi	1,500 psi	2,000 psi
$p_2$ for steel.....	7,500 psi	15,000 psi	22,500 psi	30,000 psi

Values of  $C_2$ , in.

Type of pile:				
Timber.....	0.004L*	0.008L*	0.012L*	0.016L*
Precast concrete.....	0.002L	0.004L	0.006L	0.008L
Sheet steel, pipe, and H pile....	0.003L	0.006L	0.009L	0.012L

\*  $L$  is the length from top of pile to center of driving resistance, ft.

**Selecting a Pile Hammer.** Selecting the most suitable pile hammer for a given project involves a study of several factors, such as the size and type of piles, the number of piles, the character of the soil, the location of



TABLE 14-11. TEMPORARY COMPRESSION, OR QUAKE-OF-GROUND, ALLOWANCE  $C_2$ 

Resistance to driving . . . . .	Low	Medium	High	Very high
Value of $p_1$ . . . . .	500 psi	1,000 psi	1,500 psi	2,000 psi

Values of  $C_2$ , in.

For piles of constant cross section	0.0-0.10	0.10-0.20	0.15-0.30	0.05-0.20
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the project, the topography of the site, the type of rig available, whether driving will be done on land or in water, etc. A pile-driving contractor usually is concerned with selecting the hammer that will drive the piles for a project at the lowest practical cost. As most contractors must limit their ownership to a few representative sizes and types of hammers, a selection should be made from those hammers already owned unless conditions are such that it is economical or necessary to secure an additional size or type. Naturally, more consideration should be given to the selection of a hammer for a project that requires several hundred piles than for a project that requires only a few piles.

As previously stated, the function of a pile hammer is to furnish the energy required to drive a pile. This energy is supplied by a weight which is raised and permitted to drop on top of a pile, under the effect of gravity alone or with steam acting during the downward stroke. The theoretical energy per blow will equal the product of the weight times the equivalent free fall. Since some of this energy is lost in friction as the weight travels downward, the net energy per blow will be less than the theoretical energy, the actual amount depending on the efficiency of the particular hammer. As indicated on page 352, the efficiencies of pile hammers vary from 50 to 100 per cent.

**Loss in Energy Due to Impact.** When a mass in motion, striking a mass at rest, transfers a portion of its kinetic energy to the mass at rest, there will always be some loss in energy due to impact. The proportion of energy dissipated through impact varies with the ratio of the weight of the moving mass to the weight of the stationary mass. The energy loss due to impact  $I$  is given by the following formulas:

For drop, single-acting steam, and diesel hammers

$$I = eWhP \frac{1 - k^2}{W + P} \quad (14-10)$$

For double-acting and differential-acting steam hammers

$$I = eEP \frac{1 - k^2}{W + P} \quad (14-11)$$

The kinetic energy of a hammer at the bottom of its stroke is  $WV^2/2g$ , where  $W$  is the weight of the moving mass in pounds,  $V$  is the velocity in feet per second, and  $g$  is the acceleration of gravity in feet per second per second. The energy is expressed in foot-pounds. Although the same amount of energy may be produced by a light weight and a high velocity or with a heavy weight and a low velocity, it is better to use a hammer with a heavy weight and a low velocity because more of the energy will be converted into driving a pile. The effect which varying the ratio of the weight of the ram to the weight of a pile has on the loss of energy due to

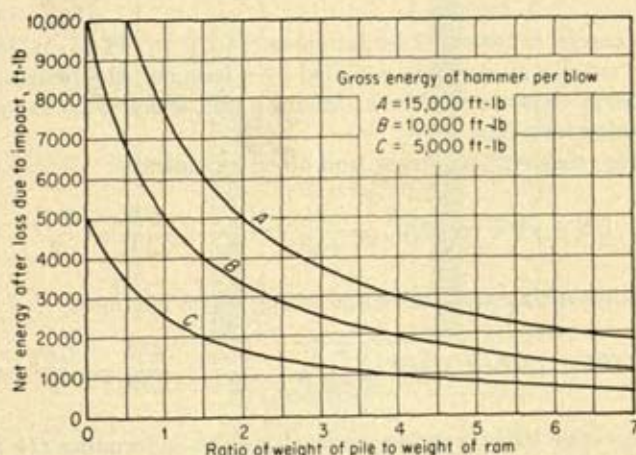


FIG. 14-19. The effect of varying the weight of the ram on the net energy available for driving a pile. (Vulcan Iron Works.)

impact is illustrated in Fig. 14-19. For a ratio of 1 the loss in energy is 50 per cent, whereas for a ratio of  $1/2$  the loss is about 33 per cent. Instead of lifting a 1,000-lb drop hammer 10 ft to obtain 10,000 ft-lb of energy it is better to use a 2,000-lb hammer and lift it 5 ft. The same reasoning applies to steam and diesel hammers.

**Energy Losses Due to Causes Other than Impact.** A. Hiley, the author of formulas (14-4) and (14-5), conducted numerous tests to determine the energy losses due to causes other than impact [12]. He found that the temporary compression of a pile head and cap, a pile, and the ground beneath and adjacent to a pile resulted in some energy losses. His analyses of these losses are primarily responsible for the values of  $C_1$ ,  $C_2$ , and  $C_3$ , given in Tables 14-9, 14-10, and 14-11, respectively. Each of these losses may be expressed by a formula as follows:

$$\text{Energy loss due to temporary compression of a pile head and cap} = \frac{UC_1}{2} \quad (14-12)$$



$$\text{Energy loss due to temporary compression of a pile} = \frac{UC_2}{2}, \text{ or } \frac{U^2 l}{2AK} \quad (14-13)$$

where  $l$  = length from top of a pile to center of driving resistance, in.

$A$  = cross-sectional area of a pile, sq in.

$K$  = modulus of elasticity of material in a pile, psi

$$\text{Energy loss due to temporary compression of soil} = \frac{UC_3}{2} \quad (14-14)$$

If the energy represented by formulas (14-10) to (14-14) is deducted from the net energy per blow supplied by a hammer, the remainder will be the energy that is available for driving a pile, which is given by one of the following formulas:

For drop, single-acting steam, and diesel hammers

$$US = eWh - eWhP \frac{1 - k^2}{W + P} - \frac{UC_1}{2} - \frac{U^2 l}{2AK} - \frac{UC_3}{2} \quad (14-15)$$

For double-acting and differential-acting steam hammers

$$US = 12eE - 12eEP \frac{1 - k^2}{W + P} - \frac{UC_1}{2} - \frac{U^2 l}{2AK} - \frac{UC_3}{2} \quad (14-16)$$

Since the energy  $US$  is in inch-pounds, all weights in formulas (14-15) and (14-16) must be in pounds and all linear dimensions must be in inches.

The extent to which energy losses may vary in driving piles under different conditions is illustrated by Fig. 14-20 [13]. Pile (a), which was a 16-in.-square concrete pile, 51 ft long, weighing 6 tons, was driven with a 4-ton winch-operated drop hammer, with a fall of 3 ft 6 in. This pile was driven into soil having high frictional resistance. Pile (b), which was a composite 14- by 14-in. timber and concrete pile, 51 ft long, weighing 2.75 tons, was driven with a 2-ton freely falling drop hammer, with a fall of 4 ft 0 in. This pile was driven into soil having low frictional resistance.

The information given in Fig. 14-20 and below was obtained from calculations and tests made while the piles were being driven:

Item	Pile (a)	Pile (b)
Weight of pile, tons.....	6	2.75
Weight of hammer, tons.....	4	2
Drop of hammer, ft.....	3.5	4
Final set of pile, per blow, in.....	$1\frac{1}{8}$	3

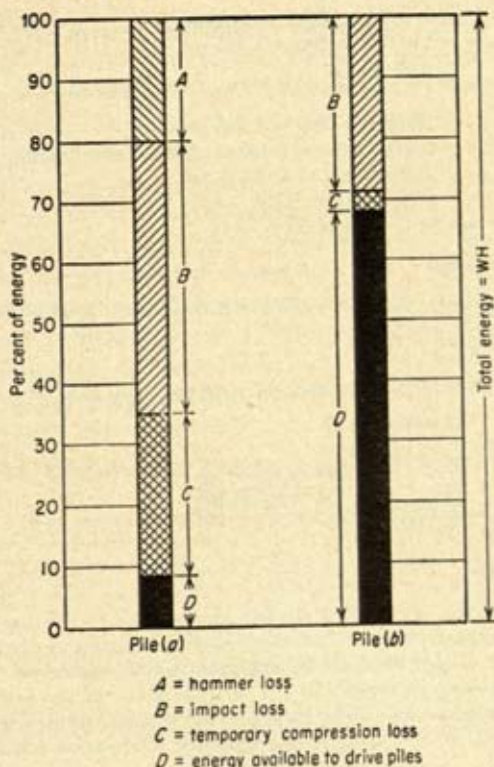


FIG. 14-20. Representative energy losses in driving piles.

**Analysis of Pile-driving Problems.** The examples which follow are intended to illustrate methods which may be used to determine whether a hammer is suitable for driving a given pile:

**EXAMPLE.** Concrete piles, 14 in. square and 50 ft long, are to be driven to full penetration into soil having normal frictional resistance. Tests indicate a pile will safely support a load of 40 tons, with a factor of safety of 3. It is planned to drive the piles with a Vulcan size 1 single-acting steam hammer. Determine the probable energy per blow available for driving a pile and the set per blow at full penetration.

For the specified hammer and piles the following information will apply:

$$W = 5,000 \text{ lb}$$

$$h = 36 \text{ in.}$$

$$e = 0.75$$

$$k = 0.25$$

$$\text{Weight of pile} = \frac{14 \times 14}{144} \times 150 \times 50 = 10,220 \text{ lb}$$

$$\text{Weight of pile cap and head} = 775 \text{ lb}$$

$$\text{Combined weight } P = 10,995 \text{ lb}$$



$$U = 3 \times 40 \times 2,000 = 240,000 \text{ lb}$$

$$p_1 = 240,000 \div 196 = 1,225 \text{ psi}$$

$$C_1 = \frac{0.25 + 0.37}{2} = 0.31 \text{ in. (from Table 14-9)}$$

$$p_2 = 240,000 \div 196 = 1,225 \text{ psi}$$

$$C_2 = 0.0049 \times 33 = 0.162 \text{ in. (from Table 14-10)}$$

$$p_3 = 240,000 \div 196 = 1,225 \text{ psi}$$

$$C_3 = 0.15 \text{ in. (from Table 14-11)}$$

$$\text{Assume } L = \frac{2}{3} \times 50 = 33 \text{ ft}$$

Apply formula (14-15).

$$\text{Net energy delivered by hammer} = eWh = 0.75 \times 5,000 \times 36 = 135,000 \text{ in.-lb}$$

$$\begin{aligned} \text{Impact loss} &= eWhP \frac{1-k^2}{W+P} \\ &= 0.75 \times 5,000 \times 36 \times 10,995 \frac{1-0.0625}{5,000+10,995} = 87,000 \text{ in.-lb} \end{aligned}$$

$$\text{Cap loss} = \frac{UC_1}{2} = \frac{240,000 \times 0.31}{2} = 37,200 \text{ in.-lb}$$

$$\text{Pile loss} = \frac{UC_2}{2} = \frac{U^2 l}{2AK} = \frac{240,000^2 \times 33 \times 12}{2 \times 196 \times 3,000,000} = 19,400 \text{ in.-lb}$$

$$\text{Soil loss} = \frac{UC_3}{2} = \frac{240,000 \times 0.15}{2} = 18,000 \text{ in.-lb}$$

$$\text{Total energy loss} = 171,600 \text{ in.-lb}$$

Since the net energy delivered by the hammer is less than the sum of the probable energy losses, it appears that this hammer will not be capable of driving a pile to full penetration. The set per blow will be zero prior to full penetration.

EXAMPLE. Consider all conditions the same as given in the preceding example, except that a Vulcan size 0 single-acting steam hammer will be used to drive the piles.

For the specified hammer and piles the following information will apply:

$$W = 7,500 \text{ lb}$$

$$h = 39 \text{ in.}$$

Other values will be the same as for preceding example

Apply formula (14-15).

$$\text{Net energy delivered by hammer} = eWh = 0.75 \times 7,500 \times 39 = 219,000 \text{ in.-lb}$$

$$\begin{aligned} \text{Impact loss} &= eWhP \frac{1-k^2}{W+P} \\ &= 0.75 \times 7,500 \times 39 \times 10,995 \frac{1-0.0625}{7,500+10,995} = 122,000 \text{ in.-lb} \end{aligned}$$

$$\text{Cap loss (see preceding example)} = 37,200 \text{ in.-lb}$$

$$\text{Pile loss (see preceding example)} = 19,400 \text{ in.-lb}$$

$$\text{Soil loss (see preceding example)} = 18,000 \text{ in.-lb}$$

$$\text{Total energy loss} = 196,600 \text{ in.-lb}$$

$$\text{Energy available to drive a pile} = 22,400 \text{ in.-lb}$$

$$\text{Net total energy delivered by hammer} = 219,000 \text{ in.-lb}$$

The set of a pile per blow is obtained as follows:

$$US = 22,400 \text{ in.-lb}$$

$$U = 240,000 \text{ lb}$$

$$S = \frac{22,400}{240,000} = 0.0933 \text{ in.}$$

**Selecting a Pile-driving Hammer.** Table 14-12 gives recommended sizes of hammers for different types and sizes of piles and driving resistances. The sizes are indicated by the theoretical foot-pounds of energy delivered per blow. Table 14-13 gives the theoretical energy per blow for several types and sizes of hammer. For each hammer listed the specified theoretical energy per blow is correct provided the hammer is operated at the designated number of strokes per minute.

In general, it is good practice to select the largest hammer that can be used without overstressing or damaging a pile. As previously shown, when a large hammer is used, a greater portion of the energy is effective in driving a pile, which produces a higher operating efficiency. Therefore, the hammer sizes given in Table 14-12 should be considered as the minimum sizes. In some instances hammers as much as 50 per cent larger may be used advantageously.

TABLE 14-12. RECOMMENDED SIZES OF HAMMERS FOR DRIVING VARIOUS TYPES OF PILES

(Size expressed in foot-pounds of energy per blow)

(Size expressed in foot-pounds of energy)

Length of piles, ft	Depth of penetra- tion	Weight of various types of piles, lb per lin ft						
		Steel sheet *			Timber		Concrete	
		20	30	40	30	60	150	400
Driving through ordinary earth, moist clay, and loose gravel, normal frictional resistance								
25	½	2,000	2,000	3,600	3,600	7,000	7,500	15,000
	Full	3,600	3,600	6,000	3,600	7,000	7,500	15,000
50	½	6,000	6,000	7,000	7,000	7,500	15,000	20,000
	Full	7,000	7,000	7,500	7,500	12,000	15,000	20,000
75	½	.....	7,000	7,500	.....	15,000	.....	30,000
	Full	.....	.....	12,000	.....	15,000	.....	30,000
Driving through stiff clay, compacted sand, and gravel, high frictional resistance								
25	½	3,600	3,600	3,600	7,500	7,500	7,500	15,000
	Full	3,600	7,000	7,000	7,500	7,500	12,000	15,000
50	½	7,000	7,500	7,500	12,000	12,000	15,000	25,000
	Full	.....	7,500	7,500	.....	15,000	.....	30,000
75	½	.....	7,500	12,000	.....	15,000	.....	36,000
	Full	.....	.....	15,000	.....	20,000	.....	50,000

\* The indicated energy is based on driving two steel-sheet piles simultaneously. In driving single piles, use approximately two-thirds of the indicated energy.

**Drilled and Underreamed Foundations.** A method of foundation construction that has become popular in recent years is to drill holes deep enough to encounter or penetrate a stable formation that is capable of



TABLE 14-13. DATA ON PILE-DRIVING HAMMERS

Hammer	Hammer size	Weight, lb		Strokes per min	Length of stroke, in.	Theoretical energy, ft-lb per blow
		Complete unit	Ram			
Vulcan, single-acting...	4	1,400	550	80	21	825
	2	6,700	3,000	70	29	7,260
	1	9,600	5,000	60	36	15,000
	0	16,250	7,500	50	39	24,375
	OR	18,050	9,300	50	39	30,225
McKiernan-Terry, single-acting	S3	9,030	3,000	65	36	9,000
	S5	12,460	5,000	60	39	16,250
	S8	18,300	8,000	55	39	26,000
	S10	22,380	10,000	55	39	32,500
	S14	31,700	14,000	60	33	37,500
Vulcan, differential-acting	18C	4,139	1,800	150	10½	3,600
	30C	7,250	3,000	133	12½	7,260
	50C	12,140	5,000	120	15½	15,100
	140C	27,980	14,000	103	15½	36,000
	200C	39,050	20,000	98	15½	50,200
McKiernan-Terry, double-acting	5	1,500	200	300	7	1,000
	7	5,000	800	225	9½	4,150
	9B3	7,000	1,600	145	17	8,750
	10B3	10,850	3,000	105	19	13,100
	11B3	14,000	5,000	95	19	19,150
Union, double-acting...	1½A	9,200	1,500	125	18	8,280
	1A	10,500	1,600	120	18	10,020
	1	10,000	1,850	130	21	13,100
	0A	17,000	5,000	90	21	22,050
	00	21,000	6,000	85	36	54,900

supporting heavy loads. These holes are filled with plain or reinforced concrete and thus serve the same purpose as piles in supporting structures. For many projects, this type of foundation is more satisfactory and less expensive than piles. It is an economical substitute for conventional spread footings.

As illustrated in Fig. 14-21, the holes may be drilled to a uniform diameter, or they may have an enlarged diameter at the bottom to give a greater bearing area. The footing illustrated in Fig. 14-21a is commonly referred to as a drilled footing, and the one illustrated in Fig. 14-21b is referred to as an underreamed footing. The upper portion of the latter is called the shaft, and the enlarged portion is called the underream. Shafts may be drilled to any desired diameter within reason, usually 12 to 96 in. Because of mechanical limitations on drilling equipment, the

maximum diameter of the underream is related to the diameter of the shaft. Underream diameters as great as 144 in. or more have been drilled. Under favorable conditions it is possible to drill holes as deep as 200 ft.

The holes are drilled by a truck-mounted rig, whose essential parts include a power unit, cable drum, boom, rotary table, drill stem, and drill. The shaft is drilled first with a large earth auger or a bucket drill, equipped with cutting blades at the bottom; then the bottom portion of the hole is enlarged with a special drill known as an underreamer.

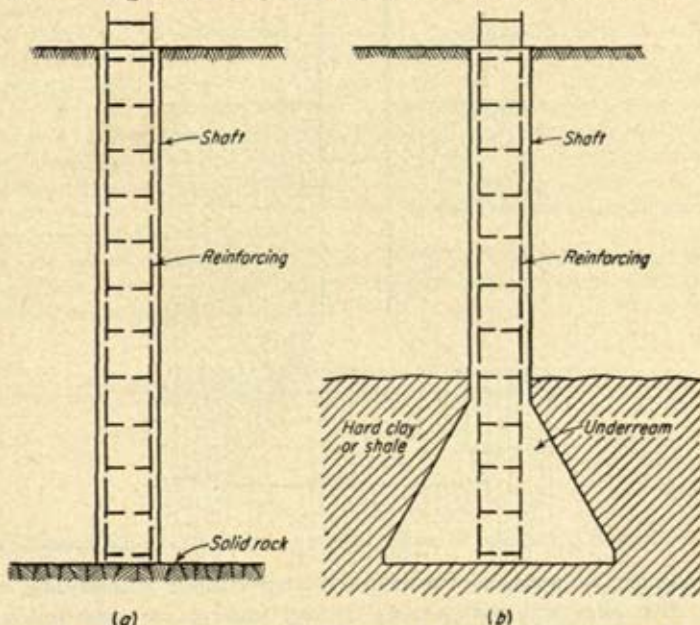


FIG. 14-21. Representative footings: (a) drilled footing; (b) underreamed footing.

Figure 14-22 illustrates a method used to drill these holes through unstable soils, such as mud, sand, or gravel, containing water. If it is possible to do so, the shaft is drilled entirely through the unstable soil; then a temporary steel casing is installed in the hole to eliminate ground water and caving. An alternate method is to add sections to the casing as drilling progresses until the full depth of bad soil is cased off. Then the hole is completed and filled with concrete, and the casing is pulled before the concrete sets.

This type of foundation has been used extensively in areas whose soils are subject to changes in moisture content to considerable depth. By placing the footings below the zone of moisture change the effect of soil movements due to changes in moisture is eliminated.



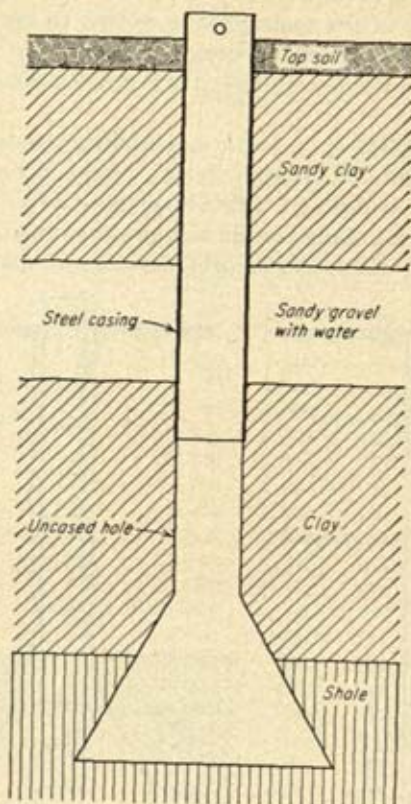


FIG. 14-22. Steel casing used to permit footing to be drilled through unstable soil.

Among the advantages of drilled and underreamed foundations, compared with piles and conventional spread footings, are the following: They

1. Are less expensive for some soils and projects
2. Are easy to vary in length to adjust for soil conditions
3. Permit inspection of soil prior to establishing depth or placing concrete
4. Eliminate damage to adjacent structures due to vibration of the pile hammer
5. Eliminate the use of forms for concrete

#### PROBLEMS

**14-1.** A 14-in.-square concrete pile, 30 ft long, was driven to full penetration by a Vulcan size 1 single-acting steam hammer. The average penetration per blow for the last 10 blows was 0.25 in. Using the *Engineering News* formula, determine the maximum safe load on this pile.

**14-2.** If the pile of Prob. 14-1 was driven with a McKiernan-Terry size 11-B-3 double-acting steam hammer, with the same penetration per blow, determine the maximum safe load on the pile.

**14-3.** A project requires a number of wood piles driven to full penetration. The piles, which will be 50 ft long, 16 in. in diameter at the butt, and 6 in. in diameter at the tip, will be treated with creosote at the rate of 16 lb per cu ft of pile. The soil is sandy clay for the full depth. Select the most suitable size single-acting steam hammer to drive the piles.

**14-4.** Select the most suitable size single-acting steam hammer to drive 18-in.-square concrete piles, 48 ft long, to full penetration into a soil having high frictional resistance.

**14-5.** Select a single-acting steam hammer to drive concrete piles, 16 in. square and 40 ft long, to full penetration into a soil having high frictional resistance. The average total energy required to drive a pile is 3,200,000 ft-lb. Use the information given in Fig. 14-19 to determine the probable energy per blow delivered to a pile by the hammer. What is the probable time, in minutes, required to drive a pile?

**14-6.** Using the information given in the text, select the most suitable double-acting steam hammer to drive the piles for each of the given conditions:

Assume that the timber piles, which are treated with creosote, weigh 55 lb per cu ft and concrete weighs 150 lb per cu ft.

For each pile show the calculated energy per blow, expressed in foot-pounds, for each size hammer selected; then show the name and size of the hammer selected.

The piles are as follows:

Timber, driven into soil with normal frictional resistance

Diameter of pile, in.		Length, ft	Penetration
Butt	Tip		
14	7	30	Full
18	6	50	Full

Concrete, driven into soil with high frictional resistance

Size, in.	Length, ft	Penetration
14 × 14	30	Full
16 × 16	45	Full
18 × 18	60	One-half

**14-7.** Concrete piles, 14 in. square and 40 ft long, are to be driven into a soil having normal frictional resistance. Tests indicate that a pile will safely support a load of 30 tons, with a factor of safety of 3. The piles will be driven with a Vulcan size 0 single-acting steam hammer. Use formula (14-15) to determine the probable net energy available for driving a pile. Determine the probable set per blow at full penetration. The weight of the pile cap and head will be 2,700 lb.



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14-8. Select a suitable single-acting and double-acting steam hammer to drive each of the following concrete piles:

Size, in.	Length, ft	Soil conditions	Penetration
12 × 12	30	Normal friction	Full
14 × 14	30	High friction	Full
16 × 16	40	High friction	Full
18 × 18	50	Normal friction	One-half

For all piles and hammers, assume that  $k = 0.25$ .

The weights of the pile caps and heads will be as follows:

Size hammer, ft-lb	Weight of cap and head, lb
7,000-9,000	1,400
13,000-15,000	1,900
20,000-27,000	2,700

For each pile and hammer selected, determine the amount of energy lost in foot-pounds per blow and the per cent of rated energy lost due to impact of the hammer on the pile. Determine the loss from the curves in Fig. 14-19 and from the formula developed by Hiley.

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## • CHAPTER 15

### PUMPING EQUIPMENT

**Introduction.** Pumps are used extensively on construction projects for such operations as those listed below:

1. Removing water from pits, tunnels, etc.
2. Unwatering cofferdams
3. Furnishing water for jetting and sluicing
4. Furnishing water for many types of utility services
5. Lowering the water table for excavations
6. Foundation grouting

Most projects require the use of one or more water pumps at various stages during the period of construction. Construction pumps frequently are required to perform under severe conditions, such as resulting from variations in the pumping head or from handling water that is muddy, sandy and trashy, or highly corrosive. The rate of pumping may vary several hundred per cent during the period of construction. The most satisfactory solution to the pumping problem may be a single all-purpose pump, or it may be to use several types and sizes of pumps, to permit flexibility in the operations. The proper solution is to select the equipment which will take care of the pumping needs adequately at the lowest total cost, considering the investment in pumping equipment, the cost of operating the pumps, and any losses that will result from possible failure of the pumps to operate satisfactorily.

For some projects a pump may be the most critical item of construction equipment. In constructing a multimillion-dollar concrete and earth-fill dam, a contractor used a single centrifugal pump to supply water from a nearby stream. The water was used to wash all concrete aggregate, for mixing and curing the concrete, and for moisture in the earth-fill dam. When the pump developed mechanical trouble and the rate of pumping dropped below the job requirements for several days, progress on the project suffered a loss of approximately 25 per cent. With the fixed costs exceeding \$4,000 per day the loss due to the partial failure of the pump exceeded \$1,000 per day.

Among the factors that should be considered in selecting construction pumps are the following:

1. Dependability
2. Availability of parts for making repairs



3. Simplicity to permit easy repairs
4. Economical installation and operation

**Classification of Pumps.** The pumps most commonly used on construction projects may be classified as:

1. Displacement
  - a. Reciprocating
  - b. Diaphragm
2. Centrifugal
  - a. Conventional
  - b. Self-priming
  - c. Air-operated

**Reciprocating Pumps.** A reciprocating pump operates as the result of the movement of a piston inside a cylinder. When the piston is moved in one direction, the water ahead of the piston is forced out of the cylinder. At the same time additional water is drawn into the cylinder behind the piston. Regardless of the direction of movement of the piston, water is forced out of one end and drawn into the other end of the cylinder. This is classified as a double-acting pump. If water is pumped during a piston movement in one direction only, the pump is classified as single-acting. If a pump contains more than one cylinder, mounted side by side, it is classified as a duplex for two cylinders, triplex for three cylinders, etc. Thus a pump might be classified as duplex double-acting, duplex single-acting, etc.

The volume of water pumped in one stroke will equal the area of the cylinder times the length of the stroke, less a small deduction for slippage through the valves or past the piston, usually about 3 to 5 per cent. If this volume is expressed in cubic inches, it may be converted to gallons by dividing by 231, the number of cubic inches in a gallon. The volume pumped in gpm by a simplex double-acting pump will be

$$Q = c \frac{\pi d^2 l n}{4 \times 231} \quad (15-1)$$

where  $Q$  = capacity of a pump, gpm

$c$  = 1 - slip allowance; varies from 0.95 to 0.97

$d$  = diameter of cylinder, in.

$l$  = length of stroke, in.

$n$  = no. strokes per min (NOTE: The movement of the piston in either direction is a stroke)

The volume pumped per minute by a multiplex double-acting pump will be

$$Q = Nc \frac{\pi d^2 l n}{4 \times 231} \quad (15-2)$$

where  $N$  = no. cylinders in pump.

The energy required to operate a pump will be

$$W = \frac{wQh}{e}$$

where  $W$  = energy, ft-lb per min

$w$  = weight of 1 gal of water, lb

$h$  = total pumping head, ft, including friction loss in pipe

$e$  = efficiency of pump, expressed decimally

The horsepower required by the pump will be

$$P = \frac{W}{33,000e} = \frac{wQh}{33,000e} \quad (15-3)$$

where  $P$  = power, hp

33,000 = ft-lb of energy per min for 1 hp

**EXAMPLE.** How many gallons of fresh water will be pumped per minute by a duplex double-acting pump, size 6 by 12 in., driven by a crankshaft making 90 rpm? If the total head is 160 ft and the efficiency of the pump is 60 per cent, what is the minimum horsepower required to operate the pump? The weight of water is 8.34 lb per gal.

**Solution.** Assume a water slippage of 4 per cent. Applying formula (15-2) the rate of pumping will be

$$\begin{aligned} Q &= Nc \frac{\pi d^2 l n}{924} \\ &= \frac{2 \times 0.96 \times \pi \times 36 \times 12 \times 180}{924} = 518 \text{ gpm} \end{aligned}$$

Applying formula (15-3), the power required by the pump will be

$$\begin{aligned} P &= \frac{wQh}{33,000e} \\ &= \frac{8.34 \times 518 \times 160}{33,000 \times 0.60} = 34.9 \text{ hp} \end{aligned}$$

The capacity of a reciprocating pump depends essentially on the speed at which the pump is operated and is independent of the head. The maximum head against which a pump will deliver water depends on the strength of the component parts of the pump and the power available to operate the pump. The capacity of this type pump may be varied considerably by varying the speed of the pump.

Because the flow of water from each cylinder of a reciprocating pump stops and starts every time the direction of piston travel is reversed, a characteristic of this type pump is to deliver water with pulsations. The amplitude of the pulsations may be reduced by using more cylinders and by installing an air chamber on the discharge side of a pump.



Among the advantages of reciprocating pumps are the following:

1. They are able to pump at a uniform rate against varying heads.
2. Their capacity can be increased by increasing the speed.
3. They have reasonably high efficiency regardless of the head and speed.
4. They are usually self-priming.

Among the disadvantages of reciprocating pumps are the following:

1. Heavy weight and large size for given capacity
2. Possibility of valve trouble, especially in pumping water containing trash
3. Pulsating flow of water
4. Danger of damaging a pump in operating against a high head

**Diaphragm Pumps.** The principle under which a diaphragm pump operates is illustrated in Fig. 15-1. The central portion of the flexible

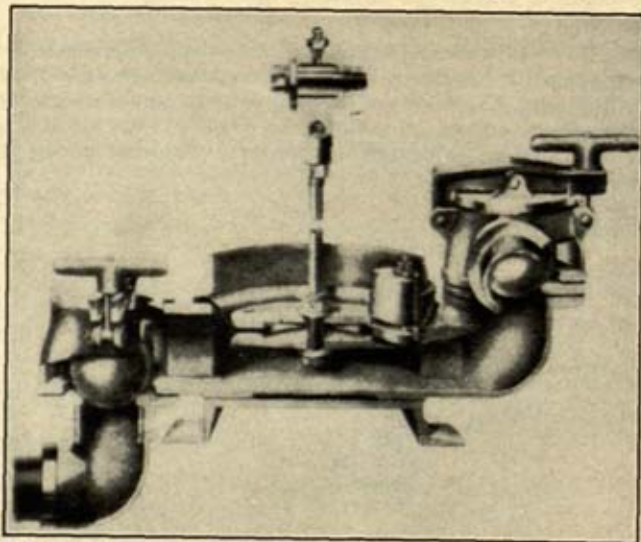


Fig. 15-1. Section through a diaphragm pump. (Marlow Pumps.)

diaphragm is alternately raised and lowered by the pump rod, which is connected to a walking beam. This action draws water into and discharges it from the pump. Because this type pump will handle clear water or water containing large quantities of mud, sand, sludge, and trash, it is popular as a construction pump. It is suitable for use on jobs where the quantity of water varies considerably, as the loss of prime during low flow does not prevent it from automatically repriming when the quantity of water increases. The accessible diaphragm may be replaced easily.

The Contractors Pump Standards of the AGC specifies that diaphragm pumps shall be manufactured in the size and capacity ratings given in Table 15-1.

TABLE 15-1. SIZES AND CAPACITY RATINGS FOR CLOSED DIAPHRAGM PUMPS

Size	Capacity, gpm, for suction lift		Nominal engine hp
	10 ft	20 ft	
3-in. single.....	3 M*	1½ M	2½-3
4-in. single.....	6 M	3 M	3-4
4-in. double.....	9 M	5 M	5-9

\* M is 1,000.

**Centrifugal Pumps.** A centrifugal pump contains a rotating element, called an impeller, which imparts to water passing through the pump a velocity sufficiently great to cause it to flow from the pump even against considerable pressure. A mass of water may possess energy due to its height above a given datum or due to its velocity. The former is potential, while the latter is kinetic, energy. One type of energy can be converted into the other under favorable conditions. The kinetic energy imparted to a particle of water as it passes through the impeller is sufficient to cause the particles to rise to some determinable height.

The principle of the centrifugal pump may be illustrated by considering a drop of water at rest at a height  $h$  above a surface. If the drop of water is permitted to fall freely, it will strike the surface with a velocity given by the formula

$$V = \sqrt{2gh} \quad (15-4)$$

where  $V$  = velocity, fps

$g$  = acceleration of gravity, equal to 32.2 ft per sec per sec at sea level

$h$  = height of fall, ft.

If the drop falls 100 ft, the velocity will be 80.4 fps. If the same drop is given an upward velocity of 80.4 fps, it will rise 100 ft. These values assume no loss in energy due to friction through air. It is the function of the centrifugal pump to give the water the necessary velocity as it leaves the impeller. If the speed of the pump is doubled, the velocity of the water will be increased from 80.4 to 160.8 fps, neglecting any increase in friction losses. With this velocity the water can be pumped to a height given by the formula

$$h = \frac{V^2}{2g} = \frac{160.8^2}{64.4} = 400 \text{ ft}$$



This indicates that if a centrifugal pump is pumping water against a total head of 100 ft, the same quantity of water can be pumped against a total head of 400 ft by doubling the speed of the impeller. In actual practice the maximum possible head for the increased speed will be less than 400 ft because of increases in losses in the pump due to friction. These results illustrate the effect which increasing the speed or the diameter of an impeller has on the performance of a centrifugal pump.

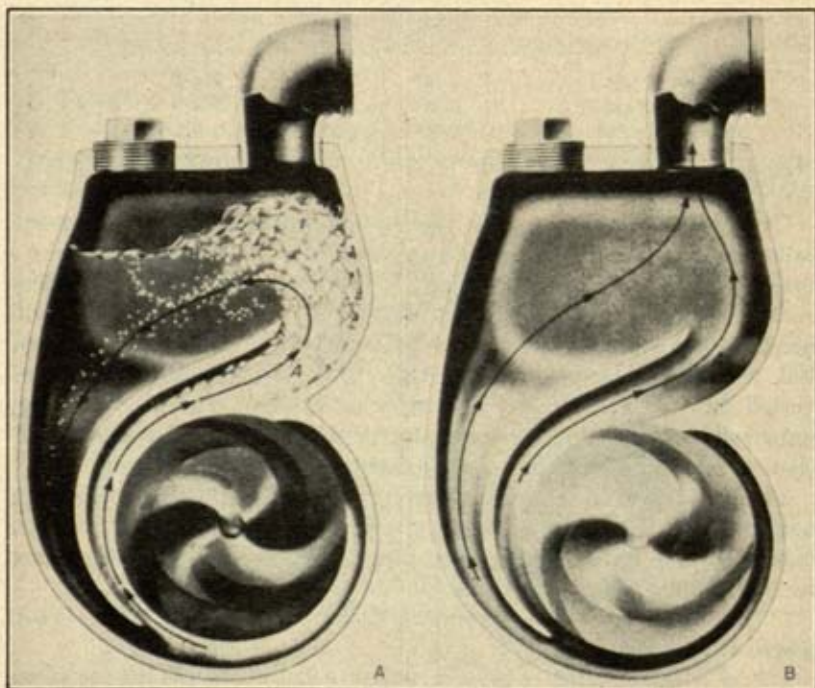


FIG. 15-2. Section through a self-priming centrifugal pump: (A) priming action; (B) pumping action. (The Gorman-Rupp Co.)

A centrifugal pump may be equipped with an open or an enclosed impeller. Although an enclosed impeller usually has higher efficiency, it will not handle water containing trash as well as an open impeller.

The power required to operate a centrifugal pump is given by formula (15-3). The efficiencies of these pumps may be as high as 75 per cent.

**Self-priming Centrifugal Pumps.** The centrifugal pumps most commonly installed in water and sewage plants are set below the level of water on the suction side because they are not self-priming. However, on construction projects, pumps frequently must be set up above the surface of the water to be pumped. Consequently, self-priming centrifugal pumps

are more suitable than the conventional types for use on construction projects. The operation of a self-priming pump is illustrated in Fig. 15-2. A check valve on the suction side of the pump permits the chamber to be filled with water prior to starting the pump. When the pump is started, the water in the chamber produces a seal which enables the pump to draw air from the suction pipe. The air and water flow through channel *A* into the chamber, where the air escapes through the discharge, and the water flows down through channel *B* to the impeller. This action continues until all the air is exhausted from the suction line and water enters the pump. When a pump is stopped, it will retain its charge of priming water indefinitely. Such a pump is self-priming to heights in excess of 25 ft when in good mechanical condition.

#### **Air-operated Centrifugal-type Sump Pumps.**

Air-operated centrifugal pumps are very useful in tunnels, foundation pits, trenches, and similar places, as they are designed to handle clear or dirty water, oil, sewage, or moderately heavy sludge. Figure 15-3 illustrates such a pump in operation on a typical job. This pump, which weighs about 56 lb, has a capacity of 185 gpm against a total head of 40 ft when supplied with compressed air at a pressure of 90 psi.

Other types and models have capacities in excess of 200 gpm for total heads of 100 ft or more. Figure 15-4 shows typical performance curves for an air-operated centrifugal pump.

**Multistage Centrifugal Pumps.** If a centrifugal pump has a single impeller, it is described as single-stage, whereas if there are two or more impellers and the water discharged from one impeller flows into the suction of another, it is described as a multistage pump. Multistage pumps are especially suitable for pumping against high heads or pressures, as each stage imparts an additional pressure to the water. Pumps of this type are used frequently to supply water for jetting, where the pressure may run as high as several hundred psi.

**Performance of Centrifugal Pumps.** The pump manufacturers will furnish sets of curves showing the performance of their pumps under different operating conditions. A set of curves for a given pump will



FIG. 15-3. Single-stage air-operated centrifugal pump. (Ingersoll-Rand Co.)



show the variations in capacity, efficiency, and horsepower for different pumping heads. These curves can be very helpful in selecting the pump that is most suitable for a given pumping condition. Figure 15-5 illustrates a set of performance curves for a 10-in. centrifugal pump. For a total head of 60 ft the capacity will be 1,200 gpm, the efficiency 52 per cent, and the required power 35 bhp. If the total head is reduced to 50 ft and the dynamic suction lift does not exceed 23 ft, the capacity will

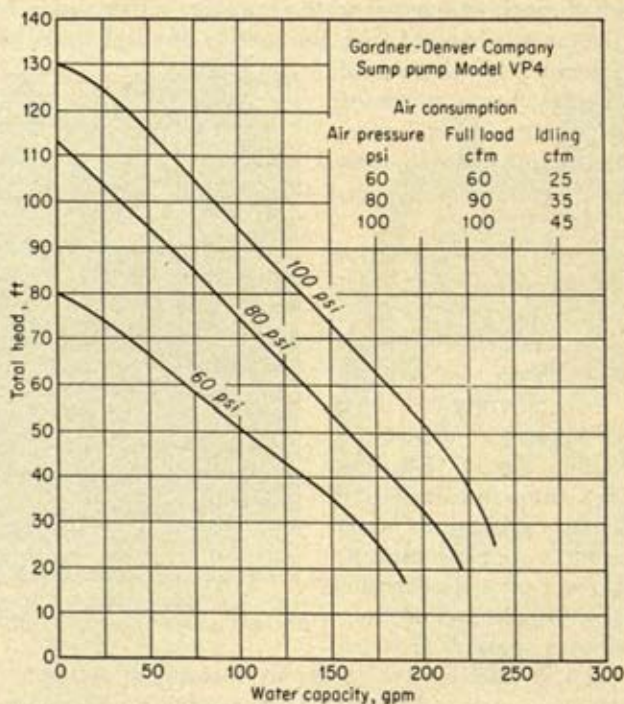


FIG. 15-4. Performance curves for air-operated centrifugal pump. (Gardner-Denver Co.)

be 1,830 gpm, the efficiency 55 per cent, and the required power 44 bhp. This pump will not deliver any water against a total head in excess of 66 ft, which is called the shutoff head.

Since a construction pump frequently is operated under varying heads, it is desirable to select a pump with relatively flat head-capacity and horsepower curves, even though efficiency must be sacrificed in order to obtain these conditions. A pump with a flat horsepower demand permits the use of an engine or an electric motor that will provide adequate power over a wide pumping range, without a substantial surplus or deficiency, regardless of the head.

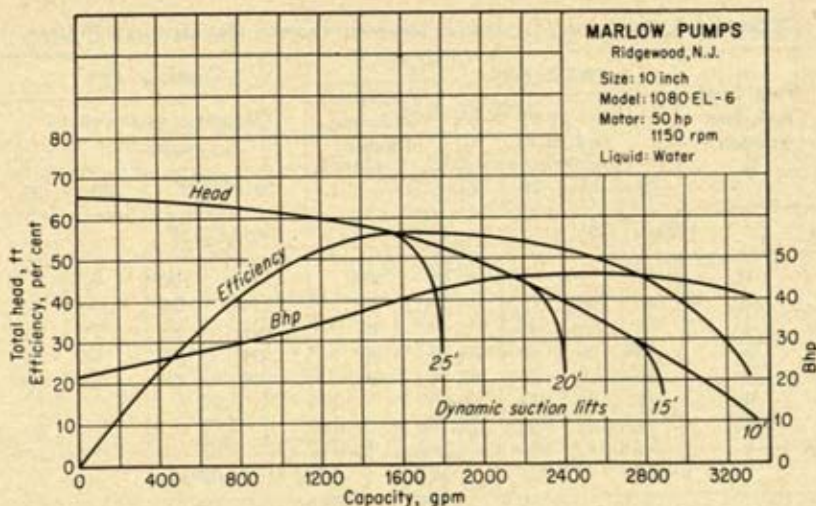


FIG. 15-5. Performance curves for centrifugal pump. (Marlow Pumps.)

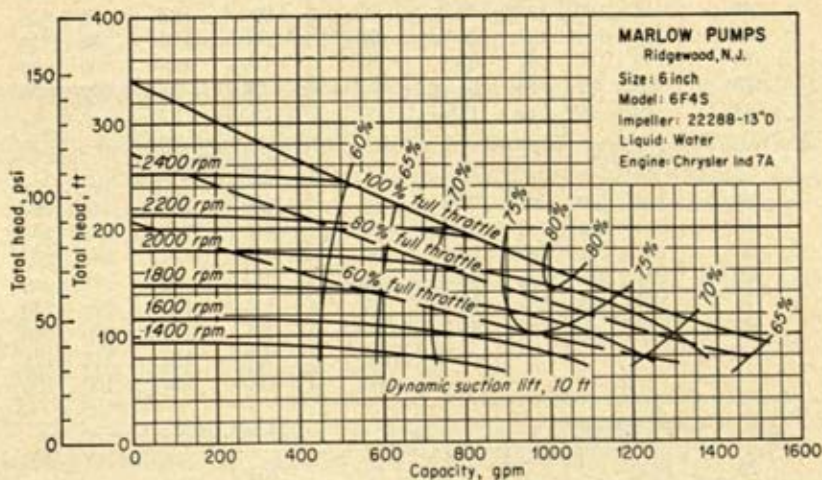


FIG. 15-6. The effect of varying the speed on the performance of a centrifugal pump. (Marlow Pumps.)

The effect of varying the speed of a centrifugal pump is illustrated by the curves in Fig. 15-6.

**Capacity Tables for Self-priming Centrifugal Pumps.** The Contractors Pump Bureau of the AGC publishes pump standards for several types of pumps, including self-priming centrifugal. The standard capacities, which were approved Mar. 1, 1954, are given in Table 15-2.



TABLE 15-2. STANDARD CAPACITIES FOR SELF-PRIMING CENTRIFUGAL PUMPS\*

Total head including friction, ft	Capacity, gpm				Total head including friction, ft	Capacity, gpm			
	Height of pump above water, ft					Height of pump above water, ft			
	10	15	20	25		10	15	20	25
Model 4-M					Model 7-M				
15	67				25	115	100	85	
20	66	59			30	114	98	84	65
25	65	58	47		40	107	92	79	63
30	63	56	47	33	50	94	78	68	57
40	54	49	44	32	60	68	55	48	43
50	37	35	34	29					
55	25	25	25	25					
Model 10-M					Model 15-M				
25	166				30	250	210	170	130
30	164	145	115	80	40	230	200	165	128
40	157	140	113	79	50	200	182	155	125
50	145	130	107	75	60	160	152	138	115
60	122	110	97	68	70	110	105	100	90
70	85	82	75	53	75	75	73	72	70
80	20	20	19	18					
Model 20-M					Model 30-M				
30	333	280	235	165	30	500	435	350	250
40	310	268	230	162	40	495	430	345	250
50	275	245	220	155	50	475	415	340	245
60	220	210	195	143	60	450	400	325	240
70	160	156	155	125	70	415	370	300	230
80	90	90	90	88	80	355	325	270	210
					90	250	240	215	175
					100	100	100	100	100
Model 40-M					Model 90-M				
25	665	...	...	...	25	1,500			
30	660	575	475	355	30	1,480	1,280	1,050	790
40	645	565	465	350	40	1,430	1,230	1,020	780
50	620	545	455	345	50	1,350	1,160	970	735
60	585	510	435	335	60	1,225	1,050	900	690
70	535	475	410	315	70	1,050	900	775	610
80	465	410	365	280	80	800	680	600	490
90	375	325	300	220	90	450	400	365	300
100	250	215	195	145	100	100	100	100	100
110	65	60	50	40					

\* Courtesy Contractors Pump Bureau of the AGC.

TABLE 15-2. STANDARD CAPACITIES FOR SELF-PRIMING CENTRIFUGAL PUMPS\*  
(Continued)

Total head including friction, ft	Capacity, gpm			
	Height of pump above water, ft			
	10	15	20	25
Model 125-M				
25	2,100	1,850	1,570	
30	2,060	1,820	1,560	1,200
40	1,960	1,740	1,520	1,170
50	1,800	1,620	1,450	1,140
60	1,640	1,500	1,360	1,090
70	1,460	1,340	1,250	1,015
80	1,250	1,170	1,110	950
90	1,020	980	940	840
100	800	760	710	680
110	570	540	500	470

\* Courtesy Contractors Pump Bureau of the AGC.

**Loss of Head Due to Friction in Pipe.** Table 15-3 gives the nominal loss of head due to water flowing through new steel pipe. The actual losses may differ from the values given in the table because of variations in the diameter of a pipe and in the condition of the inside surface.

The relationship between the head of fresh water in feet and pressure in psi is given by the formula

$$h = 2.31p$$

or

$$p = 0.434h$$

where  $h$  = depth of water or head, ft.

$p$  = pressure at depth  $h$ , psi

Table 15-4 gives the equivalent length of straight steel pipe having the same loss in head due to water friction as fittings and valves.

**Loss of Head Due to Friction in Rubber Hose.** The flexibility of rubber hose makes it a desirable substitute for pipe for use with pumps on many jobs. Such hose may be used on the suction side of a pump if it is constructed with a wire insert to prevent collapse under partial vacuum. Rubber hose is available with end fittings corresponding with those for iron or steel pipe.

Table 15-5 gives the loss in head in feet per 100 ft due to friction caused by water flowing through the hose. The values in the table apply for rubber substitutes.



TABLE 15-3. FRICTION LOSS FOR WATER, IN FEET PER 100 FT. OF CLEAN WROUGHT-IRON OR STEEL PIPE\*

Flow, gpm	Nominal diameter of pipe, in.											
	1	1¼	1½	2	2½	3	4	5	6	8	10	12
5	1.93	0.51										
10	6.86	1.77	0.83	0.25	0.11							
14	12.8	3.28	1.53	0.45	0.19							
20	25.1	6.34	2.94	0.87	0.36	0.13						
24	35.6	8.92	4.14	1.20	0.50	0.17						
30	54.6	13.6	6.26	1.82	0.75	0.26	0.07					
40	.....	23.5	10.79	3.10	1.28	0.44	0.12					
50	.....	36.0	16.4	4.67	1.94	0.66	0.18	0.06				
75	.....	.....	35.8	10.1	4.13	1.39	0.28	0.12				
100	.....	.....	62.2	17.4	8.51	2.39	0.62	0.20	0.03			
120	.....	.....	.....	24.7	10.0	3.37	0.88	0.29	0.12			
150	.....	.....	.....	38.0	15.4	5.14	1.32	0.33	0.17			
170	.....	.....	.....	48.4	19.6	6.53	1.67	0.54	0.22			
200	.....	.....	.....	66.3	26.7	8.90	2.27	0.74	0.30	0.08		
220	.....	.....	.....	.....	32.2	10.7	2.72	0.88	0.36	0.09		
260	.....	.....	.....	.....	44.5	14.7	3.24	1.20	0.49	0.13		
280	.....	.....	.....	.....	51.3	16.9	4.30	1.38	0.56	0.14		
300	.....	.....	.....	.....	.....	19.2	4.89	1.58	0.64	0.16		
340	.....	.....	.....	.....	.....	24.8	6.19	2.00	0.81	0.21		
400	.....	.....	.....	.....	.....	33.9	8.47	2.72	1.09	0.28	0.09	
500	.....	.....	.....	.....	.....	52.5	13.0	4.16	1.66	0.42	0.14	0.06
600	.....	.....	.....	.....	.....	.....	18.6	5.88	2.34	0.60	0.19	0.08
700	.....	.....	.....	.....	.....	.....	25.0	7.93	3.13	0.80	0.26	0.11
800	.....	.....	.....	.....	.....	.....	32.4	10.22	4.03	1.02	0.33	0.14
900	.....	.....	.....	.....	.....	.....	40.8	12.9	5.05	1.27	0.41	0.17
1,000	.....	.....	.....	.....	.....	.....	50.2	15.8	6.17	1.56	0.50	0.21
1,100	.....	.....	.....	.....	.....	.....	.....	19.0	7.41	1.87	0.59	0.25
1,200	.....	.....	.....	.....	.....	.....	.....	22.5	8.76	2.20	0.70	0.30
1,300	.....	.....	.....	.....	.....	.....	.....	.....	10.2	2.56	0.82	0.34
1,400	.....	.....	.....	.....	.....	.....	.....	.....	11.8	2.95	0.94	0.40
1,500	.....	.....	.....	.....	.....	.....	.....	.....	13.5	3.37	1.07	0.45
2,000	.....	.....	.....	.....	.....	.....	.....	.....	23.8	5.86	1.84	0.78
3,000	.....	.....	.....	.....	.....	.....	.....	.....	.....	12.8	4.00	1.68
4,000	.....	.....	.....	.....	.....	.....	.....	.....	.....	22.6	6.99	2.92
5,000	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	10.80	4.47

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**Selecting a Pump.** Prior to selecting a pump for a given job it is necessary to analyze all information and conditions that will affect the selection. The most satisfactory pumping equipment will be the combination of pump and pipe that will provide the required service for the least total cost. The total cost includes the installed and operating cost of the pump and pipe for the period that it will be used, with an appropriate allowance for salvage value at the completion of the project. In order to analyze the cost of pumping water, it is necessary to have certain information, such as the following:

1. Rate at which the water is to be pumped
2. Height of lift from the existing water surface to the point of discharge
3. Pressure head at discharge, if any
4. Variations in water level at suction or discharge
5. Altitude of the project
6. Height of the pump above the surface of water to be pumped
7. Size of pipe to be used, if already determined
8. Number, sizes, and types of fittings and valves in the pipe line

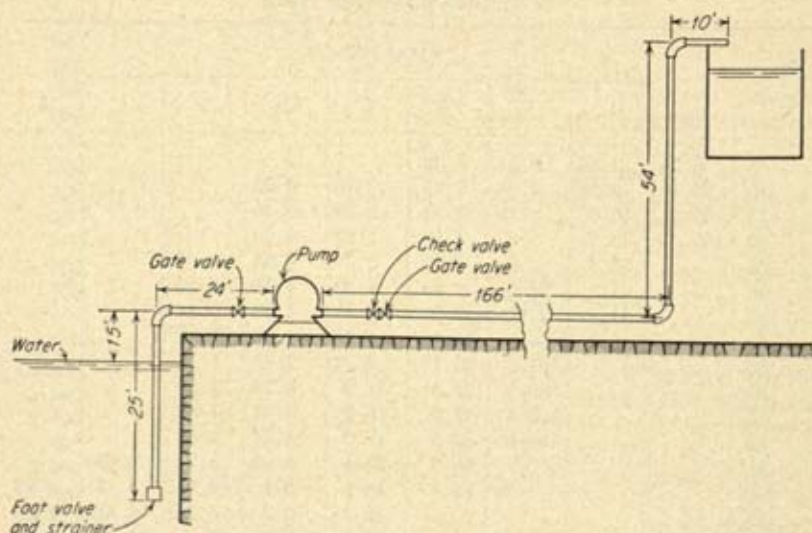


FIG. 15-7. Pump and pipe installation.

The examples which follow are intended to illustrate methods of selecting pumps and pumping systems:

**EXAMPLE.** Select a self-priming centrifugal pump, with a capacity of 600 gpm, for the project illustrated in Fig. 15-7. All pipe, fittings, and valves will be 6 in. with threaded connections.

Use the information given in Table 15-4 to convert the fittings and valves into equivalent lengths of pipe.

Item	Equivalent length of pipe, ft
1 foot valve and strainer	= 76
3 elbows @ 16 ft	= 48
2 gate valves @ 3.5 ft	= 7
1 check valve	= 63
Total	= 194
Add length of pipe	= 279
Total equivalent length of 6-in. pipe	= 473

From Table 15-3 the friction loss per 100 ft of 6-in. pipe will be 2.34 ft. The total head, including lift plus head lost in friction, will be

Lift, 15 + 54	= 69.0 ft
Head lost in friction, 473 ft @ 2.34 ft per 100 ft	= 11.1 ft
Total head	= 80.1 ft

Table 15-2 indicates that a model 90-M pump will deliver the required quantity of water.

Sometimes the problem is to select the pump and pipe line that will permit water to be pumped at the lowest total cost. The following example illustrates a method that may be used to select the most economical pumping system:

**EXAMPLE.** In operating a rock quarry it is necessary to pump 400 gpm of clear water. The pump and pipe line selected will be installed as illustrated in Fig. 15-8.

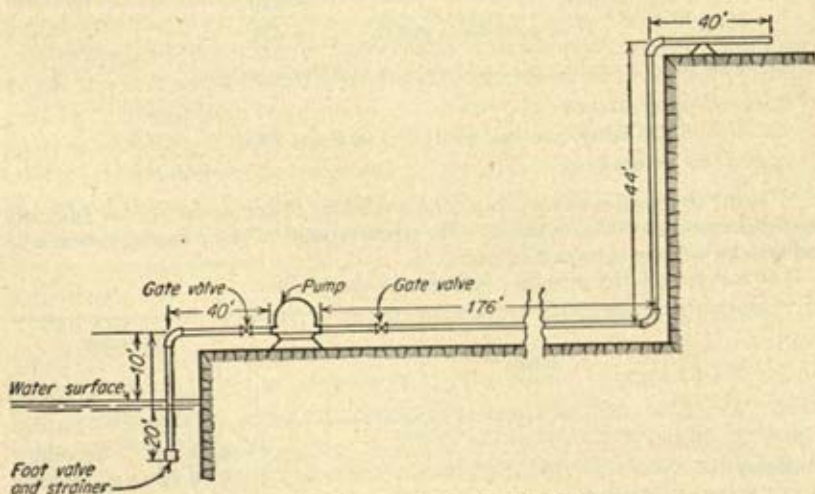


FIG. 15-8. Pump and pipe installation.

It is estimated that the pump will be operated a total of 1,200 hr per year. Compare the economy of using 4-in. and 6-in. steel pipe for the water line. Assume that the pump will have an economic life of 5 years and that the pipe line and fittings will have a life of 10 years. Also, assume that the cost of installing the pipe line will be the same regardless of the size, so that this cost may be disregarded.



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Consider the use of 4-in. pipe.

The total equivalent length of pipe will be

Item	Equivalent length of pipe, ft
1 foot valve and strainer	= 75
3 elbows @ 11 ft	= 33
2 gate valves @ 2.5 ft	= 5
Pipe	= 320
Total equivalent length	= 433

The total head, including lift and head lost in friction, will be

Lift, 10 + 44	= 54.0 ft
Head lost in friction, 433 ft @ 8.47 ft per 100 ft	= 36.7 ft
Total head	= 90.7 ft

A model 90-M self-priming pump, with a capacity of approximately 450 gpm, will be required for this installation.

Consider the use of 6-in. pipe.

The total equivalent length of pipe will be

Item	Equivalent length of pipe, ft
1 foot valve and strainer	= 76
3 elbows @ 16 ft	= 48
2 gate valves @ 3.5 ft	= 7
Pipe	= 320
Total equivalent length	= 451

The total head, including lift and head lost in friction, will be

Lift, 10 + 44	= 54.0 ft
Head lost in friction, 451 ft @ 1.09 ft per 100 ft	= 4.9 ft
Total head	= 58.9 ft

A model 30-M self-priming pump, with a capacity of approximately 450 gpm, will be satisfactory for this installation. The excess capacity of this pumping system is an advantage in favor of using 6-in. pipe.

The cost of each size pipe line, fittings, and valves will be

Item	Size pipe	
	4 in.	6 in.
320-ft pipe line	\$388.00	\$670.00
3 elbows	6.00	12.00
1 foot valve and strainer	12.00	18.00
2 gate valves	84.00	108.00
Total cost	\$490.00	\$808.00
Depreciation cost per yr, based on 10-yr life	49.00	80.80
Depreciation cost per hr, based on 1,200 hr per yr	0.04	0.07

The cost per hour for owning and operating each size pump is given in Appendix A. The combined cost per hour for each size pump and pipe-line system will be

Item	Cost per hr	
	4-in. pipe	6-in. pipe
Pump .....	\$1.31	\$0.87
Pipe, fittings, and valves .....	0.04	0.07
Total cost per hr .....	\$1.35	\$0.94

This analysis shows that the additional cost of the 6-in. pipe is more than offset by the reduction in the cost of the smaller pump.

**Wellpoint Systems.** In excavating below the surface of the ground, it is not uncommon practice to encounter ground water before reaching the bottom of a pit. For pits excavated into sand and gravel, the flow of water will be large if some method is not adopted to remove the water before it enters the pit. While the water may be permitted to flow into sumps located in the pit, then removed by pumps, the presence of such water usually creates a nuisance and interferes with the construction operations. The installation of a wellpoint system along or around the pit may lower the water table below the bottom of the excavation, thus permitting the work to be done under relatively dry conditions.

A wellpoint is a perforated tube enclosed in a screen, which is installed below the surface of the ground to collect water in order that the water may be removed from the ground. The essential parts of a wellpoint are illustrated in Fig. 15-9. The top of a wellpoint is attached to a riser pipe, which extends a short distance above the surface of the ground, where it is connected to a large pipe called a header. The header pipe is connected to the suction of a centrifugal pump. A wellpoint system may include a few or several hundred wellpoints, all connected to one or more headers and pumps.

The principle by which a wellpoint system operates is illustrated in Fig. 15-10. Figure 15-10*a* shows how a single point will lower the surface of the water table in the soil adjacent to the point. Figure 15-10*b* shows how several points, installed reasonably close together, lower the water table over an extended area. A group of wellpoints properly installed along a trench or around a foundation pit will lower the water table below the depth of excavation.

Wellpoints will operate satisfactorily if they are installed in a permeable soil such as sand or gravel. If they are installed in a less permeable soil, such as silt, it may be necessary first to sink a large pipe, say 6 to 10 in. in diameter, for each point, remove the soil from inside the pipe, install a



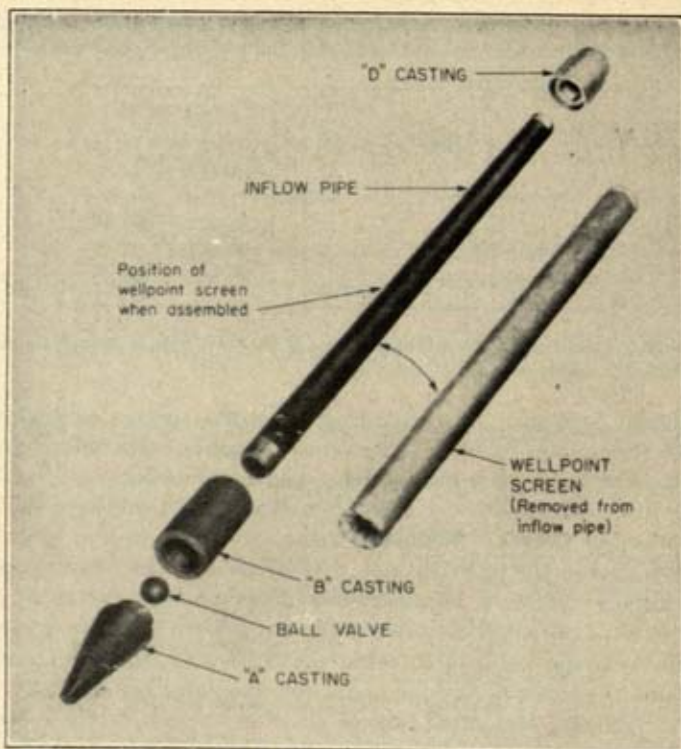


FIG. 15-9. The essential parts of a wellpoint system. (Griffin Wellpoint Corp.)

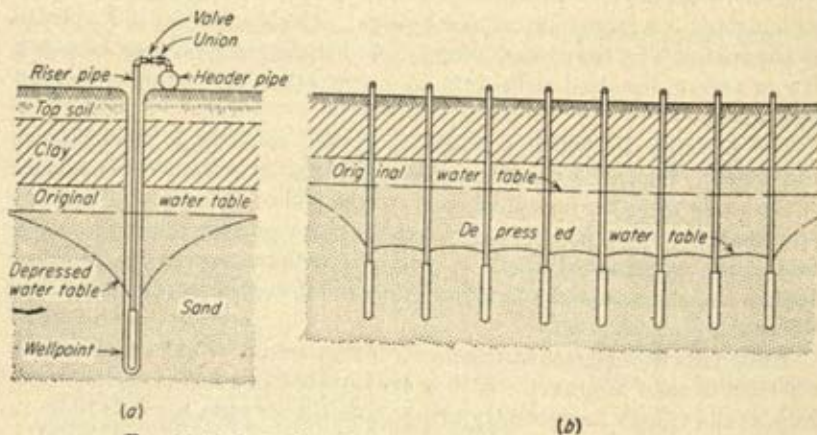


FIG. 15-10. Lowering the water table adjacent to wellpoints.

wellpoint, fill the space inside the pipe with sand or fine gravel, then withdraw the pipe. This leaves a volume of sand around each wellpoint to act as a water collector and a filter to increase the rate of flow for each point.

Wellpoints may be installed at any desired spacing, usually varying from 2 to 5 ft, along the header. The maximum height that water can be lifted is about 18 to 20 ft. If it is necessary to lower the water table to a greater depth, one or more additional stages should be installed, each

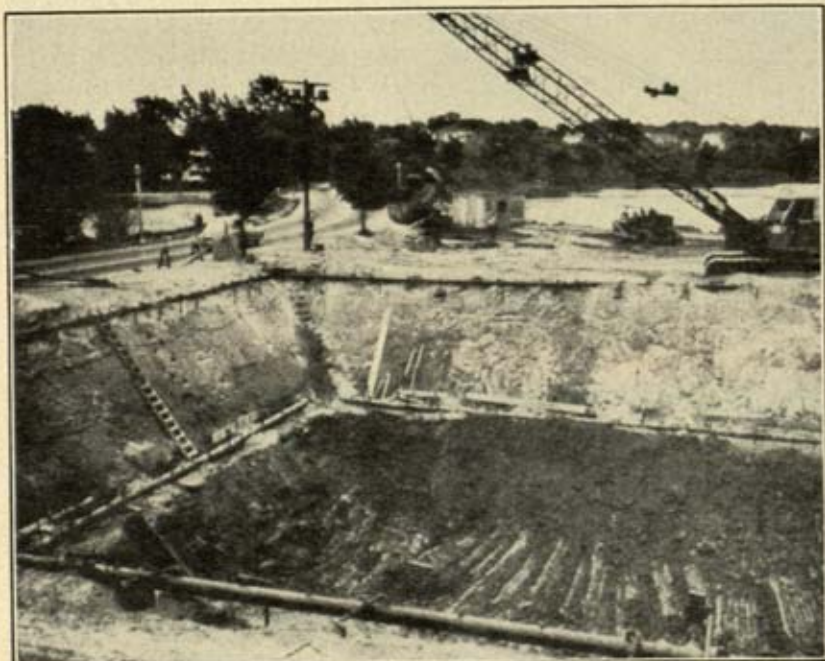


FIG. 15-11. Two-stage wellpoint installation. (*Moretrench Corp.*)

stage at a lower depth within the excavation. Figure 15-11 shows a project on which two stages were used. Figure 15-12 shows a typical wellpoint installation prior to starting excavation.

**Installing a Wellpoint System.** If the soil conditions are suitable, a wellpoint is jetted into position by forcing water through an opening at the bottom of the point. After each point is jetted into position, it is connected through a pipe or a rubber hose to a header pipe, usually 6, 8, or 10 in. in diameter. A valve is installed between each wellpoint and the header to regulate the flow of water. The header is connected to a self-priming centrifugal pump, which is equipped with an auxiliary air





FIG. 15-12. Typical wellpoint installation. (*John W. Stang Corp.*)

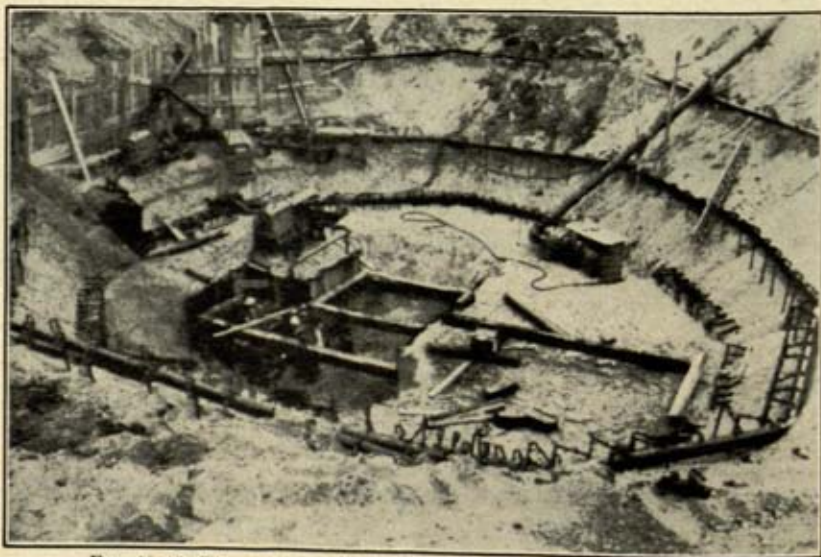


FIG. 15-13. Three-stage wellpoint installation. (*Moretrench Corp.*)

pump to remove any air from the water before it enters the pump proper [1].

**Capacity of a Wellpoint System.** The capacity of a wellpoint system depends on the number of points installed, the permeability of the soil, and the amount of water present. An engineer who is experienced in this kind of work can make tests which will enable him to estimate with reasonable accuracy the capacity necessary to lower the water to the desired depth. The flow per wellpoint will vary from 3 or 4 gpm to as much as 30 or more on some installations.

When excavating 45 to 50 ft below the surface of the water in the Colorado River for the cutoff wall for the Morelos Dam, the contractor installed three main stages, with a supplemental fourth stage of wellpoints to enclose an area of 15 acres [2]. A total of 2,750 wellpoints were serviced by 49 pumps. The maximum pumping rate was 17,400 gpm, with 2,150 wellpoints in operation. This gave an average yield of 8.1 gpm per point and 528 gpm per pump.

Prior to designing the wellpoint system for the Davis Dam on the Colorado River, a 45- by 58-ft test area was enclosed with 66 wellpoints, spaced  $3\frac{1}{2}$  ft apart, each 21 ft long. The test, which was run for 172 hr, using two 8-in. pumps, gave an average yield of 13 gpm per wellpoint [3].

## PROBLEMS

**15-1.** A two-cylinder duplex double-acting pump, size 5 by 10 in., is driven by a crankshaft which makes 120 rpm. If the water slippage is 5 per cent, how many gallons of water will the pump deliver per minute? If the total head is 120 ft and the efficiency of the pump is 60 per cent, what is the minimum horsepower required by the pump?

**15-2.** The air-operated centrifugal pump whose performance curves are given in Fig. 15-4 will be used to pump 100,000 gal of water against a total head of 60 ft. Assume that compressed air costs \$0.10 per 1,000 cu ft of free air at 60 psi, \$0.11 at 80 psi, and \$0.12 at 100 psi. Determine the cost of pumping the water when each of the three air pressures is used.

**15-3.** The 10-in. centrifugal pump whose performance curves are given in Fig. 15-5 will be used to pump water against a total head of 60 ft. The dynamic suction lift will be 15 ft. Determine the capacity and efficiency of the pump and the horsepower required to operate the pump. If the energy delivered to the pump costs \$0.03 per hp-hr, determine the cost per hour for operating the pump.

**15-4.** A centrifugal pump is to be used to pump all the water from a cofferdam whose dimensions are 90 ft long, 50 ft wide, and 20 ft deep. The water must be pumped against an average head of 48 ft. The average height of the pump above the water will be 15 ft. If the cofferdam must be emptied in 24 hr, determine the minimum model pump to use, based on AGC rating capacity.

**15-5.** A model 30-M centrifugal pump is to be used to pump water through a 4-in. pipe, 650 ft long. The water will be discharged into open air at a point 56 ft above the pond from which it will be pumped. If the pump is located 10 ft above the water, determine the capacity in gpm.



**15-6.** Use Table 15-2 to select a centrifugal pump to handle a minimum of 300 gpm of water. The water will be pumped from a pond through 420 ft of 4-in. pipe to a point 40 ft above the level of the pond, where it will be discharged into the air. The pump will be 20 ft above the water in the pond. What is the probable capacity of the pump selected?

**15-7.** Select a centrifugal pump, with a capacity of 275 gpm, for the project illustrated in Fig. 15-7. Change the size of the pipe and fittings to 4 in.

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2. Wellpoints Keep Colorado River Dams Dry, *Construction Methods and Equipment*, vol. 31, pp. 62-67, August, 1949.
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## CHAPTER 16

### COFFERDAMS

**Introduction.** A cofferdam is a temporary structure which is built to exclude earth and water from an area in order that work may be performed there under reasonably dry conditions. Cofferdams are usually required for projects such as dams, locks, and piers that are constructed in rivers or in other bodies of water. Also, they may be installed to prevent the flow of earth and water into foundation pits excavated on land.

A cofferdam does not have to be entirely watertight in order to be successful. It usually is cheaper to permit some flow of water into the working area, which water can be removed with pumps, than to attempt to make the cofferdam watertight. The most satisfactory cofferdam is the one that has adequate strength to resist all destructive forces and that will permit the exclusion from or the control of water inside the dam at the lowest total cost. The total cost includes the cost of the cofferdam, the cost of damages due to water flowing into or over the dam, and the cost of pumping water from the area inside the dam, less any salvage value after the dam is removed. For many projects it is possible to determine these costs with reasonable accuracy prior to designing the cofferdam.

**Forces Acting on Cofferdams.** The forces acting on a cofferdam may include the weight of the dam, the pressure of water, the scouring effect of moving water, the upward reaction of the earth, and the lateral pressure of earth in contact with the dam. A dam should be designed to resist the combined effect of all forces that will act on it.

**Hydraulic Pressure on Cofferdams.** The unit pressure acting at any depth below the surface of water is given by the formula

$$p = wh \quad (16-1)$$

where  $p$  = pressure, psf

$w$  = weight of a cubic foot of water, lb

$h$  = depth below surface, ft

The total pressure acting on an area below the surface of water is given by the formula

$$P = pA = whA \quad (16-2)$$

where  $P$  = total pressure, lb

$A$  = area, sq ft



If all the area  $A$  in formula (16-2) is not the same depth below the surface,  $h$  should be the depth to the center of gravity of  $A$ .

The forces acting on a section of a rock-fill cofferdam 1 ft long are shown in Fig. 16-1. For simplicity, the rock and the layer of impervious earth on the right side of the dam are assumed to weigh 120 lb. per cu ft each.

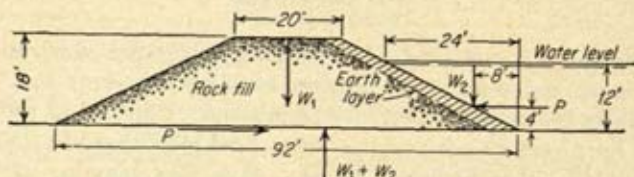


FIG. 16-1. Forces acting on a rock-fill cofferdam.

The combined weight of rock and earth will be

$$W_1 = \frac{92 + 20}{2} \times 18 \times 120 = 121,000 \text{ lb}$$

The weight of water above the outer slope of the dam will be

$$W_2 = \frac{12 \times 24}{2} \times 62.5 = 9,000 \text{ lb}$$

The horizontal pressure of the water tending to push the dam to the left is

$$P = 62.5 \times 6 \times 12 = 4,500 \text{ lb}$$

The earth on which the dam rests must exert an upward force equal to  $W_1 + W_2$ , and a horizontal force to the right equal to  $P$ , in order to maintain the stability of the dam.

**Seepage of Water into Cofferdams.** Since few cofferdams are watertight, the flow of water into a cofferdam must be considered in planning construction operations. The water may enter through openings in the dam, or it may enter through the soil underneath the dam. This discussion is devoted to the water that flows under a cofferdam and enters the construction area.

If a cofferdam is installed by driving a solid wall of sheeting, such as interlocking steel-sheet piling, through water or a pervious water-bearing soil, such as sand and gravel, into an impervious soil, such as clay or shale, the possibility of water flowing under the cofferdam is eliminated. This condition is illustrated in Fig. 16-2.

When it is not practical to drive sheeting into an impervious soil, water will flow under a cofferdam any time there is a difference in the hydro-

static levels on the two sides of the dam. The total quantity entering an area inside a cofferdam will depend on the velocity of flow and the area through which it flows. While many tests have been conducted to determine the rate of flow of ground water, variations in soil conditions prevent an accurate estimate of the flow. The results obtained by Hazen

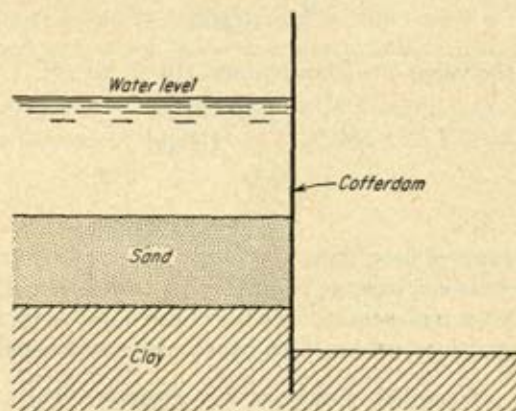


FIG. 16-2. Sheet-piling cofferdam driven into impervious soil.

for sands varying from 0.1 to 3.0 mm in effective size are given by the formula

$$v = cd^2s \frac{t + 10}{60} \quad (16-3)$$

where  $v$  = velocity of flow, m per day

$c$  = a constant, varying from 400 to 1,000

$d$  = effective size of sand, mm

$s$  = slope of hydraulic gradient

$t$  = temperature, °F

The velocity given in formula (16-3) is the velocity that would exist if no sand were present to reduce the area of passage. The presence of sand has the effect of increasing the true velocity to  $v/p$ . Hazen defined the effective size of soil as that size for which 10 per cent is smaller and 90 per cent is larger.

If the refinement of correcting for variations in temperature is omitted, as it probably should be on construction projects, and the velocity is expressed in feet per day, we get the formula

$$v = \frac{3.3cd^2}{p} s = ks \quad (16-4)$$



where  $v$  = true velocity of flow through the voids, ft per day

$p$  = porosity ratio of soil

$k = 3.3cd^2/p$

The volume of flow through a given area, expressed in gpm, is given by the formula

$$Q = \frac{vAp}{10,800} \quad (16-5)$$

Substituting the value of  $v$  from formula (16-4), we get

$$\begin{aligned} Q &= \frac{3.3cd^2s}{p} \frac{Ap}{10,800} \\ &= \frac{cd^2As}{3,270} \end{aligned} \quad (16-6)$$

where  $Q$  = volume of flow, gpm

$A$  = area of soil passage perpendicular to direction of flow, sq ft.

Table 16-1 gives representative values of  $k$  or  $3.3cd^2/p$  for use in formula (16-4). These values are based on experiments conducted by Hazen.

TABLE 16-1. VALUES OF  $k$  IN FORMULA (16-4)

Porosity ratio	Fine		Medium	Coarse		Fine gravel			
	0.10	0.20	0.30	0.40	0.50	0.80	1.00	2.00	3.00
0.25	27	112	250	460	700	1,790	2,800	11,200	25,000
0.30	43	172	386	686	1,070	2,740	4,290	17,200	38,600
0.35	60	240	540	960	1,500	3,840	6,800	24,000	54,000
0.40	82	330	740	1,320	2,060	5,280	8,250	33,000	74,000

EXAMPLE. A steel-sheet-piling cofferdam encloses an area 20 ft wide by 40 ft long. The pit is excavated to a depth 20 ft below the level of water outside the dam. The average distance from the top of the sand outside the dam to the center of the bottom of the pit is 30 ft. The soil has an effective size of 0.5 mm and a porosity of 0.3. For this porosity the value of  $c$  will probably be 400.

In formula (16-4)

$$\begin{aligned} c &= 400 & d &= 0.5 & p &= 0.3 & s &= 2\%_{30} = 0.667 \\ k &= \frac{3.3 \times 400 \times 0.5^2}{0.3} = 1,100 & \text{see Table 16-1} \end{aligned}$$

From formula (16-4)

$$v = 1,100 \times 0.667 = 734 \text{ ft per day}$$

Substituting this value of  $v$  in formula (16-5), we get

$$Q = \frac{734 \times 20 \times 40 \times 0.3}{10,800} = 16.3 \text{ gpm}$$

As indicated in Table 16-1, the effective size of the soil has considerable effect on the rate of flow of water through the soil. This is illustrated by the experience with the cofferdam for the Chain of Rocks Locks on the Mississippi River. Tests of the soil under the cofferdam and the area inside the dam indicated a probable flow of 100,000 gpm into the area. The actual flow proved to be only 5,000 to 8,000 gpm. Retests of the soil revealed that in conducting the original tests some of the fine material had washed out, which gave an apparent effective size that was too large [1].

The paths followed by particles of water flowing under a sheet-piling cofferdam are illustrated by lines 1, 2, 3, and 4 in Fig. 16-3. As line 1

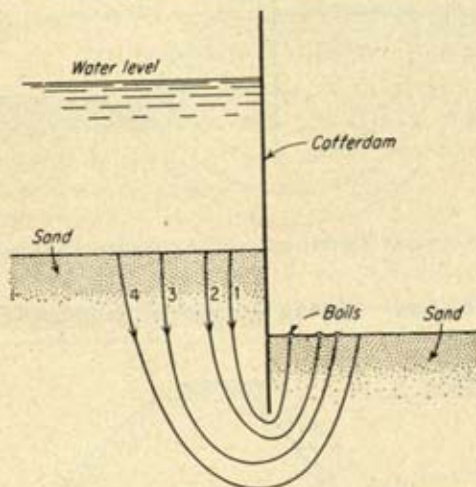


FIG. 16-3. Paths followed by water flowing under a sheet-piling cofferdam.

offers the shortest path, the velocity of flow will be the greatest along this line. The velocity along lines 2, 3, and 4 will be correspondingly less. If the velocity of the water entering the cofferdam is sufficiently high it may agitate the sand and cause boils at the bottom of the pit, as indicated.

If an impervious blanket is placed outside and a berm of earth is placed inside a sheet-piling cofferdam, as illustrated in Fig. 16-4, the increased flow distance will reduce the value of the hydraulic gradient  $s$  in formula (16-3), which will reduce the velocity and quantity of water flowing under the cofferdam. In addition to reducing the flow of water, the berm adds horizontal stability to the cofferdam.

If the velocity of water flowing under a cofferdam is high enough, it may cause a channel to form through the sand which can result in a violent eruption of the sand and water into the cofferdam, producing what is referred to as a blow-in. This, of course, can be a major catastrophe.



When the wall of a cofferdam is installed to resist the horizontal pressure of earth and ground water, it may be desirable to leave enough openings in the wall to permit the water to flow through it as a means of reducing the pressure against the wall. This plan is reported to have been used with considerable success by White and Prentis in building cofferdams on the Mississippi River [1].

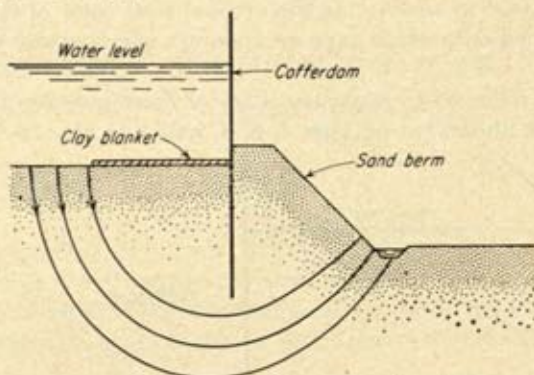


FIG. 16-4. The use of a clay blanket and a berm to reduce the flow of water under a cofferdam.

**Types of Cofferdams.** Among the types of cofferdams frequently used are the following:

1. Water cofferdams
  - a. Earth-fill
  - b. Rock-fill
  - c. Ohio type
  - d. Rock-filled crib
  - e. Sheet piling with bracing
  - f. Steel-sheet piling cells with diaphragms
  - g. Steel-sheet piling with circular cells
2. Land cofferdams
  - a. Wood-sheet piling
  - b. Steel-sheet piling
  - c. Horizontal sheeting

**Earth-fill Cofferdams.** When impervious earth is available near a project, a satisfactory and economical earth-fill cofferdam may be constructed in shallow water. As earth offers little resistance to erosion from moving water or wave action, this type dam should be limited to use in water with little or no movement. An earth-fill dam is economical for low heights, but as the height is increased, it may prove to be more expensive than some other type. An earth dam should not be used where there

is danger of overtopping by water. A typical earth-fill cofferdam is illustrated in Fig. 16-5.

**Rock-fill Cofferdams.** A rock-fill cofferdam is constructed by placing rock across a stream or around an area to be dewatered. Such a dam is quite satisfactory and economical if a large quantity of rock is available, such as from tunnels or excavations. A dam of this type is constructed by placing earth with the rock or by placing a layer of earth around the

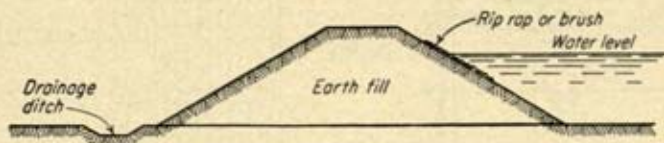


FIG. 16-5. Earth-fill cofferdam.

outside of the rock. The weight of the rock gives stability to the dam, while the earth supplies the watertight membrane. This type of dam may be overtopped by water without serious damage. Figure 16-6 shows a typical section through a rock-fill cofferdam.

Prior to constructing the Davis Dam on the Colorado River, the contractor excavated a diversion and forebay channel adjacent to the river. As soon as this channel was completed, the contractor began hauling rock onto a trestle which had been installed across the river below the entrance to the channel. Approximately 29,000 cu yd of rock was placed in 56 hr

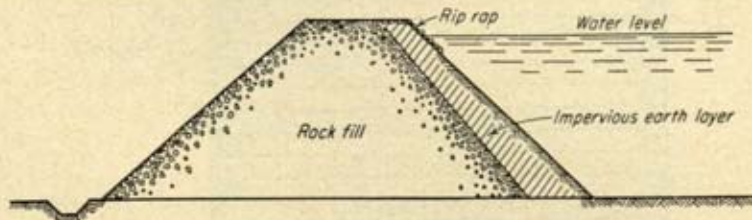


FIG. 16-6. Rock-fill cofferdam.

by 20 bottom-dump wagons. The river flow varied from 12,000 to 18,000 sec-ft during the diversion operation. It is estimated that approximately one-third of the rock was washed downstream by the river. After the rock fill was placed, the upstream face of the cofferdam was lined with impervious material to produce a watertight structure.

**Ohio River-type Cofferdams.** This type of cofferdam derives its name from its use on the Ohio River. A dam is constructed by prefabricating a continuous row of timber frames with cross bracing and steel tie rods, as illustrated in Fig. 16-7. After the frames are in place, vertical sheeting is installed on each side of the frames and the space in between is filled with impervious earth. An earth or rock berm may be placed against a



dam upstream and downstream to give greater stability and to reduce leakage through or under the dam. This type dam should not be used where the velocity of the water is high or where there is danger of serious overtopping.

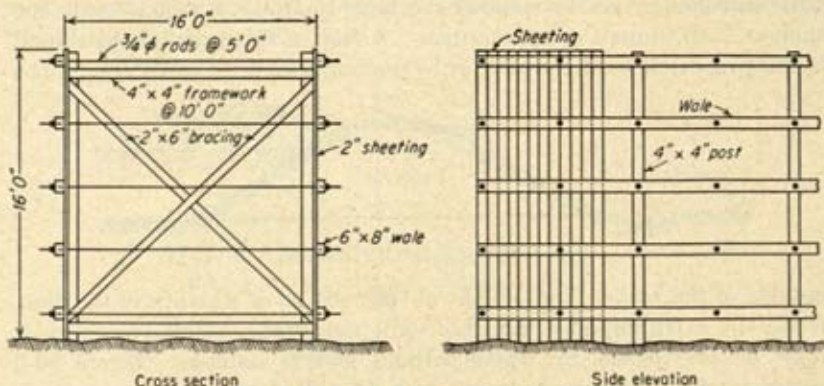


FIG. 16-7. Ohio River-type cofferdam.

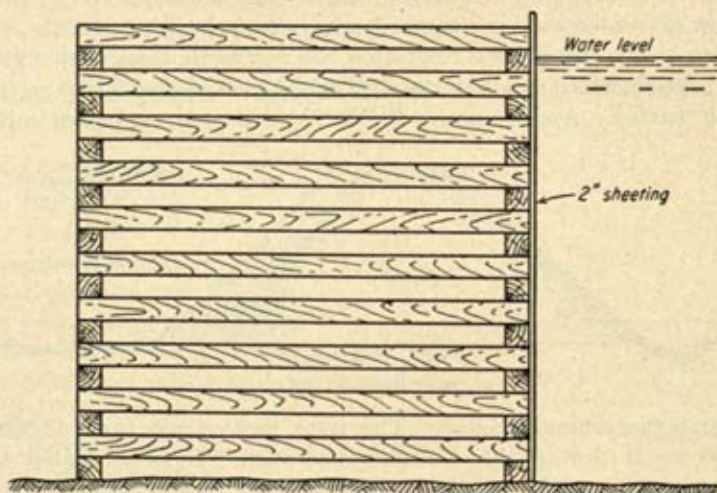


FIG. 16-8. Crib-type cofferdam.

**Crib-type Cofferdams.** A crib-type cofferdam is constructed by installing a series of connected cribs around or across an area to be dammed off. A crib is a framework of horizontal timbers installed in alternate courses to form pockets which are filled with rock and earth. The timber may be logs, or it may be heavy lumber, usually fastened together with driftpins, and sometimes with heavy vertical sheeting spiked to the timbers. A cross section through a typical crib-type cofferdam is shown in Fig. 16-8.

This type of cofferdam is used in a stream with a hard bottom, deep water, and swift current, where there is danger of overtopping, and where timber is relatively cheap. Unless these conditions exist, some other type cofferdam may be more economical. The Conwingo cofferdam had a maximum crib width of 30 ft and height of 35 ft, while the Bonneville cofferdam had a maximum crib width of 60 ft and height of 63 ft. The dimensions of a crib must be such that when it is filled with rock and earth it will be able to resist overturning and sliding with full water pressure on one side [see formulas (16-7) and (16-8)].

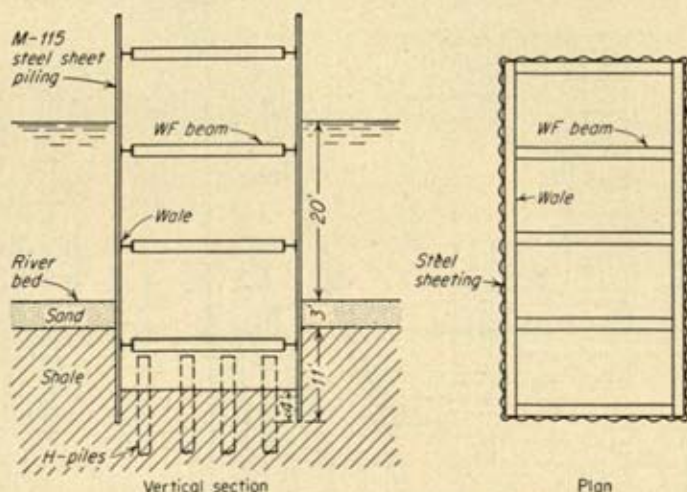


FIG. 16-9. Single-wall braced cofferdam.

**Single-wall Steel-sheet-piling Cofferdams.** A cofferdam of this type is satisfactory for use under certain conditions such as:

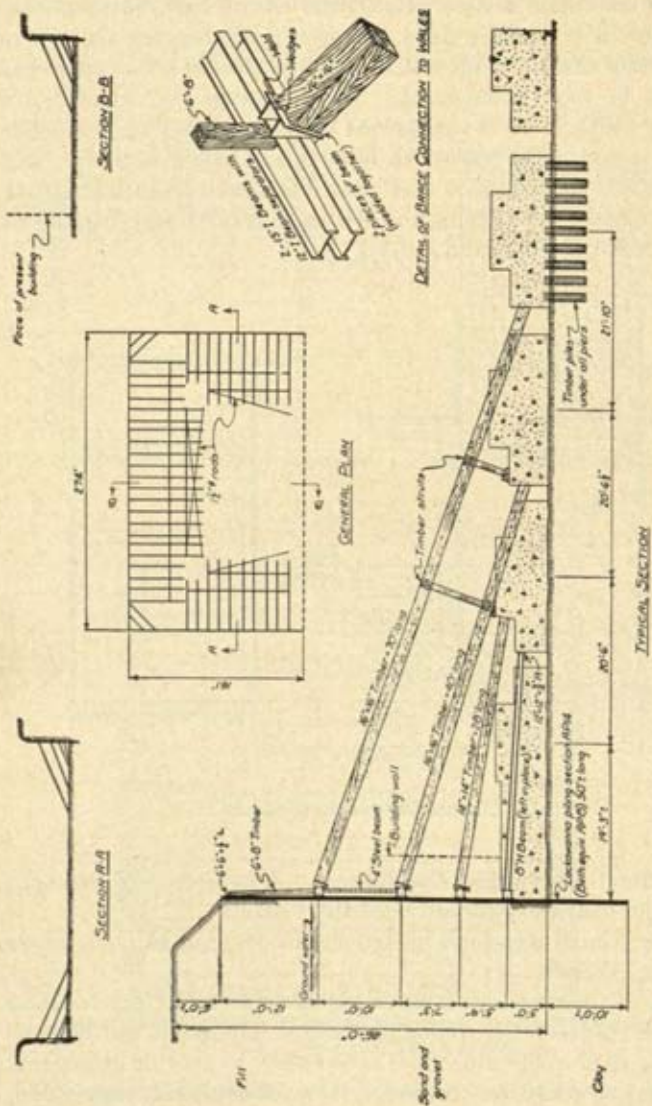
1. To enclose a small area for a bridge pier in water or on land, where the depth is not excessive

2. To enclose a land area for constructing the foundation for a building

As a single wall of steel-sheet piling has limited strength in resisting the horizontal pressure of water or earth, it is necessary to provide an internal system of bracing to resist the pressure. If the dimensions across a dam are not too great, rows of wales and cross braces will be satisfactory and economical. A single-wall braced cofferdam for a bridge pier is illustrated in Fig. 16-9.

If the dimensions inside a dam are great, such as for the basement of a building, it may not be practical to extend braces across from wall to wall. For this type of construction a system of bracing such as that illustrated in Fig. 16-10 may be used.





**COFFERDAM FOR  
NORTHWESTERN MUTUAL LIFE INS CO BUILDING**

MILWAUKEE, WIS.

Edward E. Glavin Co., Milwaukee, Wis., Building Contractor  
Gooder-Hendrichsen Co. Inc., Chicago, Ill., Shoring Contractor

Fig. 16-10. Representative bracing for a single-wall cofferdam. (Bethlehem Steel Co.)

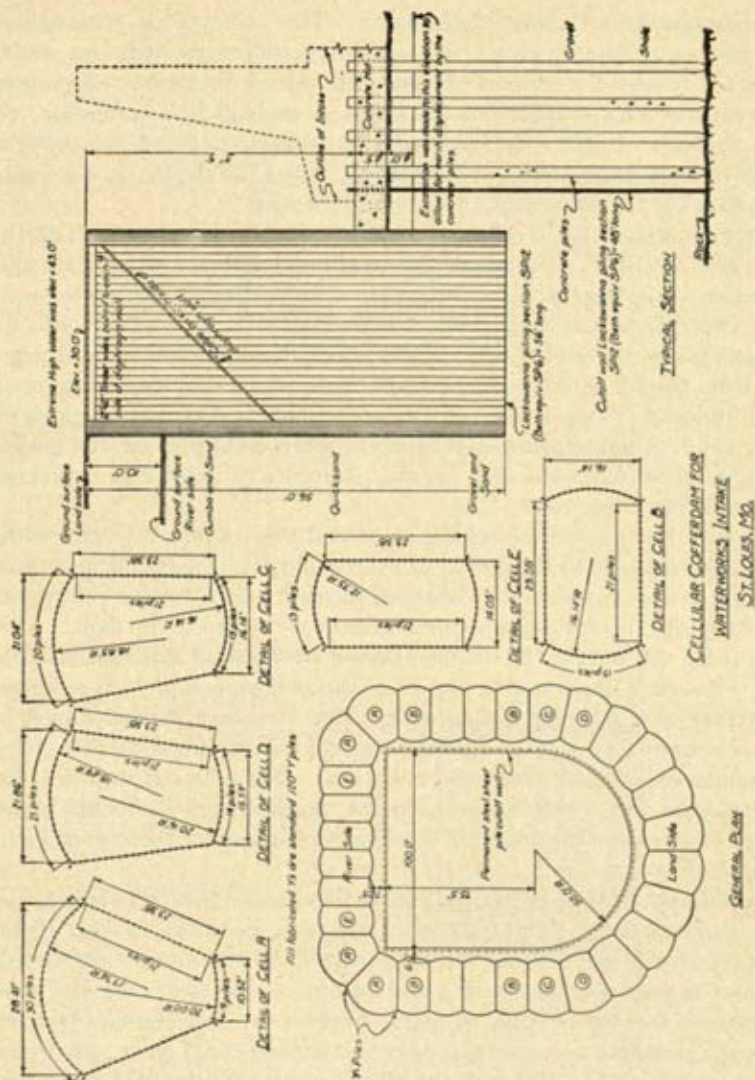


Fig. 16-11. Diaphragm-type cellular cofferdam. (Bethlehem Steel Co.)



**Diaphragm-type Cellular Cofferdams.** This structure is constructed of steel-sheet piling to give a self-sustaining cofferdam with two walls. The walls consist of a series of arc segments, which are connected at their intersections with diaphragms that extend through the cofferdam, to form a group of cells. A diaphragm is connected to the arcs with a Y pile at each connection. Figure 16-11 shows the details of a cellular cofferdam for the waterworks intake at St. Louis.

After a number of the cells are constructed, they are filled with earth, sand, gravel, or rock to increase the weight and stability of the dam and to reduce the leakage of water through it. As the diaphragm which separates two cells is a straight wall, it is necessary to fill adjacent cells at approximately the same rate. If this is not done, the unbalanced pressure from the fill will distort the diaphragm, which may cause failure of the interlocks. This is one of the objections to the diaphragm-type cofferdam. The literature on steel-sheet piling published by the manufacturers gives recommended working strengths of interlocks expressed in pounds per linear inch.

The cellular-type cofferdam will withstand overtopping by water without serious damage to the dam. If overtopping is likely to occur, one or more gated openings should be installed through a dam in order that water may flow into the enclosure before it goes over the top of the dam. This precaution will reduce the damage caused by flooding the area inside a dam. The dam illustrated in Fig. 16-11 was overtopped by 13 ft of water.

**Circular-type Cellular Cofferdams.** This structure is constructed of steel-sheet piling to give a self-sustaining cofferdam consisting of a group of circular cells joined with connecting arcs. The cells and arcs produce an enclosure to exclude water from the working area inside the dam. Figure 16-12 shows the details of a cellular cofferdam for the lock at Pickwick Landing Dam.

A cellular cofferdam is constructed by driving the piles which form the cells, then connecting the cells with two arcs, as shown in Fig. 16-14. Each cell should include four T piles, to which the arc piles are connected. In order to facilitate the spacing and driving of the piles, a circular steel template is located on steel supports where a cell is to be constructed. With the template as a guide, the piles, including the T piles, are driven in correct positions. After all the piles in a cell are driven, the template and supports are lifted out and relocated for another cell. Figure 16-13 shows several steps in constructing the cells for a circular-type cofferdam. The arcs are installed after the cells are completed.

As each cell is a stable unit, subjected to a tensile force only in the walls when filled with earth, it is possible to fill a cell immediately after it is constructed. In this respect the circular-type cofferdam is superior to the diaphragm type, which requires that adjacent cells be filled at the





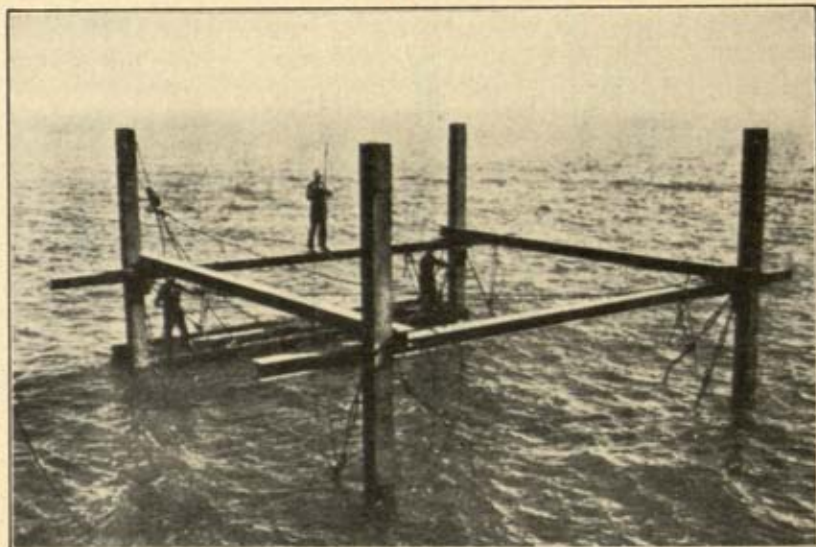


FIG. 16-13a-e. Successive steps in constructing the cells for a circular-type cofferdam. (Guy F. Atchison Co.) (a) H piles are driven in square and braced with struts and supports.

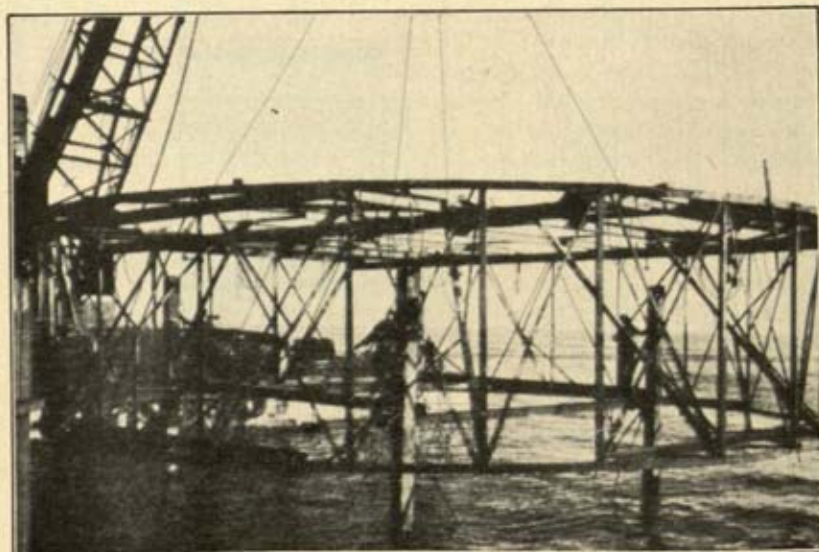


FIG. 16-13b. Prefabricated circular template is lowered over H piles.

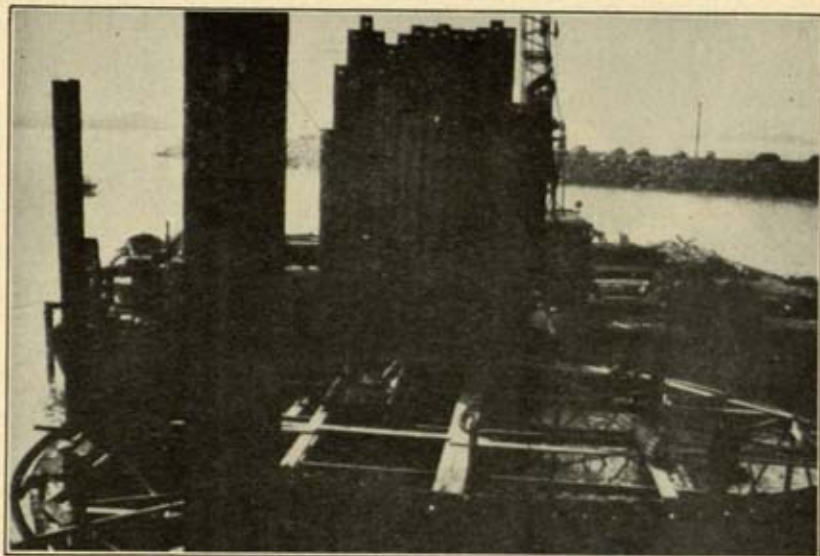


FIG. 16-13c. Sheet piles are driven around the template.

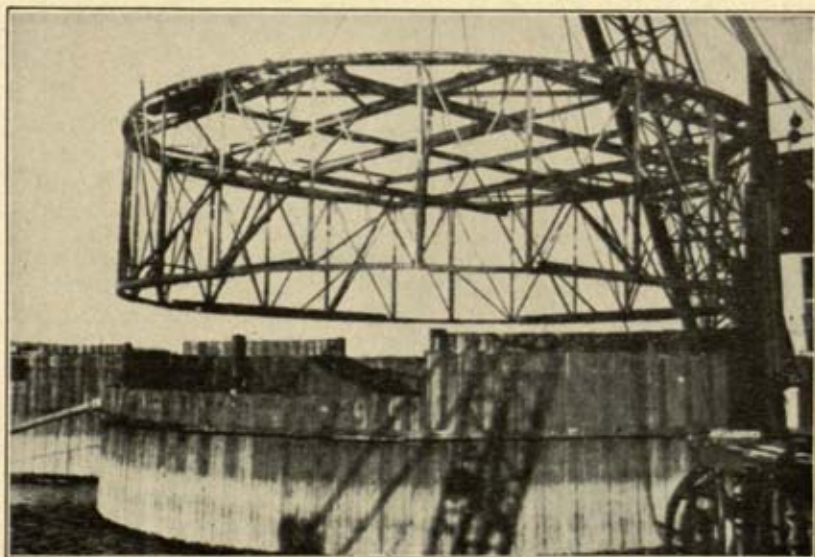


FIG. 16-13d. Template is lifted out of finished cell to be used again.



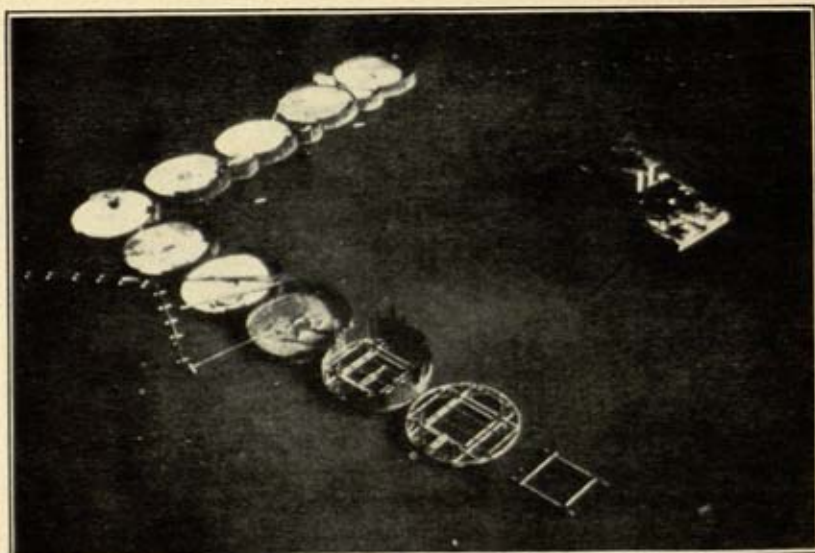


FIG. 16-13e. Stages in the construction of circular cells.

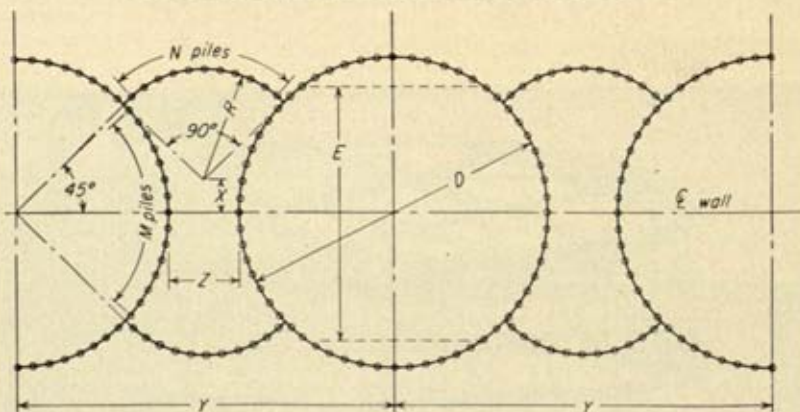


FIG. 16-14. Dimension nomenclature for cellular structures of circular-type cofferdam. Use with Table 16-2. (U.S. Steel Corp.)

same time. After a cell is filled with earth, it is possible to locate on the earth a crane or other equipment needed to construct additional cells.

The piles in the cells and arcs on the water side of a cofferdam may be extended several feet above the tops of the inside piles to give a dam extra height at relatively low cost.

Representative dimensions and other information for circular-type cofferdams are given in Fig. 16-14 and Table 16-2.

TABLE 16-2. REPRESENTATIVE DIMENSIONS AND NUMBERS OF STEEL-SHEET  
PILING FOR CIRCULAR-TYPE COFFERDAMS USING 16-IN.-WIDE PILING  
(SEE FIG. 16-14)\*

No. † piles in cell	D, ft.	Y, ft.	R, ft.	X, ft.	Z, ft.	E, ft.	No. M piles	No. N piles	Area, sq ft	
									Within circle	Between circles
48	20.36	26.47	8.54	1.16	6.11	17.40	11	9	325.6	156.3
52	22.06	27.67	8.54	1.76	5.61	18.73	12	9	382.2	160.5
56	23.76	30.08	9.39	1.76	6.32	20.11	13	10	443.4	192.8
60	25.46	31.28	9.39	2.36	5.82	21.50	14	10	509.1	196.8
64	27.16	32.48	9.39	2.97	5.32	22.91	15	10	579.4	200.0
68	28.85	34.88	10.24	2.96	6.03	24.32	16	11	653.7	236.3
72	30.55	36.15	10.24	3.60	5.60	25.70	17	11	733.1	239.5
76	32.25	37.28	10.24	4.16	5.03	27.12	18	11	816.9	241.5
80	33.95	38.48	10.24	4.77	4.53	28.53	19	11	905.3	242.8
84	35.65	40.88	11.08	4.77	5.23	29.95	20	12	998.2	283.9
88	37.34	42.08	11.08	5.36	4.74	31.24	21	12	1,095.1	285.0
92	39.04	43.28	11.08	5.97	4.24	32.52	22	12	1,197.0	285.4
96	40.74	44.48	11.08	6.57	3.74	33.94	23	12	1,303.6	284.8
100	42.43	46.88	11.93	6.56	4.45	35.36	24	13	1,414.0	330.7
104	44.14	48.09	11.93	7.17	3.95	36.70	25	13	1,530.2	330.0
108	45.83	49.28	11.93	7.77	3.45	38.10	26	13	1,649.6	328.4
112	47.52	51.68	12.78	7.76	4.16	39.51	27	14	1,773.6	378.3
116	49.22	52.88	12.78	8.36	3.66	40.91	28	14	1,902.7	376.6
120	50.92	54.08	12.78	8.97	3.16	42.34	29	14	2,036.4	374.1
124	52.62	56.48	13.63	8.97	3.86	43.77	30	15	2,174.7	428.1
128	54.31	57.68	13.63	9.56	3.37	45.10	31	15	2,316.6	425.4
132	56.01	60.08	14.48	9.57	4.07	46.55	32	16	2,463.9	482.9
136	57.71	61.28	14.48	10.17	3.57	48.00	33	16	2,615.7	480.0
140	59.41	62.48	14.48	10.77	3.07	49.42	34	16	2,772.1	476.1
144	61.11	64.88	15.33	10.76	3.77	50.83	35	17	2,933.0	537.8
148	62.80	66.08	15.33	11.37	3.28	52.21	36	17	3,097.5	533.8

\* Courtesy United States Steel Corporation.

† Includes four T piles.

In Fig. 16-14 and Table 16-2,  $E$  indicates the theoretical width of a rectangular wall having a resistance against overturning equal to that of the cellular wall. The value of  $E$  may be increased almost in proportion to the increase in the number of piles in and the diameter of a cell. The increase in the diameter of a cell will produce only a slight increase in the total number and weight of piles required for a given length cofferdam. For example, consider a circular-type cofferdam having 60 piles per cell and another having 120 piles per cell. Reference to Table 16-2 indicates that for a length of 100 ft along the center line of the dam the following will apply:



For 60 piles per cell

$$\text{No. cells, } 100 \div 31.28 = 3.2$$

$$\text{No. piles in cells, } 3.2 \times 60 = 192$$

$$\text{No. piles in arcs, } 2 \times 3.2 \times 10 = 64$$

$$\text{Total no. piles} = 256$$

$$\text{Value of } E = 21.50 \text{ ft}$$

For 120 piles per cell

$$\text{No. cells, } 100 \div 54.08 = 1.85$$

$$\text{No. piles in cells, } 1.85 \times 120 = 222$$

$$\text{No. piles in arcs, } 2 \times 1.85 \times 14 = 52$$

$$\text{Total no. piles} = 274$$

$$\text{Value of } E = 42.34 \text{ ft}$$

Thus, the value of  $E$  is almost doubled, whereas the number of piles required is increased by 7 per cent. The larger-diameter cells will require

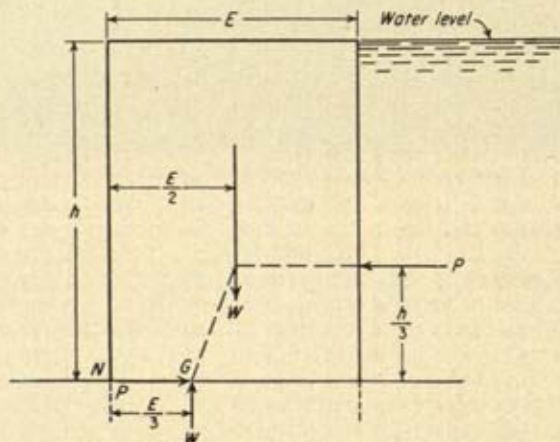


FIG. 16-15. The forces acting on a cofferdam.

more earth fill, which may add materially to the cost of a dam, and they produce higher stresses in the interlocks of the piles, which may endanger the safety of a dam.

**Designing Circular-type Cellular Cofferdams.** This type cofferdam is filled with earth within the circles and between the circles. In order that such a dam may safely resist the pressure of water on the outside, it should be designed as a gravity dam. The forces acting on it are the weight of the earth and the piling, the pressure of the water on one side, the upward reaction of the soil under the dam and the resistance of the soil under the dam to sliding. All these forces are shown in Fig. 16-15. The relationship between all forces acting on a cofferdam should be such that the resultant of  $W$  and  $P$  will intersect the bottom of the dam not closer than  $E/3$  from the inside toe at  $N$ .

Figure 16-15 shows a section through a cofferdam whose width is  $E$ , as given in Fig. 16-14 and Table 16-2. For calculation purposes, consider a section of a dam 1 ft long. Assume that all piling are the same length and that fresh water stands at the top of the dam. Assume that the earth fill inside the dam weighs 100 lb per cu ft and that the water weighs 62.5 lb per cu ft.

Let  $h$  = height of dam, ft

$E$  = theoretical width, ft, of a rectangular dam having a resistance against overturning equal to that of the cellular dam

$P$  = total pressure, lb, of water on a section of the dam 1 ft long

$W$  = weight, lb, of the earth in a section of the dam 1 ft long

$w$  = density of water, equal to 62.5 lb per cu ft

$d$  = weight of 1 cu ft of earth, assumed to be 100 lb

$A$  = area, sq ft, of section of dam on which  $P$  acts

$$P = \frac{Awh}{2} = \frac{h \times 62.5h}{2} = \frac{62.5h^2}{2}$$

$$W = dEh = 100Eh$$

If we use the narrowest width of dam permissible, the resultant of  $W$  and  $P$  will intersect the base at point  $G$ . With this condition existing, if we equate the algebraic sum of all forces acting on the dam about  $N$  to zero, we get

$$\frac{WE}{2} - \frac{Ph}{3} - \frac{WE}{3} = 0$$

Substituting the values of  $W$  and  $P$  gives

$$\frac{100E^2h}{2} - \frac{62.5h^3}{6} - \frac{100E^2h}{3} = 0$$

Divide by  $h$ , and reduce to a common denominator.

$$300E^2 - 62.5h^2 - 200E^2 = 0$$

$$62.5h^2 = 100E^2$$

$$\frac{h^2}{E^2} = \frac{100}{62.5} = 1.6$$

$$\frac{h}{E} = \sqrt{1.6} = 1.265$$

$$h = 1.265E \quad (16-7)$$

This formula gives the theoretical maximum safe height for a dam. In deriving the formula the weight of the sheet piling and the effect of skin friction on the portion of the piling driven into the soil were neglected. These forces will increase the stability and safety of the dam. If a dam



is constructed in salt water or in a flowing stream, the pressure of the water on the dam will be increased slightly, which will reduce the stability of the dam. In order to provide a factor of safety against variations that may occur, the height should be less than the value given by formula (16-7). A height not greater than the value given by formula (16-8) should be safe and satisfactory for most dams.

$$h = 1.2E \quad (16-8)$$

**Economy of Cofferdam Height.** When a cofferdam is constructed in a river, it is necessary to determine the height that will be most satisfactory.

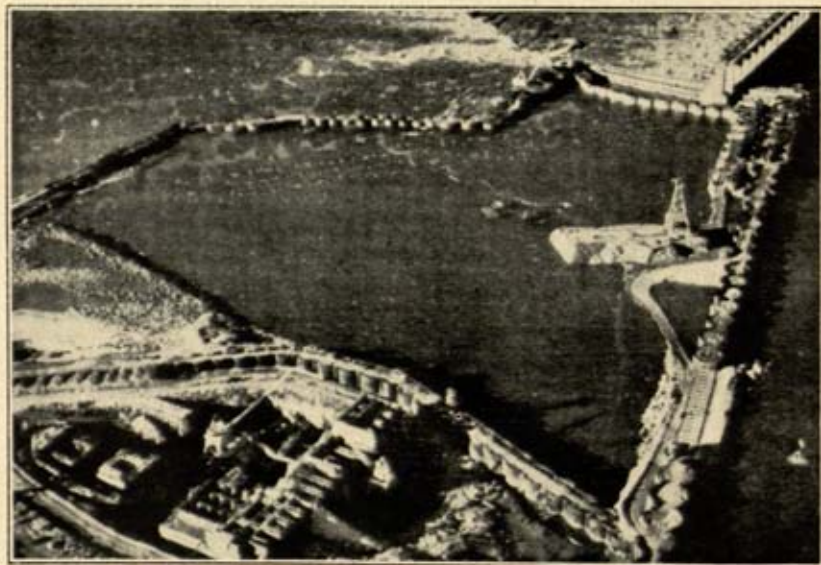


FIG. 16-16. Final cofferdam for the McNary Dam. (*Engineering News-Record*.)

The most satisfactory height is the one that will produce the lowest total cost considering the cost of the dam and the cost of risk that a flood may overtop the dam. In order to be satisfactory, a cofferdam in a river does not need to be high enough to exclude all floods that may occur in the river. The extra cost of constructing a dam to such a height might exceed the maximum damage that would be caused by a flood. Under this condition it is cheaper to assume the risk of loss due to flood damage than to build a dam high enough to eliminate the danger of overtopping.

For the rivers in the United States and throughout most of the world records are available which indicate with reasonable accuracy the frequency at which the stage of a river will rise to any desired height. For example, it is probable that the stage for a given river will reach or exceed

30 ft four times a year, 34 ft once a year, 38 ft once in 4 years, and 42 ft once in 10 years. It is impossible to predict the time at which any flood will occur, but the frequencies will apply over a reasonably long period of years. If a 34-ft-high cofferdam is in the river for 1 year, it is probable that it will be overtopped once. If the height is increased to 38 ft, the probability of it being overtopped is 1 in 4. Is it economical to increase the height from 34 to 38 ft at an extra cost of \$42,800 if the probable loss from damage due to a flood will be \$40,000? The cost of flood risk for a 34-ft-high dam is \$40,000, resulting from 1 flood in 1 year. The cost of flood risk for a 38-ft-high dam is  $\$40,000 \div 4 = \$10,000$ , resulting from 1 flood in 4 years. The increased cost of the dam is \$42,800, while the reduction in flood risk is  $\$40,000 - \$10,000 = \$30,000$ . As it is not economical to spend \$42,800 to eliminate a \$30,000 risk, the 38-ft height is not justified.

Table 16-3 illustrates a method of determining the most economical height for a cofferdam. The information in the table is based on the dam being in the river for 1 year and a probable loss due to flood damage equal to \$40,000 each time the dam is overtopped. The results show that 34 ft is the most economical height listed. It is possible that the most economical height is between 34 and 36 ft.

TABLE 16-3. THE MOST ECONOMICAL HEIGHT FOR A COFFERDAM

Height of dam, ft	Frequency of floods overtopping the dam, floods per yr	Cost of dam	Cost of flood risk	Combined cost of dam and flood risk
30	4	\$275,800	\$160,000	\$435,800
32	2	304,000	80,000	384,000
34	1	322,000	40,000	362,000
36	0.50	351,400	20,000	371,400
38	0.25	364,800	10,000	374,800
40	0.15	384,000	6,000	390,000
42	0.10	402,500	4,000	406,500

**Freezing a Cofferdam.** A method of providing a substitute for a cofferdam which has been used in Europe and the United States is to freeze a wall of ice around a hole prior to excavating. This method of excluding earth and water from a hole is especially suitable in excavating a deep shaft through loose sand and gravel containing a large quantity of ground water.

When the Potash Company of America found it necessary to install a 15-ft inside-diameter shaft to a depth of 360 ft for its mines in New Mexico, it decided to freeze a cylinder of ice around the shaft opening



as a means of excluding water and sand. It was decided to freeze the formation after an unsuccessful attempt had been made to consolidate it by pumping 1,000 sacks of cement into the sand [2, 3].

Around the circumference of a 31-ft-diameter circle 28 holes were drilled to a depth of 360 ft, to penetrate below the deepest water-bearing formation. A 6-in.-diameter steel casing whose bottom was sealed with a welded steel plate was installed to the bottom of each hole. A 2-in.-diameter pipe, with its bottom end open, was installed inside of and to the bottom of the 6-in. pipe. The top of each 2-in. pipe was connected to an 8-in.-diameter brine pipe which encircled the shaft area. The top of each 6-in. pipe was closed with a hood and connected through a 2-in. pipe to a second 8-in. header pipe. Each injection pipe was equipped with a valve to regulate the flow of brine in order to produce an equal rate of freezing around the shaft. A single 6-in. pipe, with slots through the walls, was installed in a hole near the center of the shaft to provide a means of escape for water forced to the center by the freezing process.

Brine was pumped down through the 2-in. pipes and upward through the 6-in. pipes, thence back through the refrigerating plant. After 48 days of operation the ice wall was sufficiently thick to permit the starting of excavation for the shaft.

A concrete lining was placed in 25-ft-long vertical sections as the excavation progressed. Where the excavation for the shaft encountered ice, the frozen wall was lined with corrugated-metal roofing prior to placing the concrete in order to provide an insulating membrane to prevent the concrete from freezing. Injection pipes were placed through the concrete wall to permit cement grout to be pumped behind the wall after its completion, to fill the air spaces. The grout was injected before the ice wall melted in order to prevent it from flowing outward through fissures and channels.

**Electroosmosis.** Electroosmosis is a means of stabilizing silts and soft clays by the introduction of direct electric current into the soil through electrodes placed in the ground around the area to be stabilized.

The stabilizing action is based on the phenomenon that, when a direct electric current is passed through a capillary, water moves through the capillary in the direction of the current. The water that is moved consists of the free water and of the inner part of the double layer of the boundary film of water that is adjacent to the wall of the capillary [4].

Considering the pores of a fine-grained soil as capillaries, the action of passing the direct current through the soil causes the positive ions adjacent to the soil particles to flow in the direction of the current. This action results in the free water being dragged along toward the negative electrodes, where the water accumulates and eventually flows out of the soil through pipes which are used for the negative electrodes. The con-

stant removal of the free water from the soil causes a reduction in the water content, with a resultant increase in the shearing strength of the soil. By introducing anodes of a specific type of metal, a base exchange can be obtained in certain soils, which causes the soil to become permanently stabilized. This action results in a barrier of stable soil being created around the area, which serves as a wall to retain the unstable soil that exists beyond the area influenced by the electroosmosis system.

The equipment, layout, power, and time required to stabilize a soil are governed by the size of the project and the type of soil. In general,

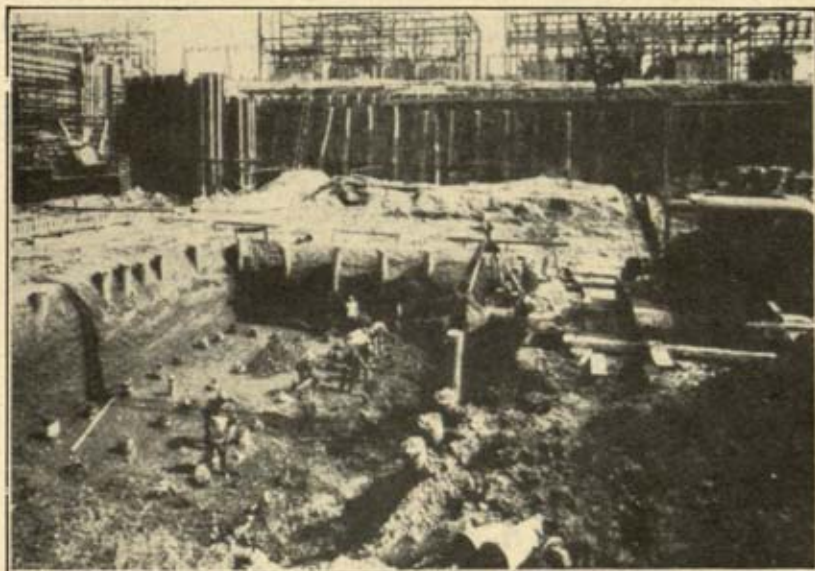


Fig. 16-17. Soil stabilized by electroosmosis system. (Griffin Wellpoint Corp.)

electroosmosis is applicable to soils having a grain size finer than that for which wellpoints normally are adaptable. The cost of this system is comparable with the cost of a wellpoint system if the latter were being used to unwater an area of similar size in a sand.

Several projects have been stabilized with electroosmosis in Europe and the United States. At the John C. Weadock power plant at Bay City, Mich., an area to be excavated was enclosed by steel-sheet piling. After the excavation had been carried down about 20 ft, the sheet piling started to move, which forced a work stoppage. An electroosmosis system was installed, and the project was completed without difficulty. The soil was a sandy silt and a clayey silt. Tests indicated that the shearing strength of the soil was increased 300 per cent by the use of electroosmosis.



Laboratory or small-scale field tests are made to determine whether electroosmosis is applicable to a soil. From these tests, the practicability of using this method of stabilization, the over-all cost of the project, and the time required for stabilization are determined.

Figure 16-17 shows excavation operation for the Wheadoek power plant after the electroosmosis system had been installed.

### PROBLEMS

**16-1.** A circular-type cofferdam, using steel-sheet piling 16 in. wide, is to be installed to completely enclose an area in water. The dam, which will be elliptical in plan, will have a perimeter of at least 810 ft, measured along the centers of the cells. The effective height of the dam will be 40 ft.

Determine the following for the dam:

- The minimum theoretical width  $E$
- The width for use in Table 16-2
- The diameter of the cells
- The number of cells in the dam
- The total number of standard and T piles required for the dam
- The volume of earth required to fill the dam to full height

**16-2.** Select the most economical height for a steel-sheet-piling cofferdam, considering the cost of the dam and the cost of risk of flooding the project inside the dam. It is estimated that any flood that overtops the dam will cause damages equal to \$20,000.

The period of construction inside the dam will be 6 months. A study of the flow of the river indicates that flood stages of the specified heights will occur as frequently as given below:

Height of dam, ft	Flood frequency	Extra cost of dam over previous height
32	3 per yr	\$ 0
34	2 per yr	8,000
36	1 per yr	8,000
38	1 per 2 yr	9,000
40	1 per 4 yr	10,000

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## CHAPTER 17

### THE PRODUCTION OF CRUSHED-STONE AGGREGATE

**Introduction.** The production of crushed-stone aggregate involves drilling, blasting, loading, transporting, crushing, screening, handling, and storing the aggregate. As the first four operations have already been discussed, this chapter will be devoted to studies of the last four operations.

In operating a quarry and a crushing plant, the drilling pattern, the amount of explosives, the size power shovel to load the stone, and the size of the primary crusher should be coordinated to assure that all stone from the quarry can pass through the opening to the crusher. It is desirable for the loading capacity of the shovel and the capacity of the crushing

TABLE 17-1. RECOMMENDED MINIMUM SIZES OF PRIMARY CRUSHERS FOR USE WITH SHOVEL DIPPERS OF THE INDICATED CAPACITIES

Capacity of dipper, cu yd	Jaw crusher, in.*	Gyratory crusher, size of opening,† in.
$\frac{3}{4}$	28 × 36	16
1	28 × 36	16
$1\frac{1}{2}$	36 × 42	20
$1\frac{3}{4}$	42 × 48	26
2	42 × 48	30
$2\frac{1}{2}$	48 × 60	36
3	48 × 60	42
$3\frac{1}{2}$	48 × 60	42
4	56 × 72	48
5	66 × 86	60

\* The first two digits are the width of the opening at the top of the crusher, measured perpendicular to the jaw plates. The second two digits are the width of the opening, measured across the jaw plates.

† The recommended sizes are for gyratory crushers equipped with straight concaves.

plant to be approximately equal. Table 17-1 gives the recommended minimum sizes of jaw and gyratory crushers required to handle the stone passing through power-shovel dippers of the specified capacities.

**Types of Crushers.** Crushers may be classified according to the stage of crushing which they accomplish, such as primary, secondary, tertiary, etc. A primary crusher receives the stone directly from a quarry and



produces the first reduction in size. The output of the primary crusher is fed to a secondary crusher, which further reduces the size. Some of the stone may pass through four or more crushers before it is reduced to the necessary fineness.

While there is no rigid classification of crushers, the following is representative of common crusher uses.

1. Primary crushers
  - a. Jaw
  - b. Gyratory
  - c. Hammer mill
2. Secondary crushers
  - a. Cone
  - b. Roll
  - c. Hammer mill
3. Tertiary crushers
  - a. Roll
  - b. Rod mill
  - c. Ball mill

As stone passes through a crusher, it undergoes a reduction in size, which may be expressed as a ratio of reduction. The ratio of reduction is the ratio of the distance between the fixed and moving faces at the top divided by the distance at the bottom of a crusher. Thus, if the distance between the two faces of a jaw crusher at the top is 16 in. and at the bottom is 4 in., the ratio of reduction is 4.

The ratio of reduction for a roll crusher is the ratio of the dimension of the largest stone that can be nipped by the rolls divided by the setting of the rolls, which is the smallest distance between the faces of the rolls.

**Jaw Crushers.** This machine is very popular as a primary crusher. It operates by allowing stone to flow into the space between two jaws, one of which is stationary, while the other is movable. The distance between the jaws diminishes as the stone travels downward under the effect of gravity and the movable jaw, until it ultimately passes through the lower opening. The movable jaw is capable of exerting a pressure sufficiently high to crush the hardest rock.

The Blake type, illustrated in Fig. 17-1, is a double-toggle crusher. The movable jaw is suspended from a shaft mounted on bearings on the crusher frame. The crushing operation is effected by rotating an eccentric shaft, which raises and lowers the pitman, which actuates the two toggles. As the two toggles are raised by the pitman, a high pressure is exerted near the bottom of the swing jaw, which partially closes the opening between the bottoms of the two jaws. This operation is repeated as the eccentric shaft is rotated.

The jaw plates, which are made of manganese steel, may be removed, replaced, or, in some cases, reversed. The jaws may be smooth, or, in the event the stone tends to break into slabs, corrugated jaws may be used to reduce the slabbing. The swing jaw may be straight, or it may be curved to reduce the danger of choking.

When the eccentric shaft of the single-toggle crusher, illustrated in Fig. 17-2, is rotated, it gives the movable jaw a vertical and horizontal motion. This type crusher is used quite frequently in portable rock-crushing plants because of its compact size, light weight, and reasonably

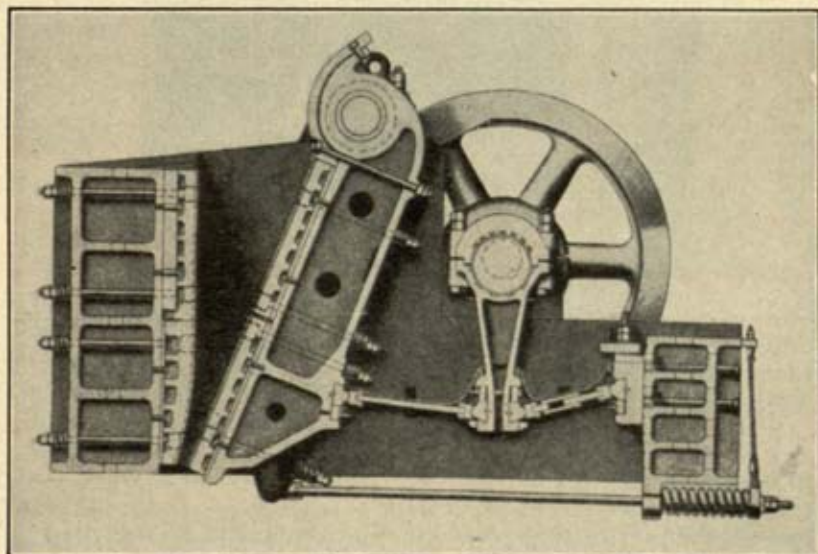


FIG. 17-1. Blake-type jaw crusher. (Allis-Chalmers Mfg. Co.)

sturdy construction. The capacity of the single-toggle crusher is usually less than that of the Blake-type unit.

When a jaw crusher is used as a primary crusher, the size may be determined by the capacity of the shovel dipper, as indicated in Table 17-1, in which case the capacity of the crusher may be secondary. A jaw crusher should have a top opening at least 2 in. wider than the largest stones that will be fed to it.

Table 17-2 gives representative capacities for various sizes of jaw crushers. As the setting may be based on the open or closed position of the bottom of the swing jaw, a capacity table should specify which setting applies. The closed position is most commonly used and is the basis for the values given in Table 17-2. The capacity is given in tons per hour for stone weighing 100 lb per cu ft when crushed.



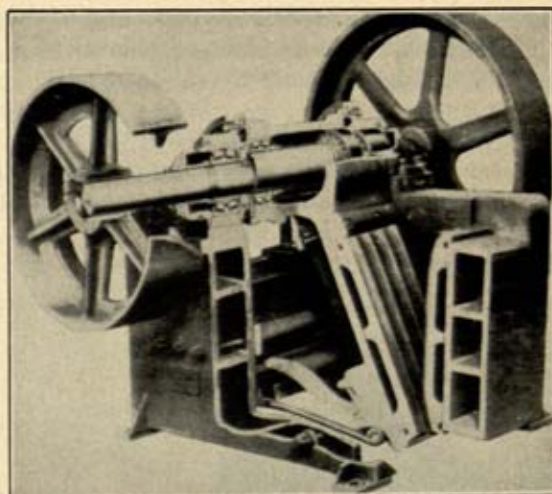


FIG. 17-2. Toggle-type jaw crusher. (Pioneer Engineering Works.)

TABLE 17-2. REPRESENTATIVE CAPACITIES OF BLAKE-TYPE JAW CRUSHERS, IN TONS OF STONE PER HOUR

Size crusher, in.*	Max rpm	Max hp	Closed setting of discharge opening, in.										
			1	1½	2	2½	3	4	5	6	7	8	9
10 × 16	300	15	11	16	20								
10 × 20	300	20	14	20	25	34							
15 × 24	275	30	..	27	34	42	50						
15 × 30	275	40	..	33	43	53	62						
18 × 36	250	60	..	46	61	77	93	125					
24 × 36	250	75	..	..	77	95	114	150					
30 × 42	200	100	..	..	..	125	150	200	250	300			
36 × 42	175	115	..	..	..	140	160	200	250	300			
36 × 48	160	125	..	..	..	150	175	225	275	325	375		
42 × 48	150	150	..	..	..	165	190	250	300	350	400	450	
48 × 60	120	180	..	..	..	..	220	280	340	400	450	500	550
56 × 72	95	250	..	..	..	..	..	315	380	450	515	580	640

\* The first two digits indicate the width of the feed opening, while the second two digits indicate the width of the jaw plates.

**Gyratory Crushers.** A section through a gyratory crusher is illustrated in Fig. 17-3. The crusher unit consists of a heavy cast-iron or steel frame, with an eccentric shaft setting and driving gears in the lower part of the unit. In the upper part there is a cone-shaped crushing chamber, lined with hard-steel or manganese-steel plates called the concaves. The

crushing member includes a hard-steel crushing head mounted on a vertical steel shaft. This shaft and head are suspended from the spider at the top of the frame, which is so constructed that some vertical adjustment of the shaft is possible. The eccentric support at the bottom causes the shaft and the crushing head to gyrate as the shaft rotates, thereby varying the width of the space between the concaves and the head. As the rock which is fed in at the top of the crushing chamber moves downward, it undergoes a reduction in size until it finally passes through the opening at the bottom of the chamber.

The size of a gyratory crusher is the width of the receiving opening, measured between the concaves and the crusher head. The setting is the width of the bottom opening and may be the open or closed dimension. When a setting is given, it should be specified if it is the open or closed dimension.

The ratio of reduction for gyratory crushers usually varies from about 5.5 to 7.5, with an average value around 6.5 for the sizes up to 42 in.

If a gyratory crusher is used as a primary crusher, the size selected may be dictated by the size of the rock from the quarry or it may be dictated by a desired capacity. When this machine is used as a secondary crusher, the capacity usually will govern the size selected. The capacity of a gyratory crusher may be increased by increasing the speed of the machine within reasonable limits.

Table 17-3 gives representative capacities of gyratory crushers, expressed in tons per hour, based on a continuous feed of stone weighing 100 lb per cu ft when crushed. The crushers with straight concaves are commonly used as primary crushers, while those with nonchoking concaves are commonly used as secondary crushers.

**Cone Crushers.** Cone, or reduction, crushers are used as secondary or tertiary crushers. They are capable of producing large quantities of uniformly finely crushed stone. A cone crusher differs from a gyratory crusher in the following respects:

1. It has a shorter cone.
2. It has a smaller receiving opening.

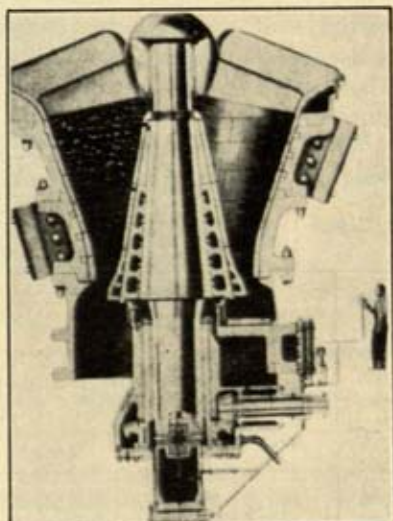


FIG. 17-3. Hydroset gyratory crusher. (Allis-Chalmers Mfg. Co.)



3. It rotates at a higher speed, from 430 to 580 rpm.

4. It produces a more uniformly sized stone with the maximum size equal to the width of the closed-side setting.

Figure 17-4 shows a section through a Symons standard cone crusher. The conical head, made usually of manganese steel and mounted on the vertical shaft, serves as one of the crushing surfaces. The other surface is the concave, which is attached to the upper part of the crusher frame. The bottom of the shaft is set in an eccentric bushing to produce the gyratory effect as the shaft rotates.

While the maximum diameter of the crusher head may be used to designate the size of a cone crusher, the size of the feed opening, which

TABLE 17-3. REPRESENTATIVE CAPACITIES OF GYRATORY CRUSHERS, IN TONS OF STONE PER HOUR

Size of crusher, in.	Counter-shaft speed, rpm	Approx. power required, hp	Open-side setting of crusher, in.															
			1½	1¾	2	2¼	2½	2¾	3	3½	4	4½	5	5½	6	6½	7	
Straight concaves																		
8	450	15-25	30	36	41	47												
10	400	25-40	..	40	50	60												
13	375	50-75	..	..	..	85	100	120	133									
16	350	60-100	..	..	..	..	..	..	160	185	210							
20	330	75-125	..	..	..	..	..	..	..	200	230	255						
30	325	125-175	..	..	..	..	..	..	..	..	310	350	390					
42	300	200-275	..	..	..	..	..	..	..	..	..	..	500	570	630	700		
54	250	225-300	..	..	..	..	..	..	..	..	..	..	..	..	675	730	785	
Modified straight concaves																		
8	450	15-25	35	40	45													
10	400	25-40	..	54	60	65												
13	375	50-75	..	..	..	..	95	112	130									
16	350	60-100	..	..	..	..	..	..	150	172	195							
20	330	75-125	..	..	..	..	..	..	..	182	200	220						
30	325	125-175	..	..	..	..	..	..	..	..	340	370	400					
42	300	200-275	..	..	..	..	..	..	..	..	..	..	607	650	690			
Nonchoking concaves																		
8	450	15-25	42	46														
10	400	25-40	51	57	63	69												
13	375	50-75	79	87	95	103	111											
16	350	60-100	..	..	107	118	128	140	150									
20	330	75-125	..	..	..	155	169	184	198	220	258	285	310					

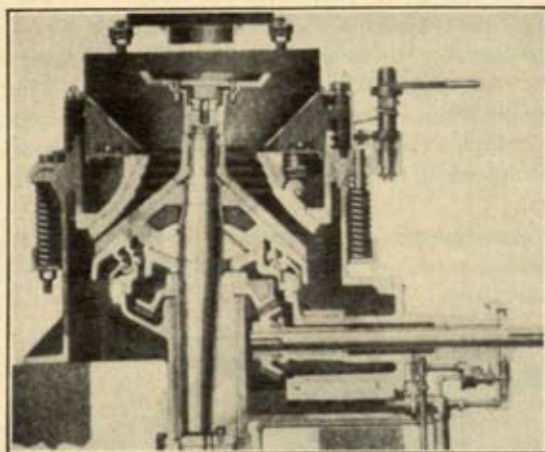


FIG. 17-4. Symons standard cone crusher. (Nordberg Mfg. Co.)

TABLE 17-4. REPRESENTATIVE CAPACITIES FOR SYMONS STANDARD CONE CRUSHERS, IN TONS OF STONE PER HOUR\*

Size of crusher, ft	Full-load speed, rpm	Power required, hp	Size feed opening, in.	Min discharge setting, in.	Discharge setting, in.											
					1/4	3/8	1/2	5/8	3/4	7/8	1	1 1/4	1 1/2	2	2 1/2	
2	575	25-30	2 1/4	1/4	15	20	25	30	35							
			3 1/4	3/8		20	25	30	35	40	45	50	60			
3	580	50-60	3 7/8	3/8		35	40	55	70	75						
			5 1/8	1/2			40	55	70	75	80	85	90	95		
4	485	75-100	5	3/8		60	80	100	120	135	150					
			7 3/8	3/4					120	135	150	170	177	185		
4 1/4	485	125-150	4 1/2	1/2			100	125	140	150						
			7 3/8	5/8				125	140	150	160	175				
			9 1/2	3/4				140	150	160	175	185	190			
5 1/2	435	150-200	7 3/8	5/8				160	200	235	275					
			8 5/8	7/8					235	275	300	340	375	450		
			9 7/8	1						275	300	340	375	450		
7	435	250-300	10	3/4					330	390	450	560	600			
			11 1/2	1							450	560	600	800		
			13 1/2	1 1/4								560	600	800	900	

\* Courtesy Nordberg Manufacturing Company.



limits the size of rocks that may be fed to the crusher, is the width of the opening at the entrance to the crushing chamber. The magnitude of the eccentric throw and the setting of the discharge opening may be varied within reasonable limits. Because of the high speed of rotation all particles passing through a crusher will be reduced to sizes no larger than the close-size setting, which should be used to designate the size of the discharge opening.

Table 17-4 gives representative capacities for the Symons standard cone crusher, expressed in tons of stone per hour for material weighing 100 lb per cu ft when crushed.

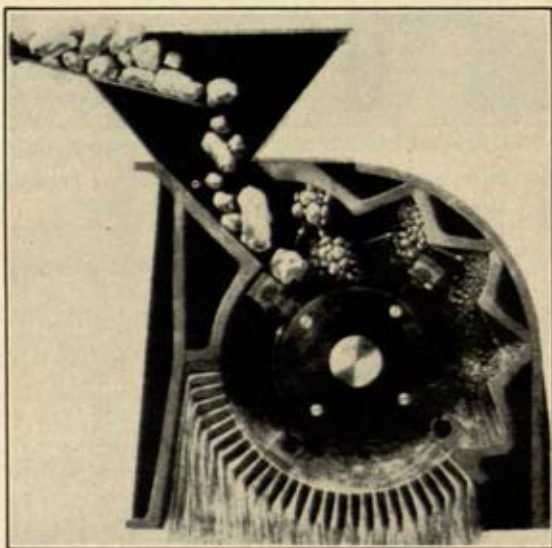


FIG. 17-5. Hammer-mill rock crusher. (*Allis-Chalmers Mfg. Co.*)

**Hammer Mills.** The hammer mill, which is the most widely used impact crusher, may be used for primary or secondary crushing. The basic parts of a unit include a housing frame, a horizontal shaft extending through the housing, a number of arms and hammers attached to a spool which is mounted on the shaft, one or more manganese-steel or other hard-steel breaker plates, and a series of grate bars whose spacings may be adjusted to regulate the width of openings through which the crushed stone flows. These parts are illustrated in the section through the crusher shown in Fig. 17-5.

As the stone to be crushed is fed to the mill, the hammers, which travel at a high speed, strike the particles, breaking them and driving them against the breaker plates, which further reduce their sizes.

The size of a hammer mill may be designated by the size of the feed opening. The capacity will vary with the size of the unit, the kind of stone crushed, the size of the material fed to the mill, and the speed of the

TABLE 17-5. REPRESENTATIVE CAPACITIES FOR HAMMER MILLS, IN TONS OF STONE PER HOUR\*

Size feed opening, in.	Size feed, in.	Shaft speed, rpm	Power required, hp	Width of openings between grate bars, in.						
				1/8	3/16	1/4	3/8	1/2	1	1 1/4
6 1/4 × 9	3	1,800	15-20	2 1/2	3 1/2	5	8	10		
12 × 15	3	1,500	50-60	9	13	17	23	29	36	39
15 × 25	6	900	100-125	18	25	31	40	47	65	70
15 × 37	6	900	150-200	27	37	47	60	71	97	105
15 × 49	6	900	200-250	36	50	63	80	95	130	140

\* Courtesy Allis-Chalmers Manufacturing Company.

shaft. Table 17-5 gives representative capacities of hammer mills expressed in tons of stone per hour for material weighing 100 lb per cu ft when crushed.

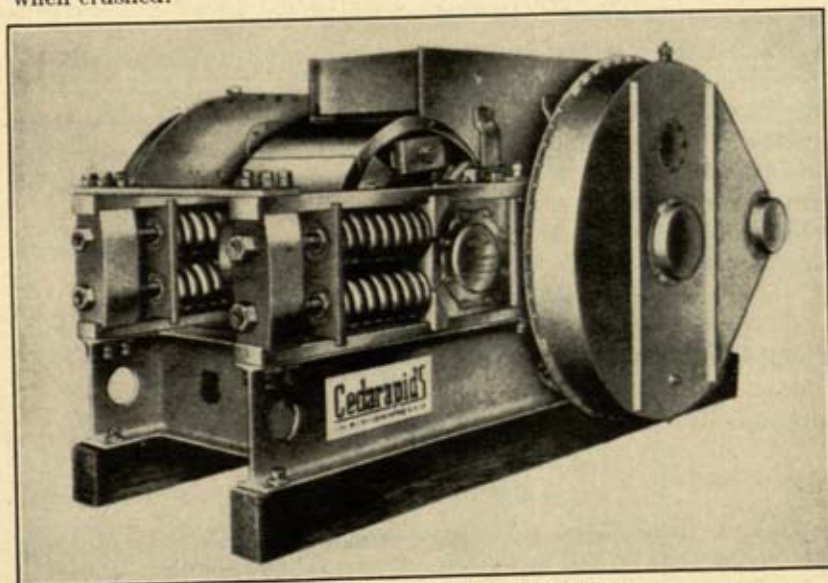


FIG. 17-6. Roll crusher. (Iowa Mfg. Co.)

**Roll Crushers.** Roll crushers are used for producing additional reductions in the sizes of stone after the output of a quarry has been subjected to one or more stages of prior crushing. A roll crusher consists of a



heavy cast-iron frame equipped with two hard-steel rolls, each mounted on a separate horizontal shaft. Most crushers are so constructed that each roll is driven independently by a flat-belt pulley or a V-belt sheave. One of the rolls is mounted on a slide frame, to permit an adjustment in the width of the discharge opening between the two rolls. The movable roll is spring-loaded to provide safety against damage to the rolls when trap iron or other noncrushable material passes through the machine. Figure 17-6 illustrates a roll crusher.

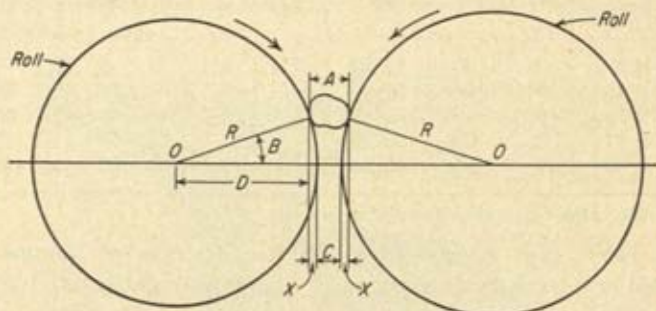


FIG. 17-7. Crushing rock between two rolls.

The maximum size of material that may be fed to a crusher is directly proportional to the diameter of the rolls. If the feed contains stones that are too large, the rolls will not grip them and pull them through the crusher. The angle of nip in Fig. 17-7, which is constant for smooth rolls, has been found to be  $16^{\circ}45'$ . The maximum-size particles that can be crushed is determined as follows:

Let  $R$  = radius of rolls

$B$  = angle of nip

$D = R \cos B = 0.9575R$

$A$  = maximum-size feed

$C$  = roll setting = size of finished product

$$\begin{aligned} X &= R - D \\ &= R - 0.9575R = 0.0425R \\ A &= 2X + C \\ &= 0.085R + C \end{aligned} \quad (17-1)$$

**EXAMPLE.** Determine the maximum-size stone that may be fed to a smooth-roll crusher whose rolls are 40 in. in diameter, when the roll setting is 1 in.

$$\begin{aligned} A &= 0.085 \times 20 + 1 \\ &= 2.7 \text{ in.} \end{aligned}$$

The capacity of a roll crusher will vary with the kind of stone, size of feed, size of the finished product, width of rolls, speed at which the rolls

rotate, and extent to which the stone is fed uniformly into the crusher. Referring to Fig. 17-7, the theoretical volume of a solid ribbon of material passing between the two rolls in 1 min would be the product of the width of the opening times the width of the rolls times the speed of the surface of the rolls. The volume may be expressed in cubic inches or cfm. In actual practice the ribbon of crushed stone will never be solid. A more realistic volume should approximate one-fourth to one-third of the theoretical volume. A formula which may be used as a guide in estimating the capacity is derived as follows:

Let  $C$  = distance between rolls, in.

$W$  = width of rolls, in.

$S$  = peripheral speed of rolls, in. per min

$N$  = speed of rolls, rpm

$R$  = radius of rolls, in.

$V_1$  = theoretical volume, cu in. or cfm

$V_2$  = actual volume, cu in. or cfm

$Q$  = probable capacity, tons per hr

$$V_1 = CWS$$

Assume  $V_2 = V_1/3$ .

$$V_2 = \frac{CWS}{3} \quad \text{cu. in. per min}$$

Divide by 1,728 cu in. per cu ft.

$$V_2 = \frac{CWS}{5,184} \quad \text{cfm}$$

Assume the crushed stone weighs 100 lb per cu ft.

$$\begin{aligned} Q &= \frac{100 \times 60V_2}{2,000} = 3V_2 \\ &= \frac{CWS}{1,728} \quad \text{tons per hr} \end{aligned} \quad (17-2)$$

$S$  may be expressed in terms of the diameter of the roll and the speed in rpm.

$$S = 2\pi RN$$

Substituting this value of  $S$  in formula (17-2) gives

$$Q = \frac{CW\pi RN}{864} \quad (17-3)$$

Table 17-6 gives representative capacities for smooth-roll crushers, expressed in tons of stone per hour for material weighing 100 lb per cu ft



when crushed. These capacities should be used as a guide only in estimating the probable output of a crusher. The actual capacity may be more or less than the given values.

TABLE 17-6. REPRESENTATIVE CAPACITIES OF SMOOTH-ROLL CRUSHERS, IN TONS OF STONE PER HOUR\*

Size crusher,† in.	Speed, rpm	Power required, hp	Width of opening between rolls, in.						
			¼	½	¾	1	1½	2	2½
16 × 16	120	15-30	15	30	40	55	85	115	140
24 × 16	80	20-35	15	30	40	55	85	115	140
30 × 18	60	50-70	15	30	45	65	95	125	155
30 × 22	60	60-100	20	40	55	75	115	155	190
40 × 20	50	60-100	20	35	50	70	105	135	175
40 × 24	50	60-100	20	40	60	85	125	165	210
54 × 24	41	125-150	24	48	71	95	144	192	240

\* Courtesy Iowa Manufacturing Company.

† The first two digits indicate the diameters of the rolls, and the last two digits indicate the widths of the rolls.

If a roll crusher is producing a finished aggregate, the reduction ratio should not be greater than 4:1. However, if a roll crusher is used to prepare feed for a fine grinder, the reduction may be as high as 7:1.

**Rod and Ball Mills.** These mills are used to produce fine aggregate, such as sand, from stone that has been crushed to suitable sizes by other crushing equipment. It is not uncommon for specifications for concrete to require the use of a homogeneous aggregate, regardless of size. If crushed stone is used for the coarse aggregate, sand manufactured from the same stone will satisfy the specifications.

A rod mill is a circular steel shell, lined on the inside with a hard mineral wearing surface, equipped with a suitable support or trunnion arrangement at each end, with a driving gear at one end. It is operated with its axis in a horizontal position. It is charged with steel rods, whose lengths are slightly less than the length of the mill. Crushed stone, which is fed through the trunnion at one end of the mill, flows to the discharge at the other end. As the mill rotates slowly, the stone is constantly subjected to the impact of the tumbling rods, which produce the desired grinding. A mill may be operated wet or dry, with or without water added. The size of a rod mill is specified by the diameter and the length of the shell, such as 8 by 12 ft, respectively. Figure 17-8 shows a section through a rod mill.

A ball mill, which uses steel balls instead of rods to supply the impact necessary to grind the stone, will produce fines with smaller grain sizes

than those produced by a rod mill. Figure 17-9 shows a section through a ball mill.

**Sizes of Stone Produced by Jaw and Roll Crushers.** While the setting of the discharge opening of a crusher will determine the maximum size stone produced, the aggregate sizes will range from slightly greater than

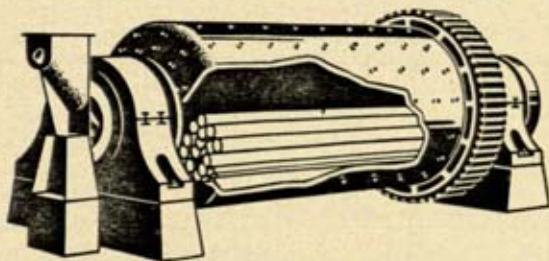


FIG. 17-8. Section through a rod mill. (Hardinge Co., Inc.)

the crusher setting to fine dust. Experience gained in the crushing industry indicates that for any given setting for a jaw or roll crusher approximately 15 per cent of the total amount of stone passing through the crusher will be larger than the setting. If the openings of a screen which receives the output from such a crusher are the same size as the

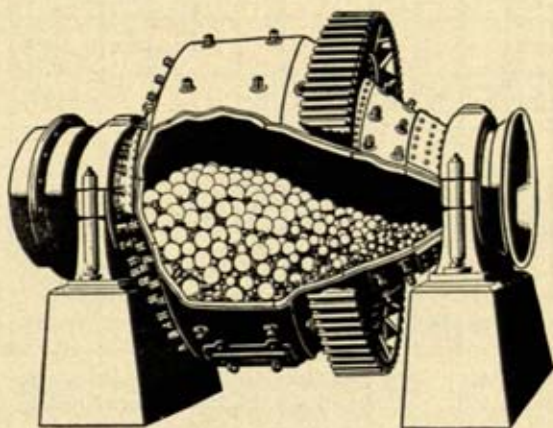


FIG. 17-9. Section through a conical ball mill. (Hardinge Co., Inc.)

crusher setting, 15 per cent of the output will not pass through the screen. Figure 17-10 gives representative values for the per cent of crushed stone passing through or retained on screens having various sizes of openings for different crusher settings. The information in this figure will apply to jaw and roll crushers only.



## PERCENTAGE CHART

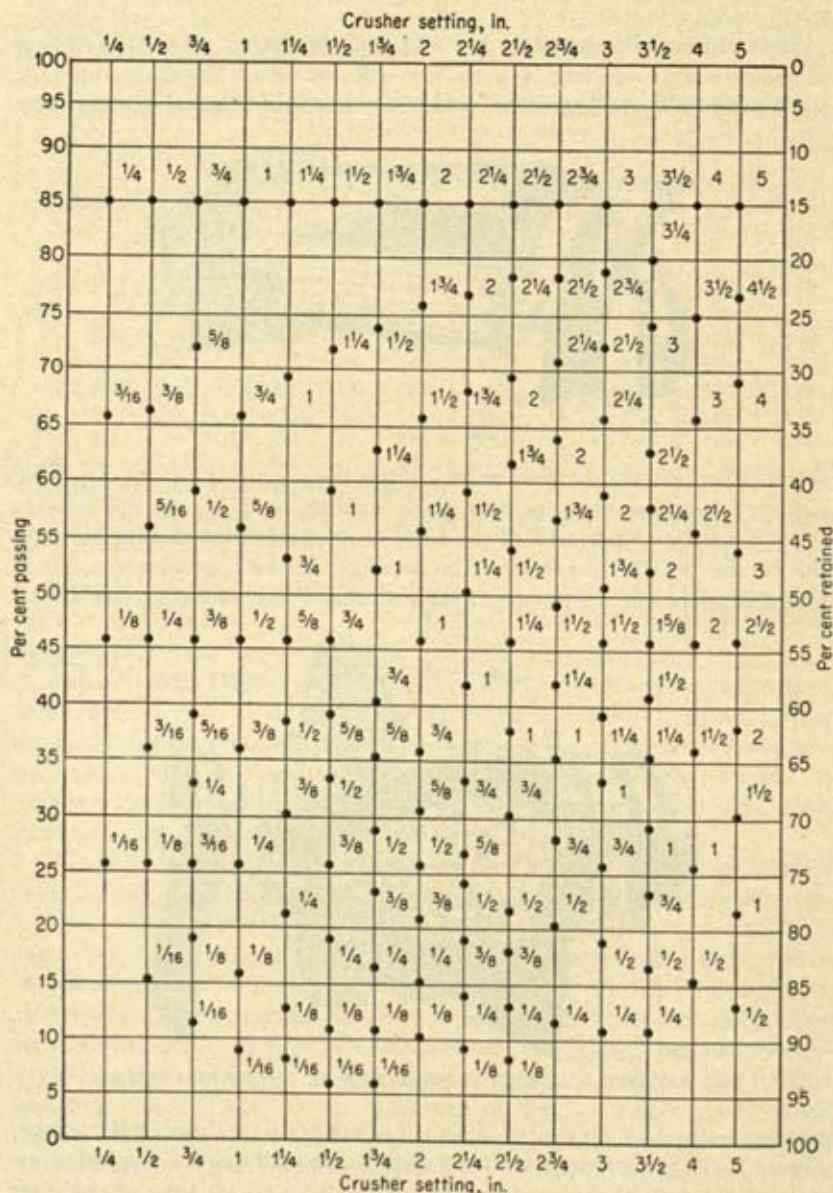


FIG. 17-10. Analysis of the size of aggregate produced by jaw and roll crushers. (Universal Engineering Corp.)

In operating a crusher, it generally is desirable to know how the product varies in sizes from the maximum to the minimum sizes. The chart in Fig. 17-10 gives the per cent of material passing or retained on screens having the size openings indicated. The chart can be applied to both jaw- and roll-type crushers. To read the chart, select the vertical line corresponding to the crusher setting. Then go down this line to the number which indicates the size of the screen opening. From the size of the screen opening proceed horizontally to the left to determine the per cent of material passing through the screen or to the right to determine the per cent of material retained on the screen.

**EXAMPLE.** A jaw crusher, with a closed setting of 3 in., produces 50 tons per hr of crushed stone. Determine the amount of stone produced in tons per hour within the following size ranges: in excess of 2 in.; between 2 and 1 in.; between 1 and  $\frac{1}{4}$  in.; less than  $\frac{1}{4}$  in.

From Fig. 17-10 the amount retained on a 2-in. screen is 42 per cent of 50 = 21 tons per hr. The amount in each of the sizes ranges is determined as follows:

Size range, in.	Per cent passing screens	Per cent in size range	Total output of crusher, tons per hr	Amount produced in size range, tons per hr
Over 2	100-58	42	50	21.0
2-1	58-33	25	50	12.5
1- $\frac{1}{4}$	33-11	22	50	11.0
$\frac{1}{4}$ -0	11-0	11	50	5.5
Total	.....	100	..	50.0

**Selecting Crushing Equipment.** In selecting crushing and screening equipment it is essential that certain information be known prior to making the selection. The information needed should include, but will not necessarily be limited to, the following items:

1. The kind of stone to be crushed
2. The maximum size and perhaps the size ranges of the feed to the plant
3. The method of feeding the crushers
4. The required capacity of the plant
5. The per cent of material falling within specified size ranges

The following example will illustrate a method which may be used to select crushing equipment:

**EXAMPLE.** Select a primary and a secondary crusher to produce 100 tons per hr of crushed limestone. The maximum-size stones from the quarry will be 16 in. The quarry stone will be hauled by truck, dumped into a surge bin, and fed to the primary crusher by an apron feeder, which will maintain a reasonably uniform rate of feed.



The aggregate will be used on a project whose specifications require the following size distributions:

Size screen opening, in.		Per cent
Passing	Retained on	
1½		100
1½	¾	42-48
¾	¼	30-36
¼	0	20-26

Consider a jaw crusher for the primary and a roll crusher for the secondary crushing. The output of the jaw crusher will be screened to remove specification sizes before the oversize material is fed to the roll crusher.

Assume a setting of 3 in. for the jaw crusher. This will give a ratio of reduction of approximately 5:1, which is satisfactory. Table 17-2 indicates a size 24- by 36-in. crusher with a probable capacity of 114 tons per hr. Figure 17-10 indicates that the product of the crusher will be distributed by sizes as follows:

Size range, in.	Per cent passing screens	Per cent in size range	Total output of crusher, tons per hr	Amount produced in size range, tons per hr
Over 1½	100-46	54	100	54.0
1½-¾	46-26	20	100	20.0
¾-¼	26-11	15	100	15.0
¼-0	11-0	11	100	11.0
Total	.....	100	...	100.0

As the roll crusher will receive the output from the jaw crusher, the rolls must be large enough to handle 3-in. stone. Assume a setting of 1½ in. From formula (17-1) the minimum radius will be 17.7 in. Try a 40- by 20-in. crusher with a capacity of approximately 105 tons per hr for a 1½-in. setting.

For any given setting the crusher will produce about 15 per cent stone having at least one dimension larger than the setting. Thus, for a given setting 15 per cent of the stone that passes through the roll crusher will be returned for recrushing. The total amount of stone passing through the crusher, including the returned stone, is determined as follows:

Let  $Q$  = total amount of stone through the crusher

$$\begin{aligned}
 \text{Then} \quad & 0.15Q = \text{amount of returned stone} \\
 & 0.85Q = \text{amount of new stone} \\
 & Q = \frac{\text{amount of new stone}}{0.85} \\
 & = \frac{54}{0.85} = 63.5 \text{ tons per hr}
 \end{aligned}$$

The 40- by 20-in. crusher will handle this amount of stone easily. The distribution of the output of this crusher by size range will be as follows:

Size range, in.	Per cent passing screens	Per cent in size range	Total amount through crusher, tons per hr	Amount produced in size range, tons per hr
1½-¾	85-46	39	63.5	24.8
¾-¼	46-18	28	63.5	17.8
¼-0	18-0	18	63.5	11.4
Total	.....	85	.....	54.0

Now combine the output of each crusher by specified sizes.

Size range, in.	From jaw crusher, tons per hr	From roll crusher, tons per hr	Total amount, tons per hr	Per cent in size range
1½-¾	20.0	24.8	44.8	44.8
¾-¼	15.0	17.8	32.8	32.8
¼-0	11.0	11.4	22.4	22.4
Total	46.0	54.0	100.0	100.0

**Scalping Crushed Stone.** The term scalping, as used in this chapter, refers to a screening operation which is performed to remove from the main mass of stone to be processed that stone which is too large for the crusher opening or is small enough to be used without further crushing. Scalping may be performed ahead of a primary crusher, and it represents good crushing practice to scalp all crushed stone following each successive stage of reduction.

Scalping ahead of a primary crusher serves two purposes. The use of a grizzly, which consists of a number of widely spaced parallel bars, will prevent oversize stones from entering the crusher and blocking the opening. If the product of the quarry contains such stones, it is desirable to remove them ahead of the crusher.

The product of the quarry may contain dirt, mud, or other debris which is not acceptable in the finished product and therefore must be removed from the stone. Scalping should accomplish this removal. Also, the product of the quarry may contain an appreciable amount of stone which was reduced by the blasting operation to specification sizes. In this event it may be good economy to remove such stone ahead of the primary crusher, thereby reducing the total load on the crusher and increasing the over-all capacity of the plant. Figure 17-11 illustrates a commercial bar grizzly.



It usually is good practice and economical to install a scalper after each stage of reduction to remove specification sizes. This stone may be transported to grading screens, where it can be sized and placed in appropriate storage. Any stone removed ahead of a crusher will reduce the

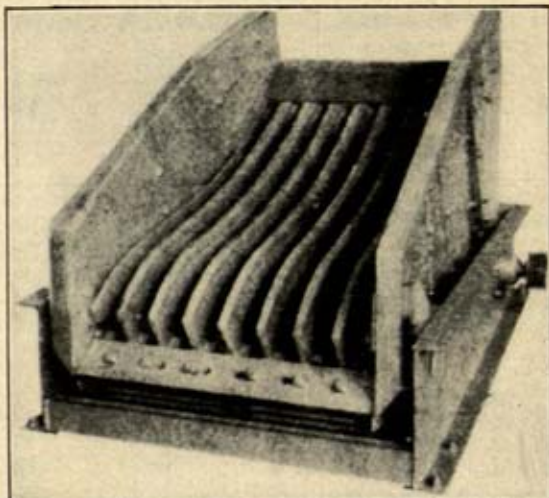


FIG. 17-11. Bar grizzly. (Nordberg Manufacturing Co.)

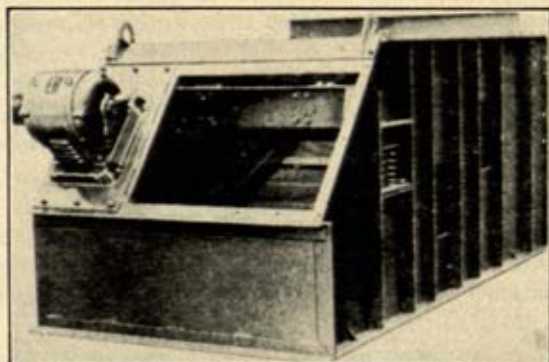


FIG. 17-12. Rod grizzly. (Nordberg Manufacturing Co.)

total load on the crusher, which will permit the use of a smaller crusher or an increase in the output of the plant.

Figure 17-12 illustrates a rod-type scalping unit.

**Feeders.** The capacity of a crusher will be increased if the stone is fed to it at a uniform rate. Surge feeding tends to overload a crusher, and then the surge is followed by an insufficient supply of stone. This type

of feeding, which reduces the capacity of a crusher, may be eliminated by using a mechanical feeder ahead of a crusher. The installation of such a feeder may increase the capacity of a jaw crusher as much as 15 per cent. An apron-type feeder, as illustrated in Fig. 17-13, is suitable for use ahead of a primary crusher.

**Surge Piles.** A stationary stone-crushing plant may include several types and sizes of crushers, each probably followed with a set of screens and a belt conveyor to transport the stone to the next crushing operation or to storage. A plant may be designed to provide temporary storage for stone between the successive stages of crushing. This plan has the

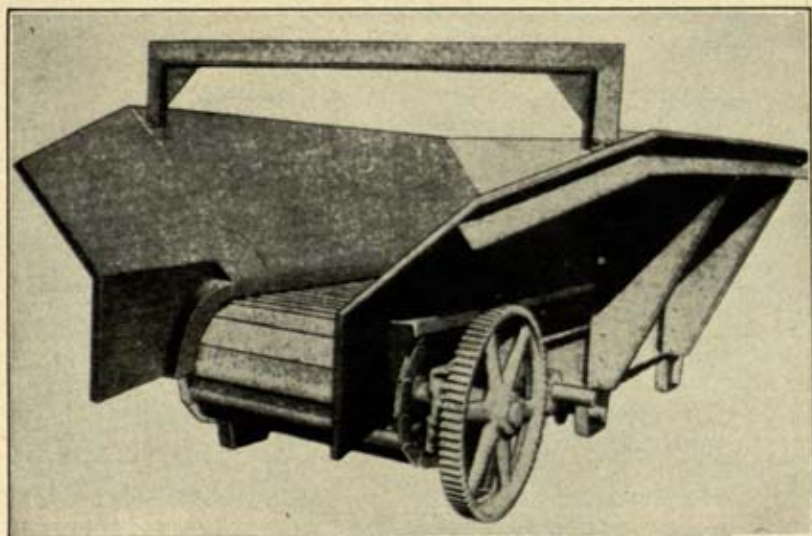


FIG. 17-13. Apron-type feeder. (*Universal Engineering Corp.*)

advantage of eliminating or reducing the surge effect that frequently exists when the crushing, screening, and handling operations are conducted on a straight-line basis. The stone in temporary storage, which is referred to as a surge pile, ahead of a crusher may be used to keep at least a portion of a plant in operation. Within reasonable limits the use of a surge pile ahead of a crusher permits the crusher to be fed uniformly at the most satisfactory rate, regardless of variations in the output of other equipment ahead of the crusher. The use of surge piles has enabled some plants to increase the production by as much as 10 to 20 per cent.

Among the arguments against the use of surge piles are the following:

1. They require additional area for storage.
2. They require the construction of storage bins or reclaiming tunnels.
3. They increase the amount of handling of stone.



The decision to use or not use surge piles should be based on the advantages and disadvantages for each plant.

**Screening Aggregate.** Screening of crushed stone is necessary in order that the aggregate may be separated by size ranges. Most specifications covering the use of aggregate stipulate that the different sizes shall be combined to produce a blend having a given size distribution. Persons who are responsible for preparing the specifications for the use of aggregate realize that crushing and screening cannot be done with complete

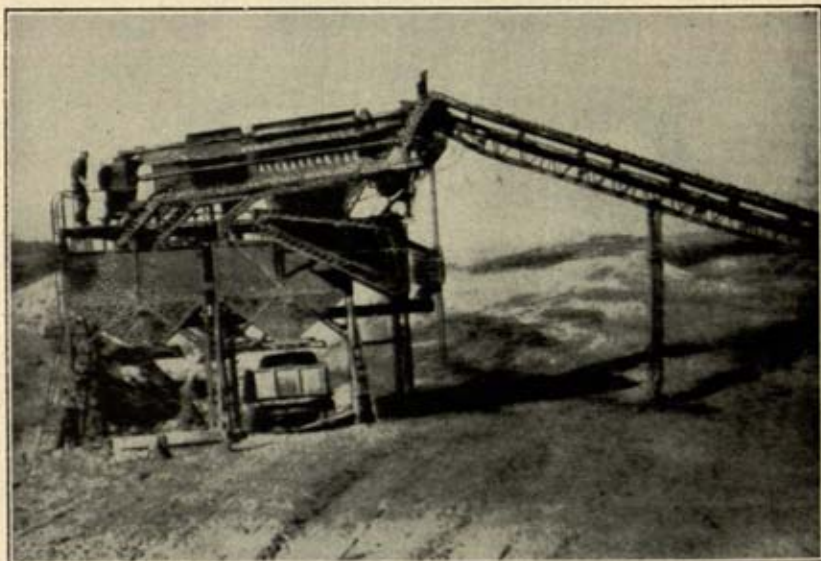


FIG. 17-14. Revolving screen with scrubber in operation. (*Pioneer Engineering Works.*)

precision, and, accordingly, they allow some tolerance in the size distribution. The extent of tolerance may be indicated by a statement such as that the quantity of aggregate passing a 1-in. screen and retained on a  $\frac{1}{4}$ -in. screen shall be not less than 30 or more than 40 per cent of the total quantity of aggregate.

**Revolving Screens.** Revolving screens have several advantages over other types of screens, especially when they are used to wash and screen sand and gravel. The operating action is slow and simple, and the maintenance and repair costs are low. If the aggregate to be washed contains silt and clay, a scrubber can be installed near the entrance end of a screen in order that the material may be agitated in water. At the same time streams of water may be sprayed on the aggregate as it moves through the screen. Figure 17-14 shows a revolving screen with a

scrubber in operation. The aggregate, which is separated by sizes, is stored temporarily in the bins below the screen.

**Vibrating Screens.** The vibrating screen is the most widely used screen for aggregate production. Figure 17-15 shows a multiple-deck screen unit of this type. The steel frame may be designed to permit the installation of one or more screens, one above the other. Each screen is referred to as a deck. The vibration is obtained by means of an eccentric shaft, a counterweighted shaft, or electromagnets attached to the frame or to the screens.

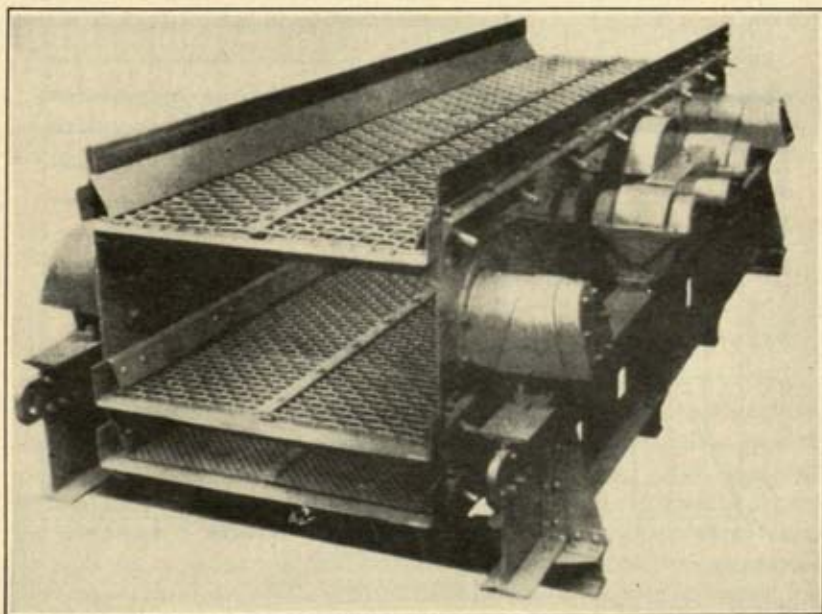


FIG. 17-15. Horizontal vibrating screens. (The W. S. Tyler Co.)

A unit is installed with a slight slope from the receiving to the discharge end, which, combined with the vibrations, causes the aggregate to flow over the surface of the screen. Most of the particles that are smaller than the openings in a screen will drop through the screen, while the over-size particles will flow off the screen at the discharge end. For a multiple-deck unit the sizes of the openings will be progressively smaller for each lower deck.

A screen will not pass all material whose sizes are equal to or less than the dimensions of the openings in the screen. Some of this material may be retained on and carried over the discharge end of a screen. The efficiency of a screen may be defined as the ratio of the amount of material



passing through a screen divided by the total amount that is small enough to pass through, with the ratio expressed as a per cent. The highest efficiency is obtained with a single-deck screen, usually amounting to 90 to 95 per cent. As additional decks are installed, the efficiencies of these decks will decrease, being about 85 per cent for the second deck and 75 per cent for the third deck.

The capacity of a screen is the number of tons of material that 1 sq ft will pass per hour. The capacity will vary with the size of the openings, kind of material screened, moisture content, and other factors. Because

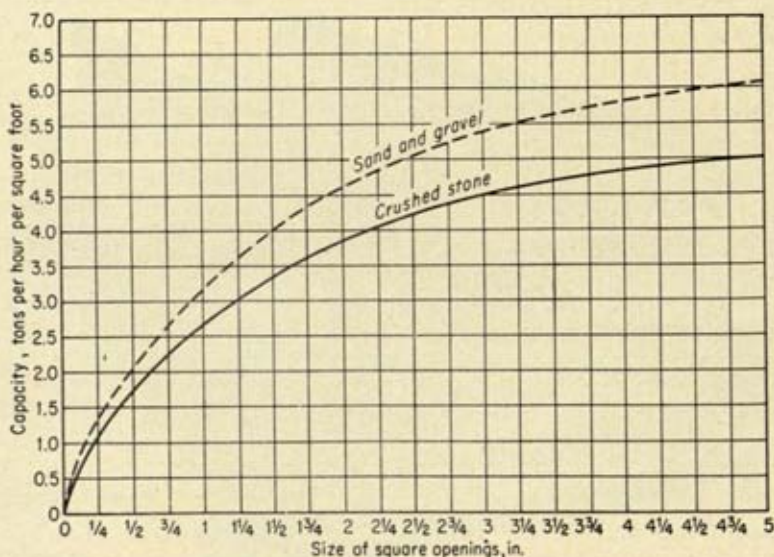


FIG. 17-16. Screen-capacity chart.

of the factors that affect the capacity of a screen it will seldom if ever be possible to calculate in advance the exact capacity of a screen. If a given number of tons of material must be passed per hour, it is good practice to select a screen whose total calculated capacity is 10 to 25 per cent greater than the quantity to be screened.

The chart in Fig. 17-16 gives capacities for dry screening which may be used as a guide in selecting the correct size screen for a given flow of material. The capacities given in the chart should be modified by the application of appropriate correction factors. Representative values of these factors are given hereafter.

**Efficiency Factors.** If a low screening efficiency is permissible, the capacity of a screen may be higher than the values given in Fig. 17-16.

Table 17-7 gives factors by which the chart values of capacities may be multiplied to obtain corrected capacities for given efficiencies.

TABLE 17-7. EFFICIENCY FACTORS

Permissible screen efficiency, %	Efficiency factor
95	1.00
90	1.25
85	1.50
80	1.75
75	2.00

**Deck Factors.** This is a factor whose value will vary with the particular deck position for multiple-deck screens. The values are given in Table 17-8.

TABLE 17-8. DECK FACTORS

For deck No.	Deck factor
1	1.00
2	0.90
3	0.75
4	0.60

**Aggregate-size Factors.** The capacities of screens given in Fig. 17-16 are based on screening dry material which contains particle sizes such as would be found in the output of a representative crusher. If the material to be screened contains a surplus of small sizes, the capacity of the screen will be increased, whereas if the material contains a surplus of large sizes, the capacity of the screen will be reduced. Table 17-9 gives representative factors which may be applied to the capacity of a screen to correct for the effect of fine or coarse particles.

TABLE 17-9. AGGREGATE-SIZE FACTORS

Per cent of aggregate less than $\frac{1}{2}$ the size of screen opening	Aggregate- size factor
10	0.55
20	0.70
30	0.80
40	1.00
50	1.20
60	1.40
70	1.80
80	2.20
90	3.00



**Determining the Size Screen Required.** Figure 17-16 gives the theoretical capacity of a screen in tons per hour per square foot based on material weighing 100 lb per cu ft when crushed. The corrected capacity of a screen is given by the formula

$$Q = ACEDG \quad (17-4)$$

where  $Q$  = capacity of screen, tons per hr

$A$  = area of screen, sq ft

$C$  = theoretical capacity of screen, tons per hr per sq ft

$E$  = efficiency factor

$D$  = deck factor

$G$  = aggregate-size factor

The minimum area of a screen to provide a given capacity is determined from the formula

$$A = \frac{Q}{CEDG} \quad (17-5)$$

**EXAMPLE.** Determine the minimum-size single-deck screen, having 1½-in.-square openings, for screening 120 tons per hr of dry crushed stone, weighing 100 lb per cu ft when crushed. A screening efficiency of 90 per cent is satisfactory. An analysis of the aggregate indicates that approximately 30 per cent of it will be less than ¾ in. in size. The values of the factors to be used in formula (17-5) are

$Q = 120$ tons per hr	
$C = 3.32$ tons per hr per sq ft	Fig. 17-16
$E = 1.25$	Table 17-7
$D = 1.0$	Table 17-8
$G = 0.8$	Table 17-9

Substituting these values in formula (17-5), we get

$$A = \frac{120}{3.32 \times 1.25 \times 1.0 \times 0.8} = 36.1 \text{ sq ft}$$

In view of the possibility of variations in the factors used, and to provide a margin of safety, it is recommended that a 4- by 10-ft screen be selected.

**Portable Crushing and Screening Plants.** Many types and sizes of portable crushing and screening plants are used in the construction industry. When there is a satisfactory deposit of stone near a project that requires stone aggregate, it frequently will be more economical to set up a portable plant and produce the crushed stone instead of purchasing it from a commercial source.

A typical portable crushing and screening plant is illustrated in Fig. 17-17. The stone from the quarry is fed to the plant by a belt conveyor at the right. This particular machine is designed to permit the quarry product to pass over a bottom-deck screen, which removes the material smaller than the screen, thereby reducing the load on the primary crusher. The oversize stone from this screen is fed to a jaw crusher, thence to a

belt conveyor, which returns it to a top-deck screen, where the specification sizes are removed. The oversize stone is fed to a roll crusher. Portable plants commonly use a jaw crusher for primary crushing and a roll crusher for secondary crushing. Changes in the specification sizes may be met, over a reasonably wide range, by adjusting the crusher settings and changing the sizes of the screens.

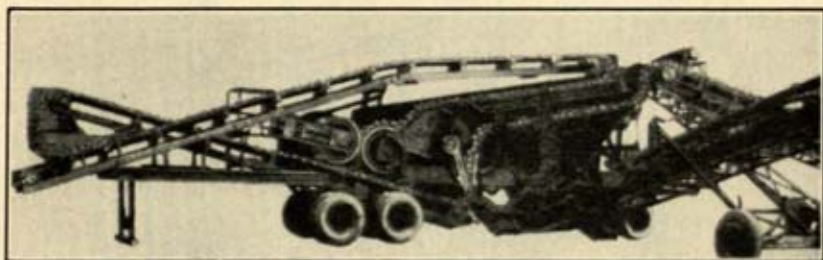


FIG. 17-17. Portable rock-crushing plant. (*Pioneer Engineering Works.*)

**Flow Diagrams of Aggregate-processing Plants.** Figure 17-18 illustrates a flow diagram for a portable aggregate-processing plant. By passing the stone from the quarry over a screen before it goes to the primary crusher, any stone within the specification sizes will be removed prior to crushing. This arrangement should increase the output of the plant.

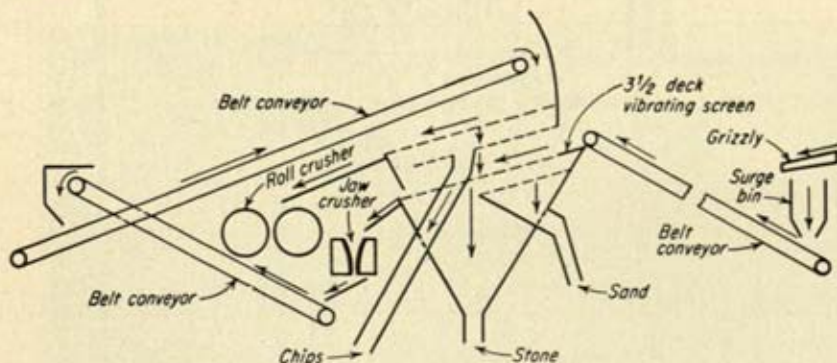


FIG. 17-18. Flow diagram of a portable aggregate processing plant.

Representative sizes of crushers and other equipment for this plant are as follows:

- Jaw crusher, 10 × 36 in.
- Roll crusher, 40 × 22 in.
- Vibrator screen, 4 × 12 ft, 3½ decks
- Feeder, 4-ft hopper
- Feeder conveyor, 30 in. wide, 50 ft long
- Return conveyor, 24 in. wide



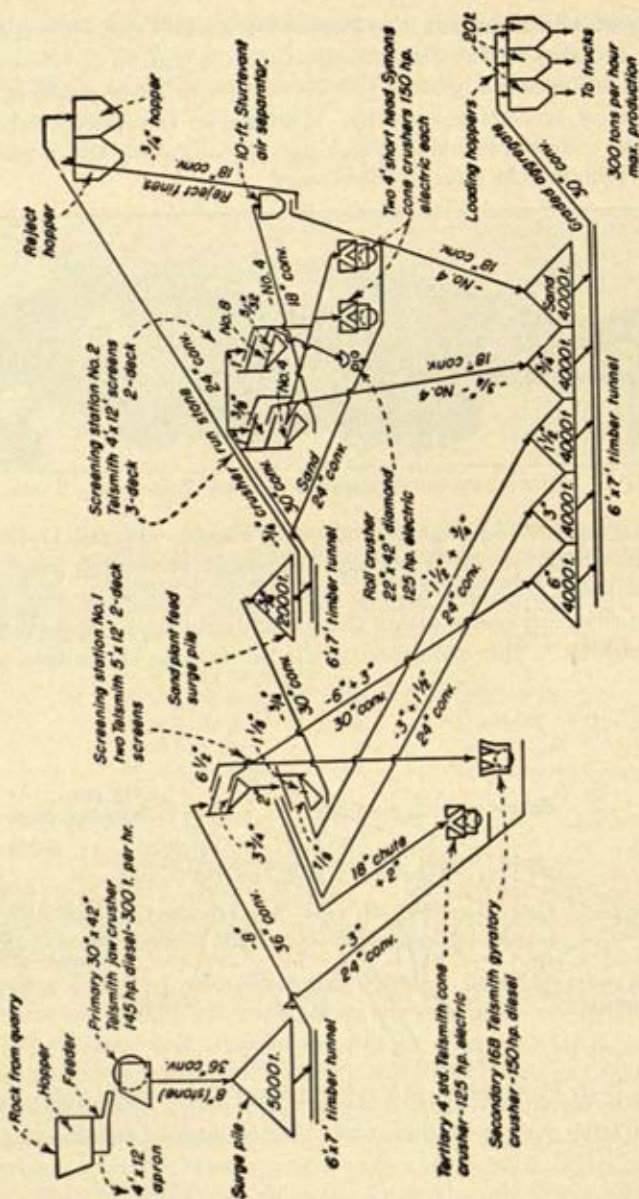


FIG. 17-19. Flow diagram for the aggregate processing plant at the Philpott Dam. (Construction Methods and Equipment.)

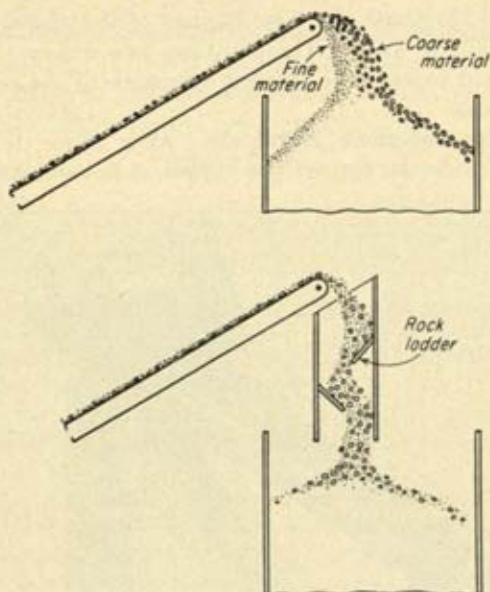


FIG. 17-20. Method of preventing the segregation of aggregate discharged from a conveyor belt.



FIG. 17-21. Pile of aggregate showing segregation. (U.S. Bureau of Reclamation.)



Figure 17-19 illustrates the flow diagram of the aggregate processing plant for the Philpott Dam. This plant was located near the quarry, and trucks were used to haul the finished aggregate to the concrete mixing plant at the dam.

**Handling Crushed-stone Aggregate.** After stone is crushed and screened to provide the desired size ranges, it is necessary to handle it

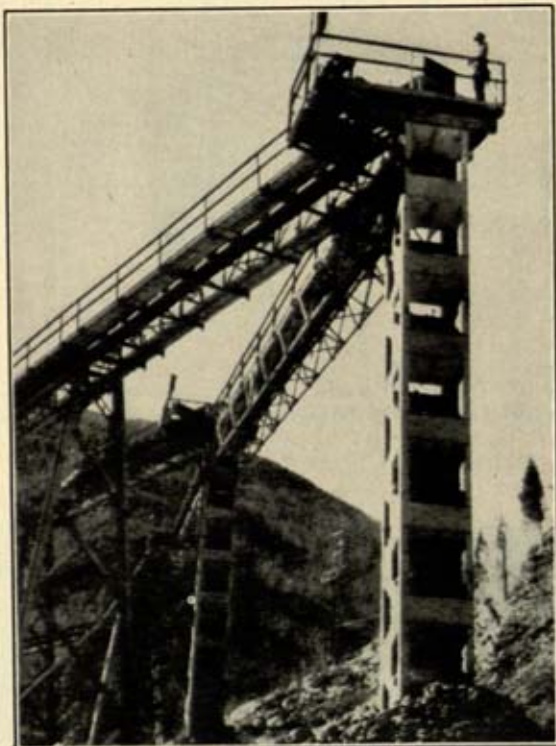


FIG. 17-22. A rock ladder used to reduce the segregation of aggregate. (U.S. Bureau of Reclamation.)

carefully or the large and small particles may be separated, thereby destroying the blend in sizes, which frequently is essential to a satisfactory aggregate. If aggregate is permitted to flow freely off the end of a belt conveyor, especially at some height above the storage bin, the material will be segregated by sizes, as illustrated in Fig. 17-20. A strong cross wind tends to separate the smaller sizes from the larger sizes. Specifications covering the production of aggregate frequently stipulate that the aggregate transported by a belt conveyor shall not be permitted to fall freely from the discharge end of a belt. The end of the belt should be

kept as low as possible, and the aggregate should be discharged through a rock ladder, containing baffles, to prevent segregation.

Figure 17-21 shows a pile of badly segregated aggregate, while Fig. 17-22 shows a rock ladder used to reduce segregation.

### PROBLEMS

**17-1.** A jaw crusher, with a closed setting of 4 in., produces 200 tons per hr of crushed stone. Determine the number of tons per hour produced within the following size ranges: in excess of 3 in.; between 3 and  $1\frac{1}{2}$  in.; between  $1\frac{1}{2}$  and  $\frac{3}{4}$  in.; less than  $\frac{3}{4}$  in.

**17-2.** Select a jaw crusher for primary crushing and a roll crusher for secondary crushing to produce 150 tons per hr of limestone rock. The maximum-size stone from the quarry will be 16 in. The crushed stone will be used on a project whose specifications require the following size distribution:

Size screen opening, in.		Per cent
Passing	Retained on	
$2\frac{1}{2}$	..	100
$2\frac{1}{2}$	1	40-50
1	$\frac{1}{4}$	30-40
$\frac{1}{4}$	0	20-38

Specify the size of and setting for each crusher selected.

**17-3.** A jaw crusher and a roll crusher are used as a primary and a secondary crusher, respectively, to produce 100 tons per hr of stone. The specifications for the stone require the following size distribution:

Size screen openings, in.		Per cent
Passing	Retained on	
2	..	100
2	1	42-44
1	$\frac{1}{4}$	32-34
$\frac{1}{4}$	0	25-27

Select crushers to produce this aggregate. Is it possible to produce aggregate to meet the requirements of the specifications without a surplus in any of the size ranges? Criticize the specifications for the aggregate with respect to the limitations on the size distribution.

**17-4.** A size 24- by 36-in. jaw crusher is set to operate with a 3-in. opening. The output from this crusher is discharged on a screen with  $1\frac{1}{2}$ -in. openings, whose efficiency is 90 per cent. The aggregate that does not pass through the screen goes to a 40- by 20-in. roll crusher, set at  $1\frac{1}{2}$  in. The output from the roll crusher is fed back over the  $1\frac{1}{2}$ -in. screen.



Determine the maximum output of the plant in tons per hour.

Determine the output of the plant in tons per hour for each of the following two sizes:  $1\frac{1}{2}$  to  $\frac{1}{2}$  in.; less than  $\frac{1}{2}$  in.

**17-5.** Estimate the probable output of a portable crushing plant equipped with the following units:

1 jaw crusher, size  $15 \times 24$  in.

1 roll crusher, size  $24 \times 16$  in.

1 set of horizontal vibrating screens, with 2 decks, and with  $1\frac{1}{2}$ - and  $\frac{1}{2}$ -in. openings

The specifications require that 100 per cent of the aggregate shall pass a  $1\frac{1}{2}$ -in. screen and 40 per cent shall pass a  $\frac{1}{2}$ -in. screen.

Assume that 20 per cent of the stone from the quarry will be smaller than  $1\frac{1}{2}$  in. and that this aggregate will be removed by passing the quarry product over the screens prior to sending it to the jaw crusher. The aggregate will weigh 100 lb per cu ft.

Determine the maximum output of the plant, including the aggregate removed by the screens prior to crushing, expressed in tons per hour.

**17-6.** The output from a 36- by 42-in. jaw crusher, with a closed setting of 3 in., is passed over a single horizontal vibrating screen with  $1\frac{1}{2}$ -in. openings. The permissible screen efficiency is 85 per cent. Use the information available in the text to determine the minimum-size screen, expressed in square feet, to handle the output of the crusher.

**17-7.** The output from a 36- by 42-in. jaw crusher, with a closed setting of 3 in., is to be screened into the three following sizes:  $2\frac{1}{2}$  to  $1\frac{1}{2}$  in.;  $1\frac{1}{2}$  to  $\frac{1}{2}$  in.; less than  $\frac{1}{2}$  in. A three-deck horizontal vibrating screen will be used to separate the three sizes. The stone will weigh 100 lb per cu ft. The permissible screen efficiency is 90 per cent. Determine the minimum-size screen for each deck, expressed in square feet, to handle the output of the crusher.

## CHAPTER 18

### FORMS FOR CONCRETE STRUCTURES

**Introduction.** The cost of concrete for most structures includes the cost of the concrete in place plus the cost of the forms that are required to support the concrete until it gains sufficient strength to support itself. The cost of forms frequently will exceed the cost of the concrete alone. This is demonstrated by examples where ready-mixed concrete may be purchased at a job for prices varying from \$10.00 to \$15.00 per cubic yard, but the contract price for a finished structure may be as high as \$30.00 to \$60.00, or more, per cubic yard, excluding the reinforcing steel. Because of the large portion which forms contribute to the cost of concrete, it seems evident that any effort to effect economy in concrete structures should be concentrated primarily on reducing the cost of forms.

**Form Requirements.** As concrete is in a plastic state when it is placed, it is necessary to use forms to confine and support it until it is rigid and self-supporting. Because of its initial plasticity, concrete can be cast in any desired shape, provided forms can be built to conform with that shape. However, complicated forms are expensive.

Forms for concrete structures should be:

1. Strong enough to resist the pressure or the weight of the fresh concrete plus any superimposed loads
2. Rigid enough to retain the shape without undue deformation
3. Economical in terms of the total cost of the forms, concrete, and surface finishing the concrete, when it is required

Forms should be designed by a person who has a knowledge of forces and the strength of materials. Guessing can result in forms which are underdesigned or overdesigned. The former is dangerous, because form failures are expensive, while the latter is unnecessarily expensive. A correct design should eliminate both of these possibilities.

Sometimes the specifications for a concrete structure impose rigid limitations on the variations in shape. Under such conditions form rigidity will be more important than strength.

When a concrete surface must be free from form marks and other blemishes, it may be economical to use expensive form materials in contact with the concrete surfaces in order to eliminate or reduce the cost of finishing the surfaces after the forms are removed. However, if the concrete surface will not be exposed or if surface defects are not objectionable, the form linings should be selected in the interest of economy. This may be illustrated by a concrete retaining wall which requires a smooth finish on the exposed surface. The sheathing for the exposed surface might be



new plywood, while the sheathing for the back surface can be any material that is strong enough to resist the pressure of the concrete.

**The Cost of Forms.** The chief items which affect the cost of forms are materials and labor in making, erecting, and removing the forms.

Materials include lumber, steel, nails, bolts, and form connectors, such as wall ties, etc. If the shape of a concrete member is such that little or no salvage value can be realized from the materials after a single use, the cost of materials will be high, whereas if the materials can be used a great many times, the cost per use may be relatively low. A concrete wall may require 3 fbm of form lumber, costing \$0.10 per fbm, for each square foot of exposed surface. If the lumber can be used only once, the cost will be \$0.30 per square foot, whereas if it can be used 10 times, the cost will be \$0.03 per square foot. Although the initial cost of steel forms usually will be considerably higher than for wood forms, the large number of uses under favorable conditions may reduce the cost per use to less than for wood.

The cost of labor includes the cost of making, erecting, and removing the forms. If forms can be fabricated into shapes that can be reused several times by simply reassembling the component parts, the labor cost of fabricating will occur only once. For successive uses the labor cost will involve erection and removal only.

**EXAMPLE.** Compare the cost of lumber and labor for 100 sq ft of forms for concrete columns based on using the forms once versus using them six times. The forms will be assembled with adjustable steel clamps. If dressed and matched (*D* and *M*) sheathing is used, it will require 1.7 fbm of lumber per square foot of exposed surface [1].

For a single use, assuming no salvage value for the lumber, the cost will be

Lumber, 100 sq ft $\times$ 1.7 fbm per sq ft = 170 fbm @ \$0.10	= \$17.00
Carpenter making, 100 sq ft $\times$ 3.0 hr per 100 sq ft = 3 hr @ \$2.50	= 7.50
Helper making, 100 sq ft $\times$ 1.0 hr per 100 sq ft = 1 hr @ \$1.25	= 1.25
Carpenter erecting, 100 sq ft $\times$ 6.0 hr per 100 sq ft = 6 hr @ \$2.50	= 15.00
Helper erecting and removing, 100 sq ft $\times$ 5.0 hr per 100 sq ft = 5.0 hr @ \$1.25	= 6.25
Total cost	= \$47.00
Cost per sq ft, \$47.00 $\div$ 100	= 0.47

Based on six uses, with no salvage value for the lumber, the cost will be

Lumber, from previous list	= \$ 17.00
Carpenter making, from previous list	= 7.50
Helper making, from previous list	= 1.25

For five additional uses

Carpenter erecting, 5 $\times$ \$15.00	= 75.00
Helper erecting and removing, 5 $\times$ \$6.25	= 31.25
Total cost	= \$132.00
Cost per sq ft per use, \$132 $\div$ 600	= 0.22

This example illustrates the effect which the multiple use of forms has on the cost of forms per use. The reduction frequently is sufficient to justify designing a structure with members the same size even though loading conditions might permit the use of smaller members for a portion of the structure.

**Designing a Project for Form Economy.** Opportunities for form economy originate with the design of a structure. In order to permit the greatest economy, consistent with the type of structure, the designer must have a reasonable knowledge of the cost of forms. Certain shape and finish requirements may be desirable, and in many instances they are justified, even though they increase the cost of a structure. At least the designer should consider their value to determine whether the increased cost is justified.

Among the steps which a designer can take to effect economy in concrete forms are the following:

1. Reduce the number of irregular shapes to a minimum.
2. Duplicate the sizes and shapes of structural members when practical to permit reuses of forms.
3. Design a structure to permit the use of cost-saving commercial forms, such as metal pans or corrugated steel sheets for decking for floor slabs.
4. Have a constructor review the preliminary plans to suggest methods of reducing the cost of forms without sacrificing the quality of the structure.
5. Consider the use of tilt-up, slip-form, or other cost-saving construction methods.
6. Allow the use of construction joints to permit the reuse of forms.
7. Specify a quality of workmanship no finer than is needed for the project.
8. Do not specify unreasonable limitations on the dimensions of structural members.
9. Design structural members to permit the use of commercial sizes of lumber without ripping, when practical.
10. Permit the constructor to use his own methods of building the forms by holding him responsible for adequacy only.
11. Permit the constructor to remove and reuse forms as soon as it is safe to do so.

**The Constructor and Form Economy.** Among the steps which a constructor can take to reduce the cost of forms for concrete structures are the following:

1. Design the forms to provide adequate but not excessive strength and rigidity.
2. Fabricate the forms into modular sizes to permit more reuses without refabricating, when practical.



3. Prepare working drawings for all except the simplest forms prior to fabricating the forms.

4. Prefabricate form sections on the ground, using power equipment, in order to reduce labor costs and unnecessary delays on the job. Labor is much more efficient when working on the ground than when working on a scaffold.

5. When possible adopt assembly-line methods in fabricating forms to increase the efficiency of the workmen.

6. Use the most economical form material, considering initial cost and reuses.

7. Use laborsaving commercial connectors, such as wall ties, when their use is permitted.

8. Use commercial forms when they are cheaper than conventional forms.

9. Use no more nails than are needed to join the forms together safely.

10. When possible use scaffold or double-headed nails to facilitate their removal and to reduce the damage to lumber.

11. Remove forms as soon as it is permissible.

12. Clean and oil forms after each use.

13. If they are permissible, install construction joints to reduce the total quantity of form material required and to permit the carpenters to work more continuously.

**Materials for Forms.** The materials used for forms may be dictated by economy, necessity, or a combination of the two factors. The materials most commonly used include lumber, plywood, steel, and aluminum, either separately or in combination. If the material will be used only a few times, lumber usually will be more economical than steel or aluminum. However, if the forms can be fabricated into panels or other shapes which will be used many times, the greater number of uses obtainable with steel and aluminum may result in a lower cost per use than with lumber.

Forms for some structures and members should be made out of steel as a matter of expediency. These include forms for round columns, curved surfaces, monolithic sewers, tunnels, etc.

**The Size of Form Sections.** If forms are prefabricated into panels or sections, it is desirable to fabricate sizes as large as the concrete members or the methods of handling will permit, as such use should reduce the time and labor costs in erecting and removing the forms.

If the forms are handled by men, the weight of a panel should be limited to approximately 75 lb per man. If they are handled by power equipment, the size will be limited by the lengths of lumber available, the dimensions of the concrete structure, or the capacity of the hoisting equipment.

Figure 18-1 illustrates the use of large prefabricated form panels in constructing a flood wall along the Ohio River at Portsmouth, Ohio. The forms for this 7,290-ft-long structure were handled by a gantry crane.

**Properties of Lumber.** Table 18-1 gives the properties of various kinds of lumber used for forms. The given properties are based on using lumber of a quality not lower than the specified grade. If a lower-grade lumber is used, the working stresses should be reduced to values which are safe for the particular grade.

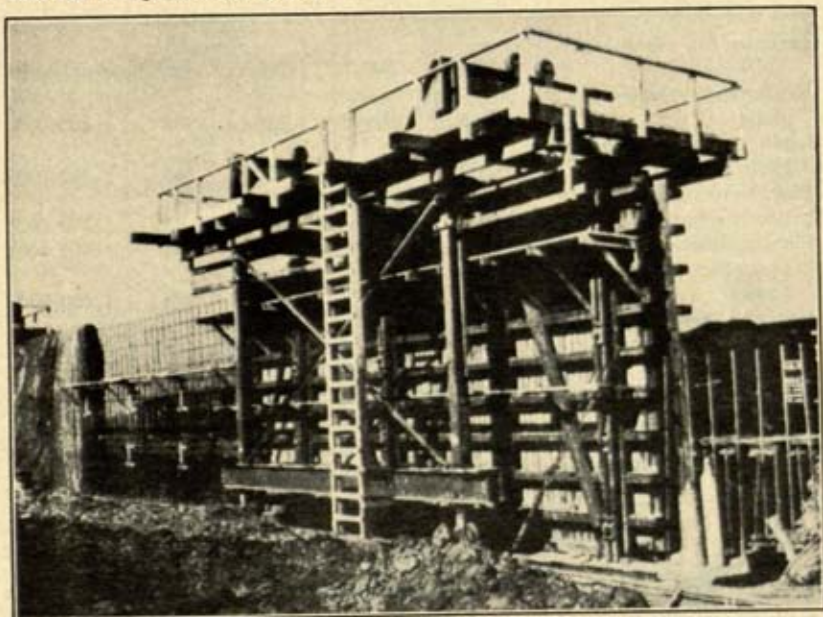


FIG. 18-1. Prefabricated forms used to construct a flood wall. (*Corps of Engineers, U.S. Army.*)

The dimensions and dimensional properties of various sizes of lumber commonly used for forms are given in Table 18-2. The values are based on using rough or S4S lumber. In the table,  $b$  designates the width and  $h$  the depth, expressed in inches.

**Pressure Produced by Concrete.** When concrete is placed in forms, it produces a pressure perpendicular to the forms which is proportional to the density and the depth of the concrete in a liquid state. As the concrete sets, it changes from liquid to a solid, with a reduction in the pressure exerted on the forms. The time required for the initial set is longer for a low temperature than for a high one. The depth of concrete in a liquid state varies with the temperature and the rate of filling the forms. If the forms are filled at a rate of 6 ft per hr, the maximum pressure will be greater than if the forms are filled at a rate of 2 ft per hr.



TABLE 18-1. PROPERTIES OF VARIOUS KINDS OF LUMBER USED FOR FORMS\*

Kind of lumber	Safe working stress, psi				Modulus of elasticity, psi
	Extreme fiber in bending	Compression perpendicular to grain	Compression parallel to grain	Horizontal shear	
Douglas fir, coast region, No. 1 grade.....	1,800	490	1,500	150	1,600,000
Hemlock, west coast, No. 1 grade.....	1,800	450	1,340	125	1,400,000
Larch, common structural grade.....	1,800	490	1,650	150	1,500,000
Pine, Norway, common structural grade.....	1,375	450	970	95	1,200,000
Pine, southern, No. 1 grade	1,800	490	1,500	155	1,600,000
Pine, southern longleaf, No. 1 grade.....	2,125	570	1,750	190	1,600,000
Redwood, heart, structural grade.....	1,625	400	1,375	120	1,200,000
Spruce, eastern, structural grade.....	1,625	375	1,220	120	1,200,000

\* National Design Specifications for Stress-grade Lumber and Its Fastenings, 1950 rev. ed., recommended by the National Lumber Manufacturers Association.

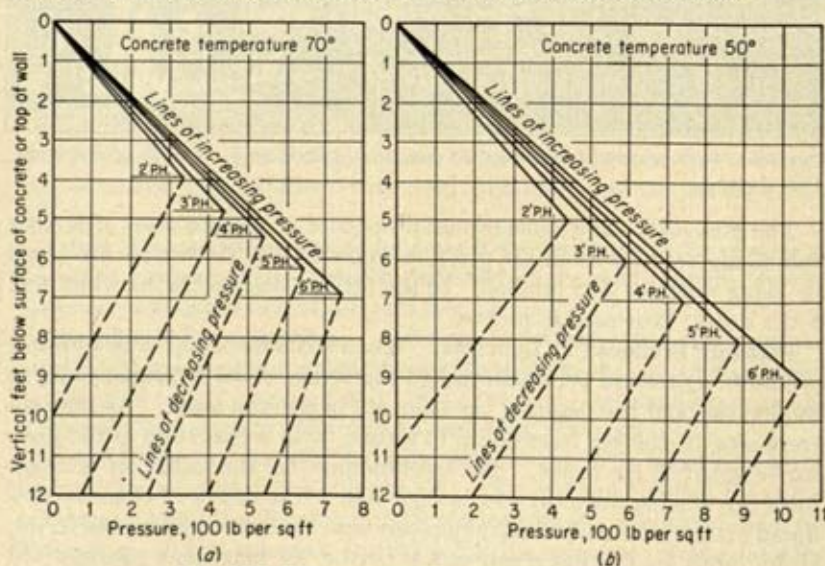


FIG. 18-2. Pressure exerted on forms by concrete.

If the forms for a tall structure, such as a wall, are filled over a period of several hours at a uniform rate and at a uniform temperature, the depth of maximum pressure, measured below the surface of the concrete, will remain constant. Thus, the point of maximum pressure will rise at the same rate that the forms are filled.

TABLE 18-2. PROPERTIES OF AMERICAN STANDARD LUMBER SIZES COMMONLY USED FOR FORMS

Nominal size, in.	Dressed size, S4S, in.	Area of section, sq in.		Moment of inertia, $I = \frac{bh^3}{12}$ in. <sup>4</sup>				Section modulus $S = \frac{bh^2}{6}$ in. <sup>3</sup>			
		Rough	S4S	Rough		S4S		Rough		S4S	
				Min	Max	Min	Max	Min	Max	Min	Max
1 × 4	2½ × 3½	4.0	2.83	0.33	5.33	0.14	3.01	0.67	2.66	0.37	1.71
1 × 6	2½ × 5½	6.0	4.40	0.50	18.00	0.22	11.25	1.00	6.00	0.57	4.12
1 × 8	2½ × 7½	8.0	5.86	0.67	42.66	0.30	26.25	1.33	10.66	0.76	7.30
1½ × 4	1½ × 3½	5.0	3.85	0.65	6.65	0.36	4.22	1.04	3.32	0.68	2.32
1½ × 6	1½ × 5½	7.5	5.98	0.98	22.50	0.56	15.82	1.56	7.50	1.06	5.60
1½ × 8	1½ × 7½	10.0	7.97	1.30	53.33	0.75	37.40	2.08	13.30	1.41	9.93
1½ × 4	1½ × 3½	6.0	4.76	1.13	8.00	0.68	5.19	1.50	4.00	1.04	2.87
1½ × 6	1½ × 5½	9.0	7.38	1.69	27.00	1.06	19.46	2.25	9.00	1.62	6.91
1½ × 8	1½ × 7½	12.0	9.84	2.25	64.00	1.41	46.15	3.00	16.00	2.15	12.23
2 × 4	1½ × 3½	8.0	5.89	2.67	10.67	1.30	6.45	2.67	5.33	1.60	3.56
2 × 6	1½ × 5½	12.0	9.14	4.00	36.00	2.01	24.10	4.00	12.00	2.48	8.57
2 × 8	1½ × 7½	16.0	12.19	5.33	85.33	2.68	57.13	5.33	21.33	5.30	15.23
2 × 10	1½ × 9½	20.0	15.44	6.67	166.67	3.40	116.10	6.67	33.33	4.18	24.24
2 × 12	1½ × 11½	24.0	18.69	8.00	288.00	4.11	205.95	8.00	48.00	5.06	35.82
3 × 4	2½ × 3½	12.0	9.53	9.00	16.00	5.48	10.38	6.00	8.00	4.16	5.75
3 × 6	2½ × 5½	18.0	14.77	13.50	54.00	8.48	38.93	9.00	18.00	6.47	13.84
3 × 8	2½ × 7½	24.0	19.69	17.9	128.00	11.29	92.29	12.00	32.00	8.62	24.61
3 × 10	2½ × 9½	30.0	24.94	22.5	250.00	14.28	187.55	15.00	50.00	10.91	39.48
3 × 12	2½ × 11½	36.0	30.19	27.0	432.00	17.30	332.69	18.00	72.00	13.23	57.36
4 × 4	3½ × 3½	16.0	13.14	21.3	21.3	14.35	14.35	10.67	10.67	7.95	7.95
4 × 6	3½ × 5½	24.0	20.39	32.0	72.0	22.28	53.80	16.00	24.00	12.31	19.10
4 × 8	3½ × 7½	32.0	27.19	42.6	170.7	29.70	126.80	21.33	42.67	16.42	34.00
4 × 10	3½ × 9½	40.0	34.44	53.3	333.3	37.80	258.80	26.67	66.67	20.80	54.40
4 × 12	3½ × 11½	48.0	41.69	64.0	576.0	45.60	458.50	32.00	96.00	25.20	80.00

Since fresh concrete is not a perfect liquid, it is impossible to determine the exact pressure that will be exerted on the forms. Tests indicate that the pressure is influenced by the following factors, which should be considered by the designer:

1. Rate of filling the forms
2. Temperature of the concrete
3. Method of placing the concrete, whether by hand puddling or by vibrating

Figure 18-2 shows the pressures exerted at various depths for the speci-



fied rates of filling forms. If the concrete is mechanically vibrated, the pressures should be increased approximately 25 per cent.

Table 18-3 gives the maximum pressure exerted for various rates of filling forms and temperatures. The pressures in this table are based on vibrating the concrete as it is placed.

TABLE 18-3. MAXIMUM PRESSURE, IN PSF, EXERTED BY CONCRETE WHEN VIBRATED\*

(Pressure shall not exceed 150 X depth of concrete in feet)

Rate of filling forms, ft per hr	Temperature, °F				
	80	70	60	50	40
1	425	438	463	500	550
2	475	500	550	625	725
3	525	563	638	750	700
4	575	625	725	875	1,075
5	625	688	813	1,000	1,250
6	675	750	900	1,125	1,425
7	725	813	988	1,250	1,600
8	775	875	1,075	1,375	1,775
9	825	938	1,165	1,500	1,950
10	875	1,000	1,250	1,600	2,125
15	1,125	1,313	1,688	2,250	2,995
20	1,375	1,675	2,125	2,875	3,875

\* Courtesy Williams Form Engineering Corporation.

**Fundamentals of Form Design.** As previously stated, forms should be strong enough to resist the stresses produced in them and rigid enough to limit the deformation to the values permitted. Table 18-1 gives maximum unit stresses which are considered safe for various kinds of lumber. These stresses, which are higher than are permitted in a permanent structure, have an adequate factor of safety for forms. If the lumber used is of a lower grade than that specified in the table, the stresses should be reduced accordingly.

Sometimes specifications limit the deflection of forms in order to eliminate objectionable bulges on the concrete surface. Representative deflection limits might be  $\frac{1}{8}$  in. or  $\frac{1}{270}$  of the span of the sheathing, studs, or wales. For most forms the size and spacing of studs and wales will be governed by the stresses in bending and shear, while deflection may limit the maximum span for sheathing.

In developing formulas for designing forms, the following symbols will be used:

$w$  = uniform load, lb per lin ft

$w'$  = uniform load, psf

- $p$  = pressure, lb per lin ft  
 $p'$  = pressure, psf  
 $P$  = safe load on a shore, lb  
 $l$  = span from center to center of supports, in.  
 $L$  = span from center to center of supports, ft  
 $b$  = width of member, in.  
 $h$  = depth of member, in.  
 $g$  = height of shore, in.  
 $f$  = extreme fiber stress due to bending, psi  
 $v$  = horizontal shearing stress, psi  
 $V$  = external shear in a member, lb  
 $M$  = bending moment in a member, in.-lb  
 $M'$  = resisting moment of a member, in.-lb  
 $I$  = moment of inertia of a member =  $bh^3/12$   
 $S$  = section modulus of a member =  $bh^2/6$   
 $E$  = modulus of elasticity, psi  
 $D$  = deflection of a member, in.

In all instances the actual dimensions of a member should be used in designing forms (see Table 18-2).

**Stresses Due to Bending.** For most forms, sheathing, studs, and wales are continuous over several supports, and the maximum bending moment is given by the formula

$$M = \frac{12wL^2}{10} = 1.2wL^2 \quad (18-1)$$

The resisting moment of a member is given by the formula

$$M' = \frac{fbh^2}{6} \quad (18-2)$$

Equating formulas (18-1) and (18-2) and solving for  $L$ , we get

$$1.2wL^2 = \frac{fbh^2}{6}$$

$$L = 0.372h \sqrt{\frac{fb}{w}}$$

or

$$l = 4.464h \sqrt{\frac{fb}{w}} \quad (18-3)$$

The maximum safe load per linear foot is given by the formula

$$w = \frac{fbh^2}{7.2L^2} \quad (18-4)$$



**Shearing Stresses.** When a form member is subjected to transverse forces, shearing stresses in the member may govern the size of the member or the length of span. This is especially true for short spans and heavy loads. The external shear, which occurs at a support, is given by the formula

$$V = \frac{wL}{2} \quad (18-5)$$

The maximum unit shearing stress is

$$\begin{aligned} v &= \frac{1.5V}{bh} \\ \text{or} \quad v &= \frac{1.5wL}{2bh} \end{aligned} \quad (18-6)$$

Solving for  $L$ , we get

$$\begin{aligned} L &= \frac{2vbh}{1.5w} \\ \text{or} \quad l &= \frac{16vbh}{w} \end{aligned} \quad (18-7)$$

**Compression Stresses.** When joists rest on sills or studs bear against wales, the areas of contact are subjected to compression stresses which act perpendicular to the wood fibers. As the safe stresses in compression perpendicular to the fibers are considerably less than those permitted in bending or parallel to the fibers, the stresses at these areas of contact should be checked to see that the safe values are not exceeded. Table 18-1 gives the maximum safe values of compression stresses perpendicular to the grain.

**Deflection of Forms.** When a member, supported at each end, is subjected to a uniform load along its full length, the maximum deflection is given by the formula

$$D = \frac{5wl^4}{384 \times 12 \times EI} \quad (18-8)$$

Solving for  $l$ , we get

$$l = 5.51 \sqrt[4]{\frac{EID}{w}} \quad (18-9)$$

If the lumber is grade 1 Douglas fir or southern pine,  $E = 1,600,000$ .  $I = bh^3/12$ . For sheathing assume  $D$  is limited to  $1/8$  in. Substituting these values in formula (18-9), we get

$$l = 62.6 \sqrt[4]{\frac{bh^3}{w}} \quad (18-10)$$

If a member extends continuously over several supports, such as sheathing over studs, the maximum deflection is given by the formula

$$D = \frac{wl^4}{384 \times 12 \times EI} \quad (18-11)$$

Solving for  $l$ , we get

$$l = 8.24 \sqrt[4]{\frac{EID}{w}} \quad (18-12)$$

Substituting the values of  $E$ ,  $I$ , and  $D$ , as given heretofore, we find the maximum value of  $l$  for sheathing to be

$$l = 93.5 \sqrt[4]{\frac{bh^3}{w}} \quad (18-13)$$

The designer should use the value of  $l$  given by formula (18-10) instead of the value from formula (18-13) unless he is certain that the sheathing will be continuous over several supports. The length of a span adjacent to an end joint should be limited to the value given by formula (18-10).

**Safe Load on Shores.** The maximum safe load on a shore with a rectangular cross section is given by the formula

$$P = 1,000 \left(1 - \frac{g}{80b}\right) bh \quad (18-14)$$

**EXAMPLE.** Determine the maximum spacing of studs for 1-in. nominal-thickness sheathing when the forms for a concrete wall will be filled at the rate of 4 ft per hr and the temperature of the concrete is 70 deg. The maximum permissible deflection of the sheathing is  $\frac{1}{8}$  in. Grade 1 southern pine will be used for the sheathing.

*Sheathing.* Consider a horizontal strip of sheathing 12 in. wide.

$$\begin{aligned} b &= 12 \text{ in.} \\ h &= 2\frac{5}{8} \text{ in.} \\ w &= 625 \text{ psf} \quad (\text{Table 18-3}) \end{aligned}$$

Using formula (18-10),

$$\begin{aligned} l &= 62.6 \sqrt[4]{\frac{12(2\frac{5}{8})^3}{625}} \\ &= 62.6 \times 0.308 = 19.25 \text{ in.} \end{aligned}$$

Using formula (18-13),

$$\begin{aligned} l &= 93.5 \sqrt[4]{\frac{12(2\frac{5}{8})^3}{625}} \\ &= 93.5 \times 0.308 = 28.8 \text{ in.} \end{aligned}$$

The latter value should not be used for short pieces of sheathing or near the ends of sheathing.



**Wall Forms.** The typical wall form shown in Fig. 18-3 includes sheathing,

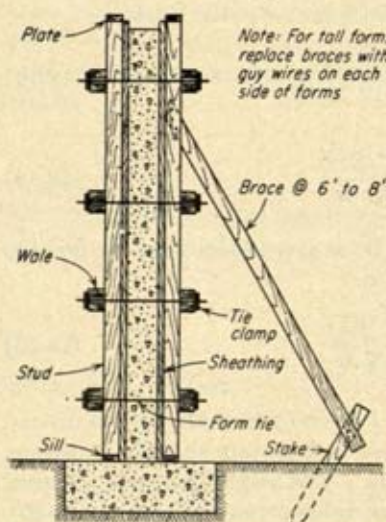


FIG. 18-3. Wood forms for a concrete wall.

ing, studs, wales, ties, and braces. If lumber is used for the sheathing, it may be 1, 1 $\frac{1}{4}$ , 1 $\frac{1}{2}$ , or 2 in. thick. Plywood usually is  $\frac{5}{8}$  or  $\frac{3}{4}$  in. thick.

As illustrated in Fig. 18-3, the opposite sides of the wall forms are held in the correct positions by steel-form ties, which resist the bursting pressure of the concrete and serve as spreaders to govern the width of the space between the forms. Figure 18-4a illustrates a popular type of form tie for light wall construction. This tie has a safe working strength of 3,000 lb. It is designed to be snapped off 1 in. inside the concrete wall.

Figure 18-4b illustrates a coil-type form tie which is suitable for use on heavy wall forms. It has safe working

strengths of 6,000 and 9,000 lb. Many other types of form ties are available.

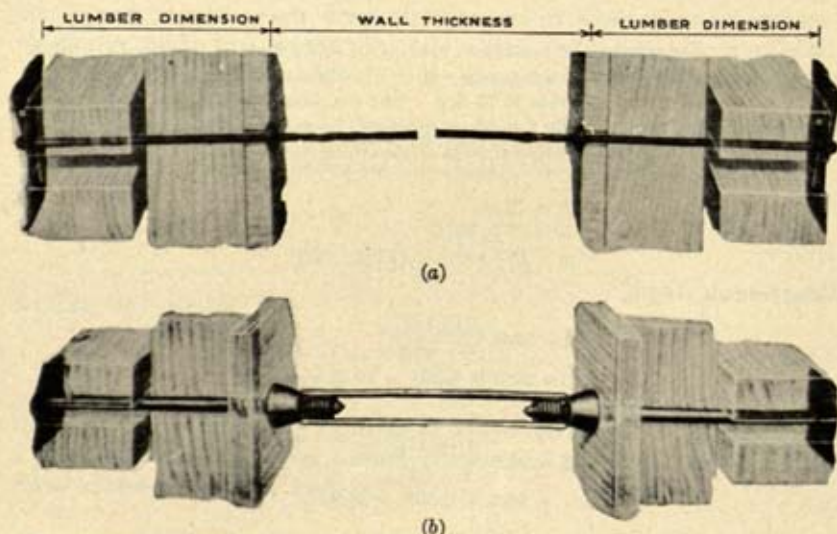


FIG. 18-4. Form ties for concrete walls: (a) snap tie; (b) coil tie. (Superior Concrete Accessories, Inc.)

Figure 18-5 illustrates a representative method of constructing forms for the corner of a wall.

Table 18-4 gives safe spacings for studs, wales, and form ties for wall forms subjected to various pressures. If the concrete is to be vibrated, the pressure selected should allow for the effect of vibration, as indicated in Table 18-3. The values are based on using S4S grade 1 yellow pine or Douglas fir lumber, having the indicated nominal sizes. The spacing of the form ties is based on using ties with adequate working strength to resist the loads to which they will be subjected.

TABLE 18-4. SAFE SPACING OF STUDS, WALES, AND FORM TIES FOR FORMS\*  
(All dimensions in inches)

			Max pressure, psf							
			350	450	550	600	650	750	900	1,000
Spacing of studs for safe value of sheathing										
For 1-in. sheathing.....			24	22	20	20	18	18	16	15
For 2-in. sheathing.....			44	41	39	36	37	32	30	28
Spacing of wales for safe value of studs										
Studs:	Sheathing:									
2 × 4	1		33	30	29	28	28	26	25	24
3 × 4	1		42	38	36	35	35	33	32	30
4 × 4	1		48	45	42	42	40	39	38	36
2 × 6	1		51	47	45	43	44	40	39	37
2 × 6	2		38	34	32	32	31	30	29	27
3 × 4	2		31	28	26	26	25	24	23	22
3 × 6	2		48	43	41	40	38	37	36	34
Spacing of form ties for safe value of wales										
Double wales:	Studs:	Sheathing:								
2 × 4	2 × 4	1	36	33	30	29	28	27	25	24
2 × 4	4 × 4	1	30	27	25	24	24	22	21	20
2 × 4	2 × 6	1	29	26	24	23	23	22	20	19
3 × 4	3 × 4	1	40	37	34	33	32	31	28	27
2 × 6	2 × 4	1	56	51	47	46	44	42	40	37
2 × 6	4 × 4	1	46	42	39	38	37	35	32	30
2 × 6	2 × 6	1	45	40	38	37	36	34	32	30
2 × 6	2 × 6	2	52	48	45	43	42	40	37	35
3 × 6	2 × 6	2	65	61	57	55	53	50	47	45
2 × 6	3 × 6	2	46	43	40	38	38	35	33	31
3 × 4	3 × 4	2	46	43	41	39	38	36	34	32
3 × 6	3 × 6	2	58	54	50	49	48	45	42	40

\* Basic information from Universal Form Clamp Company.



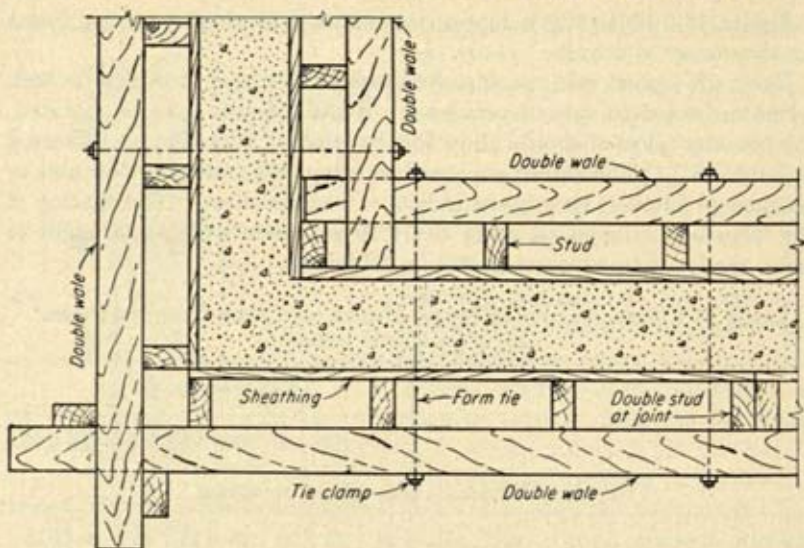


FIG. 18-5. Form detail for the corner of a concrete wall.

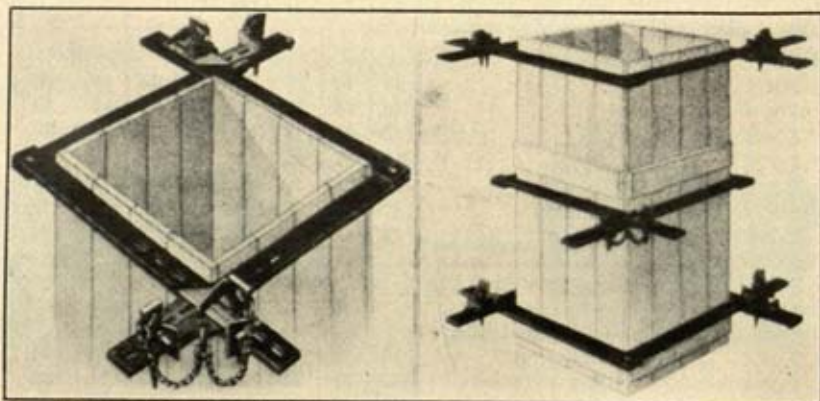


FIG. 18-6. Wood forms and adjustable clamps for concrete columns. (Symons Clamp and Mfg. Co.)

**Column Forms.** Forms for columns usually are made of vertical planks, with nominal thickness 1 in., or plywood. The component parts are prefabricated, then assembled in the location where they will be used. Adjustable steel clamps generally are used to resist the pressure from the concrete, as illustrated in Fig. 18-6.

Forms should be designed to resist the high pressure resulting from quick filling. If the forms are filled in 30 min or less, the concrete may

TABLE 18-5. SAFE SPACING OF ADJUSTABLE STEEL-COLUMN CLAMPS, IN INCHES\*

Height of column, ft	Story height, ft											
	For 1-in.-nominal-thickness sheathing with 1 × 4 cleats						For ¾-in. plywood with vertical 2 × 4 cleats					
	10	12	14	16	18	20	10	12	14	16	18	20
10	32	32	32	32	32	32	33	33	33	33	33	33
	20	20	20	20	20	20	27	27	27	27	27	27
	16	16	16	16	16	16						
	13	13	13	13	13	13	24	24	24	24	24	24
	13	13	13	13	13	13						
	12	12	12	12	12	12	21	21	21	21	21	21
	8	12	12	12	12	12	9					
	6						6	20	20	20	20	20
12		12	12	12	12	12		13	19	19	19	19
		8	11	11	11	11		6				
		6										
14			11	11	11	11			18	18	18	18
			10	10	10	10						
			6					6				
16				10	10	10				15	17	17
				8	10	10				9		
				6								
				6	10	10				6	16	16
18					9	9					15	15
					9	9						
					6	8					6	
												12
20						8						
						8						12
						6						6

\* Basic information furnished by Symons Clamp and Manufacturing Company.



exert the full hydrostatic pressure based on a weight of approximately 150 lb per cu ft.

Table 18-5 gives safe spacing for adjustable steel clamps based on the form material and limiting the deflection of the sheathing between clamps to not more than  $\frac{1}{8}$  in.

**Shores.** The capacities of commercial shores are given by the manufacturers of the shores. The capacities of wood shores may be determined from formula (18-14), which is based on using lumber comparable to Douglas fir or southern pine, grade No. 1.

**EXAMPLE.** Determine the maximum safe load on a 4- by 4-in. rough pine shore 10 ft long, with no horizontal braces.

$$P = 1,000 \left( 1 - \frac{10 \times 12}{80 \times 4} \right) \times 4 \times 4 \\ = 10,000 \text{ lb}$$

Table 18-6 gives safe loads on wood shores for various unsupported lengths. For example, a 14-ft long shore, with 2-way horizontal braces, has an unsupported length of 7 ft. The values are based on using lumber equal in strength to Douglas fir or southern pine, grade No. 1. The load should be applied at the center of the cross section of a straight shore.

TABLE 18-6. CAPACITIES OF WOOD SHORES\*

Unsupported length of shore, ft	Maximum safe load, lb					
	3 × 4 in.		4 × 4 in.		6 × 6 in.	
	Rough	S4S	Rough	S4S	Rough	S4S
4	9,600	7,300	13,600	10,900	32,400	26,800
5	9,000	6,800	13,000	10,400	31,500	26,000
6	8,400	6,200	12,400	9,800	30,600	25,200
7	7,800	5,700	11,800	9,300	29,700	24,400
8	7,200	5,200	11,200	8,800	28,800	23,600
9	6,600	4,600	10,600	8,200	27,900	22,800
10	6,000	4,100	10,000	7,700	27,000	22,000
11	5,400	3,500	9,400	7,100	26,100	21,200
12	4,800	3,000	8,800	6,600	25,200	20,400
14	.....	.....	7,600	5,500	23,400	18,600
16	.....	.....	.....	.....	21,600	17,000
18	.....	.....	.....	.....	19,800	15,400

\* These values are permissible provided the stringer can transmit the loads without exceeding the allowable compression stresses perpendicular to the grain.

**Forms for Beam-and-slab-type Floor Construction.** Figure 18-7 illustrates a method of constructing forms for the beams and slab for this

type of floor system, using commercial lumber. Plywood,  $\frac{5}{8}$ - or  $\frac{3}{4}$ -in. thick, or 1-in. planks may be used for the decking. Plywood is installed more rapidly and has a higher salvage value, which may offset its higher cost compared to planks.

The maximum spacing of the joists will be limited by the strength or the permissible deflection of the decking. The maximum spacing of the stringers will be limited by the strength or the permissible deflection of the joists. The maximum spacing of shores under the stringers will be

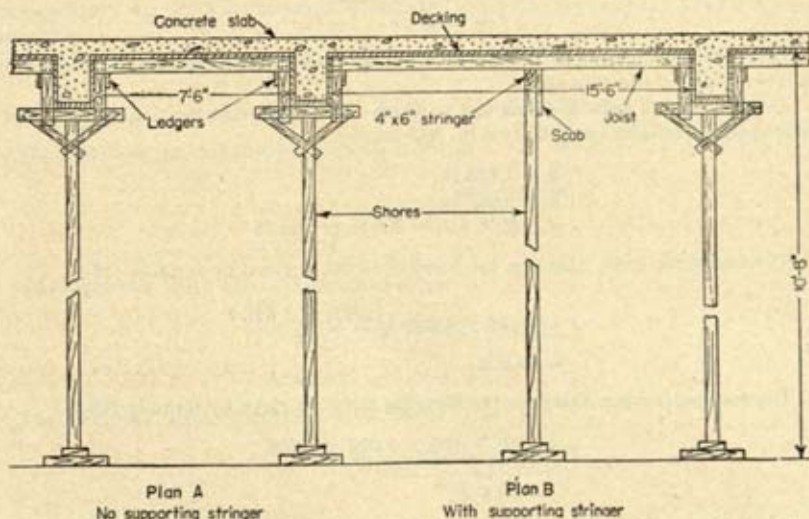


FIG. 18-7. Wood forms for beam-and-slab concrete floor. (By permission from "Estimating Construction Costs," by R. L. Peurifoy. Copyright 1953, McGraw-Hill Book Company, Inc.)

limited by the strength or the permissible deflection of the stringers or the capacity of the shores.

In designing forms for beams and slab, the design load should equal the weight of the concrete, plus an additional load of 40 to 50 psf, to provide for the weight of buggies and workers and for the storage of materials on the slab during construction.

**EXAMPLE.** Design the forms for a concrete slab 6 in. thick, whose net width between beam faces is 15 ft 6 in. Use 1-in. lumber for the decking and 2-in.-thick lumber for the joists.

The total load on the decking will be

$$\begin{array}{rcl}
 \text{Concrete} & = & 75 \text{ psf} \\
 \text{Live load} & = & 40 \text{ psf} \\
 \text{Total} & = & 115 \text{ psf}
 \end{array}$$



## 458 CONSTRUCTION PLANNING, EQUIPMENT, AND METHODS

Consider a strip of decking 12 in. wide. Solving formula (18-3), the maximum spacing of joists, based on the bending stress in the decking, will be

$$l = 4.464 \times \frac{25}{32} \sqrt{\frac{1,800 \times 12}{115}} \\ = 47.8 \text{ in.}$$

Formula (18-10) gives the maximum spacing of the joists for a  $\frac{1}{8}$ -in. deflection of the decking.

$$l = 62.6 \sqrt[4]{\frac{12(\frac{25}{32})^2}{115}} \\ = 29.5 \text{ in.}$$

Limit the joist spacing to 24 in., which will permit the use of commercial lengths of lumber, with no end wastage.

*Joists.* Several sizes of joists can be used. The larger the joist the greater the safe span. Consider using 2- by 6-in. S4S lumber.

$$b = 1.625 \text{ in.} \\ h = 5.625 \text{ in.} \\ w = 2 \times 115 = 230 \text{ lb per lin ft}$$

The maximum span, based on the bending stress, is given by formula (18-3).

$$l = 4.464 \times 5.625 \sqrt{\frac{1,800 \times 1.625}{230}} \\ = 89.6 \text{ in.}$$

The maximum span, based on the shearing stress, is given by formula (18-7).

$$l = \frac{16 \times 155 \times 1.625 \times 5.625}{230} \\ = 98.5 \text{ in.}$$

The maximum span, based on a deflection of  $\frac{1}{8}$  in., is given by formula (18-9).

$$E = 1,600,000 \text{ psi} \quad (\text{Table 18-1}) \\ I = 24.1 \quad (\text{Table 18-2}) \\ l = 5.51 \sqrt[4]{\frac{1,600,000 \times 24.1 \times 1}{230 \times 8}} \\ = 66.5 \text{ in.}$$

If 2- by 6-in. joists are used, the maximum span will be about 5 ft 6 in. This will require two rows of stringers.

Consider using 2- by 8-in. rough lumber.  $I = 85.33$ . The maximum span, based on deflection, will be

$$l = 5.51 \sqrt[4]{\frac{1,600,000 \times 85.33 \times 1}{230 \times 8}} \\ = 91 \text{ in.} \\ = 7 \text{ ft } 7 \text{ in.}$$

Although the maximum span for a 2- by 8-in. joist seems slightly less than one-half the width of the slab, the supports at the beam forms will reduce the net span to about 7 ft 4 in. Thus, 2- by 8-in. joists will be satisfactory. One row of stringers will be installed under the mid-points of the joists.

**Stringers.** The stringer selected should be wide enough to transfer the loads from the joists without exceeding the allowable compression stress between the joists and the stringer. The load from a joist will be the load on an area about 7 ft 4 in. wide and 2 ft long.

$$\text{Load} = 7.33 \times 2 \times 115 = 1,685 \text{ lb}$$

$$\text{Allowable compression stress, 490 psi} \quad (\text{Table 18-1})$$

$$\text{Required area, } 1,685 \div 490 = 3.44 \text{ sq in.}$$

$$\text{Min thickness of stringer, } 3.44 \div 2 = 1.72 \text{ in.}$$

Consider a 4- by 6-in. S4S stringer. If we assume that the load coming on the stringer from the joists is uniformly distributed along the stringer, the error will not be serious.

$$b = 3.625 \text{ in.}$$

$$h = 5.625 \text{ in.}$$

$$w = 7.33 \text{ sq ft} \times 115 \text{ psf} = 845 \text{ lb per lin ft}$$

The maximum span, based on bending stress, is

$$l = 4.464 \times 5.625 \sqrt{\frac{1,800 \times 3.625}{845}}$$

$$= 69.7 \text{ in.}$$

The maximum span, based on shearing stress, is

$$l = \frac{16 \times 155 \times 3.625 \times 5.625}{845}$$

$$= 60 \text{ in.}$$

The maximum span, based on a deflection of  $\frac{1}{8}$  in., is

$$l = 5.51 \times \sqrt[4]{\frac{1,600,000 \times 53.80 \times 1}{845 \times 8}}$$

$$= 60 \text{ in.}$$

The stringers will be supported on commercial shores, whose capacities are 3,500 lb each for the length required. Based on the capacities of the shores the maximum spacing of the shores will be

$$l = 3,500 \div 845 = 4.15 \text{ ft}$$

The spacing of the shores will be reduced to 4 ft in order that 16-ft-long stringers may be used.

Table 18-7 gives information for the design of joists and stringers for forms for concrete slab, based on a fiber stress in bending of 1,800 psi.

**Forms for Beams.** Figure 18-7 shows a common method of constructing forms for concrete beams for a beam-and-slab-type floor system.

Figure 18-8 shows a modified method of constructing and supporting beam forms which was developed by one contractor [2]. The assembly was designed to be fastened together by wedges, with no erection nailing required. The primary objectives were to reduce the damage to the forms in order to obtain more uses and to reduce the amount of labor required to erect and remove the forms.



TABLE 18-7. SAFE SPANS FOR FORMS FOR CONCRETE SLABS BASED ON USING 1-IN. DECKING AND JOISTS SPACED 2 FT 0 IN. ON CENTERS\*

Thickness of slab, in. ....	4	5	6	8	10
Total load, psf. ....	90	103	115	140	165

## Safe span for joists of indicated sizes

Size joist, in.:					
2 × 4 S4S	5 ft 0 in.	5 ft 0 in.	4 ft 6 in.	4 ft 0 in.	4 ft 0 in.
2 × 6 S4S	7 ft 0 in.	7 ft 0 in.	6 ft 6 in.	6 ft 0 in.	5 ft 6 in.
2 × 8 S4S	9 ft 6 in.	9 ft 0 in.	8 ft 6 in.	8 ft 0 in.	7 ft 6 in.
2 ft 10 S4S	11 ft 0 in.	11 ft 0 in.	10 ft 6 in.	9 ft 6 in.	9 ft 0 in.

## Safe span for stringers of indicated sizes

Size stringer, in.:	Stringer spacing:					
2 × 8 S4S	6 ft 0 in.	5 ft 6 in.	4 ft 6 in.	4 ft 0 in.	3 ft 6 in.	4 ft 6 in.
2 × 10 S4S	7 ft 0 in.	5 ft 6 in.	4 ft 6 in.	4 ft 0 in.		
4 × 4 S4S	5 ft 0 in.	4 ft 6 in.	4 ft 0 in.	4 ft 0 in.	3 ft 6 in.	
4 × 6 S4S	6 ft 0 in.	6 ft 0 in.	6 ft 0 in.	5 ft 6 in.	5 ft 0 in.	4 ft 6 in.

\* The safe spans are based on a fiber stress of 1,800 psi of net size for the joist or stringer. Wood capable of resisting this stress must be used, or the span must be reduced.

The safe span for the 2- by 8-in. and 2- by 10-in. stringers is based on an allowable compressive stress of 500 psi on the area between the stringer and a 4- by 4-in. S4S shore.

Bottom forms were  $\frac{5}{8}$ -in. plywood stiffened by a continuous 2- by 3-in. plank, fastened beneath and flush with each long edge. Nailed under the 2- by 3-in. stiffeners were 1- by 4-in. transverse cleats, which extended far enough out on each side to carry a continuous longitudinal 2- by 3-in. ribband nailed to their tops. Between the stiffened edge of the bottom sheathing and the inner beveled face of the ribband there was a space for a side-form panel to be rested on the cleats and held tightly by wood wedges. The  $\frac{5}{8}$ -in. plywood sheathing of the beam side forms carried two continuous horizontal 2- by 4-in. battens. The lower batten coincided with the bottom edge of the sheathing, while the upper batten was located to serve as a ledger for notched joists that supported the  $\frac{5}{8}$ -in. plywood decking for the floor slab.

Beam forms rested on a framework of transverse 4- by 4-in. joists atop longitudinal 4- by 6-in. stringers, which were supported by 3- by 4-in. shores. To hold the members in place, yet make removal easy, cleats on the underside of the stringers fitted over the tops of the shores, whose opposite sides were fitted with scabs to keep them centered under the stringer.

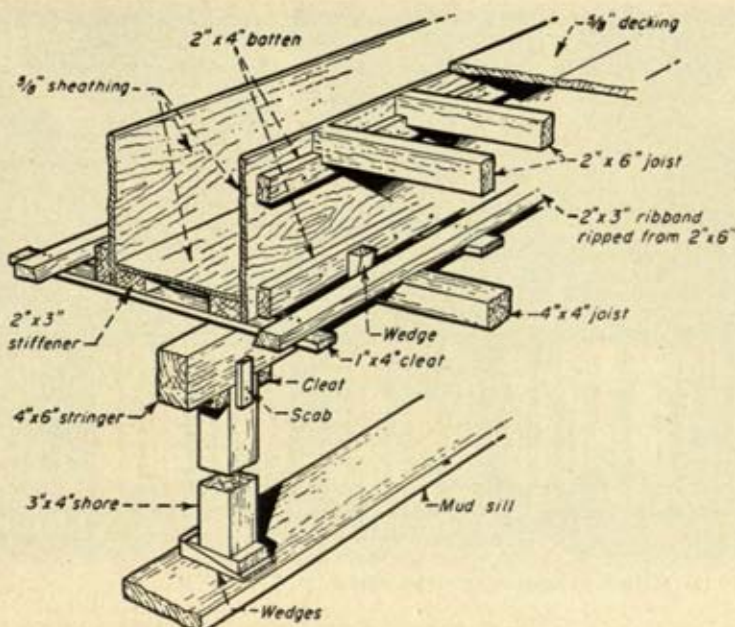


FIG. 18-8. Beam-and-slab forms designed to be used without nails. (*Construction Equipment and Methods.*)

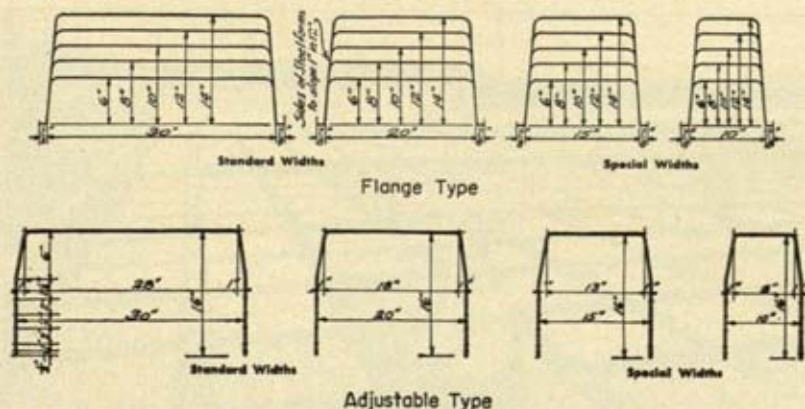


FIG. 19-9. Cross-section dimensions of Meyer Steelforms. (*Ceco Steel Products Corp.*)

**Forms for Metal-pan- and Concrete-joist-type Concrete Floors.** This type of concrete floor is proving to be very popular for several reasons. The amount of concrete required to support a given floor loading is less than is required for a beam-and-slab-type floor. The combined cost of forms and concrete generally will be about 10 per cent less than the cost



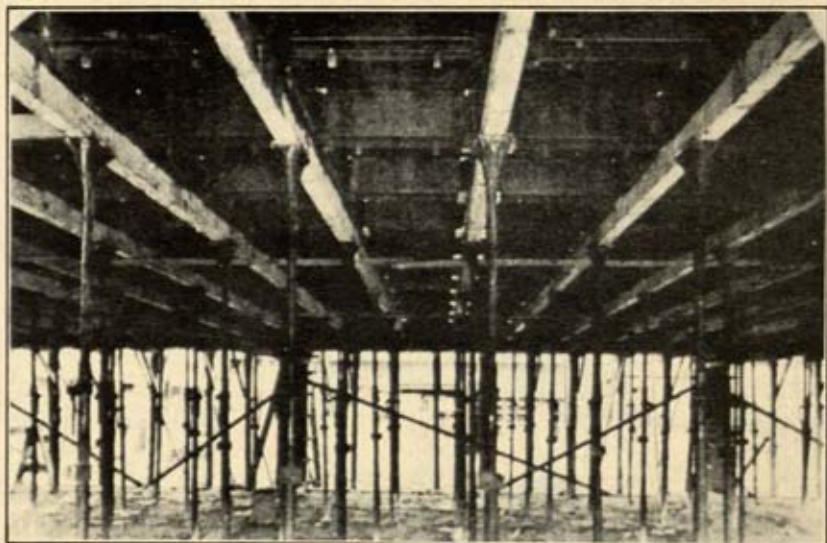


FIG. 18-10a-c. Metal-pan-type concrete-floor construction. (*J & B Manufacturing Co.*) (a) Method of supporting metal pans.

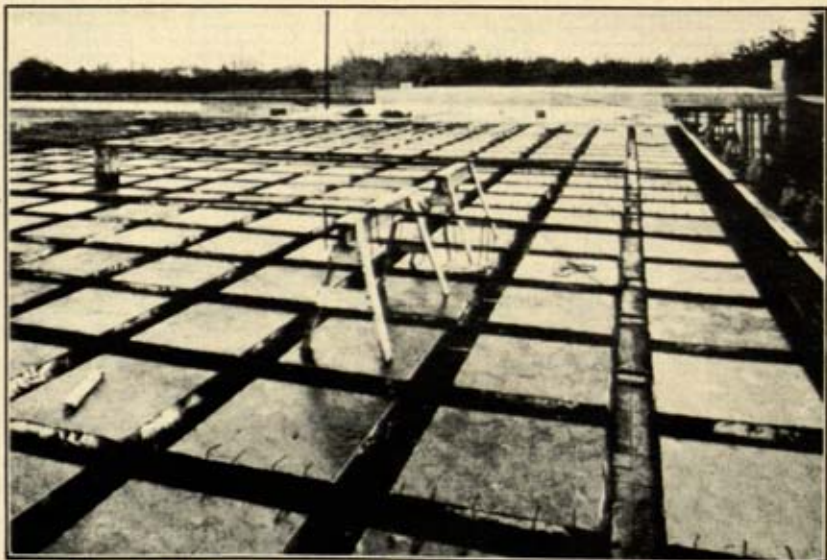


FIG. 18-10b. Top view of forms prior to placing concrete.

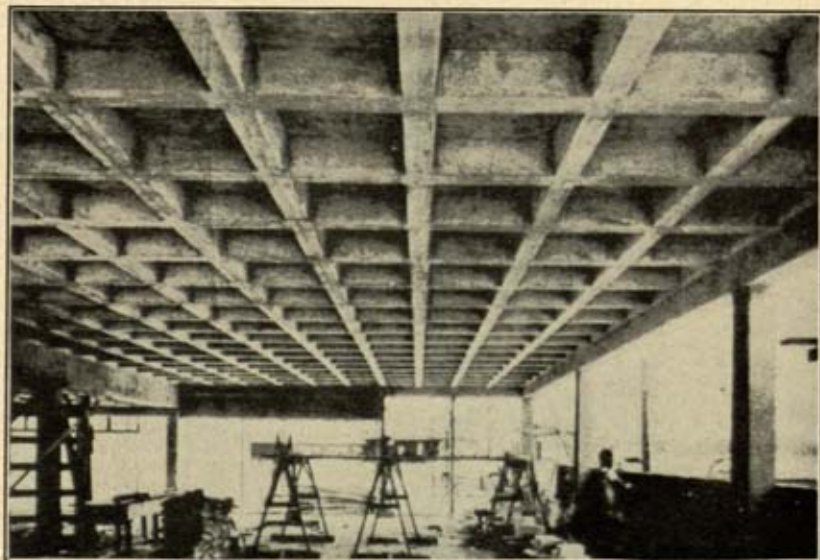


Fig. 18-10c. View of the underside of the concrete floor.

for a beam-and-slab-type floor. The reduction in the weight of the concrete floor may permit the use of smaller and less expensive beams, columns, and footings. Figure 18-9 gives the cross-section dimensions of two types of metal pans. Figure 18-10 shows one type of metal pans in place, the method of supporting the pans, and a view of the underside of the concrete floor after the pans were removed.

Metal pans may be rented for a given project, or the furnishing, installation, and removal may be sublet to a contractor who specializes in this type work.

**Cellular-steel Forms for Concrete Floors.** Lightweight cellular-steel panels may be used for the forms and structural units of floor systems. These units offer a number of advantages when compared with other types of floor systems, such as light weight, the elimination of forms for the concrete slab, rapid construction, and a supply of raceways for electric utility services. Figure 18-11 shows a typical section of an installation.

The units are manufactured from steel shapes and plates, which are electrically welded together to form various sections. They are available in depths up to 9 in., widths up to 24 in., and lengths up to 25 ft. The units are installed on and welded directly to the steel beams of a structure, or they may be supported by concrete beams.

The combined cost of the cellular units and the concrete slab usually will exceed the cost of a conventional concrete-slab floor, but the value of



the saving in construction time plus the liberal supply of utility ducts will justify the additional cost on some projects.

**Combined Corrugated-steel Forms and Reinforcement for Concrete Floors.** A complete-floor system which is suitable for floors supporting light to heavy loads may be constructed with a combined corrugated-steel form and reinforcing unit plus concrete. The sheets serve as longitudinal reinforcing for positive moment, while transverse wires, welded across the

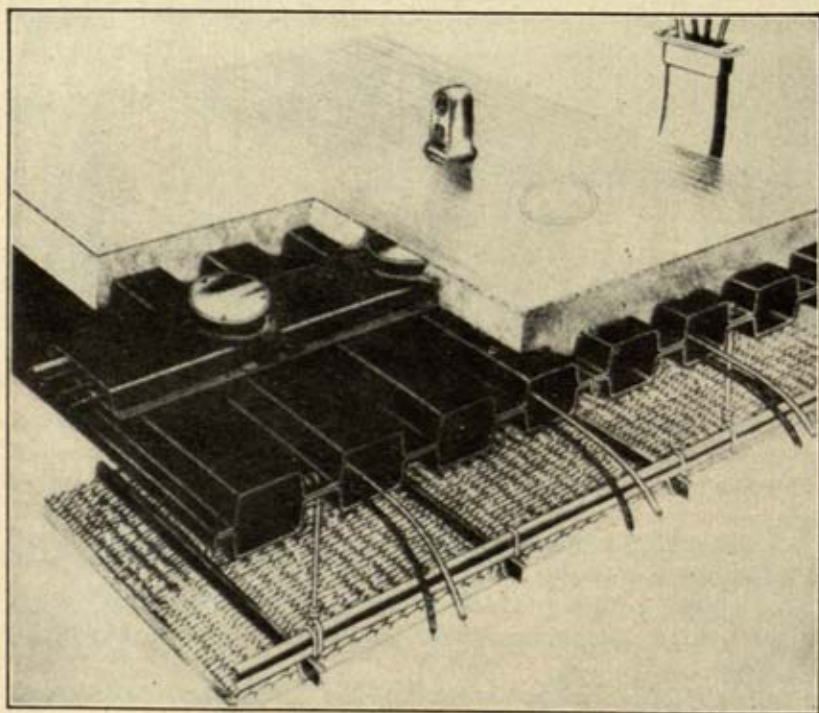


FIG. 18-11. Typical section of a floor system using cellular-steel units. (H. H. Robertson Company.)

corrugations by the manufacturer, provide temperature reinforcement in the slab as well as mechanical anchorage between the concrete and the sheets. Figure 18-12 shows a typical installation of this type form. The name of this product is Cofar.

The combined cost of the forms, reinforcing and concrete slab should be about 15 to 20 per cent less than for a comparable flat slab floor.

**Tilt-up Concrete Walls.** A method of building concrete walls that is proving economical and successful is to cast the walls in panels on the concrete-floor slab, let them cure a few days, then tilt them into position.

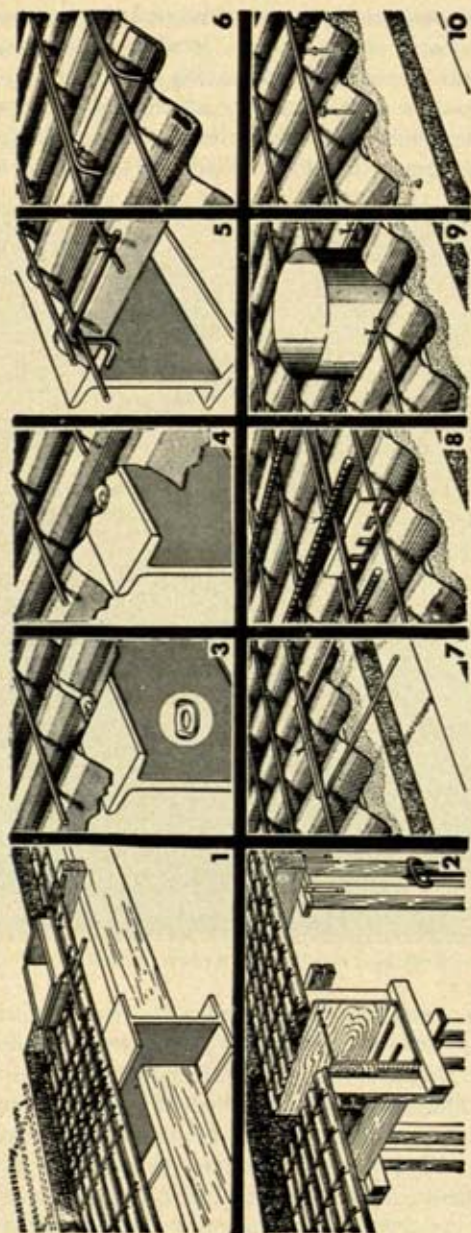


Fig. 18-12. Installation of corrugated-steel forms. (Granco Steel Products Company.)



This type of construction has several advantages compared with conventional cast-in-place concrete walls. It affords a substantial reduction in the cost of forms, permits the reinforcing and concrete to be placed at floor level, reduces the cost of the structure, and permits considerable saving in the time required for construction. As an example illustrating the speed of constructing by this method, for a building 525 ft long by

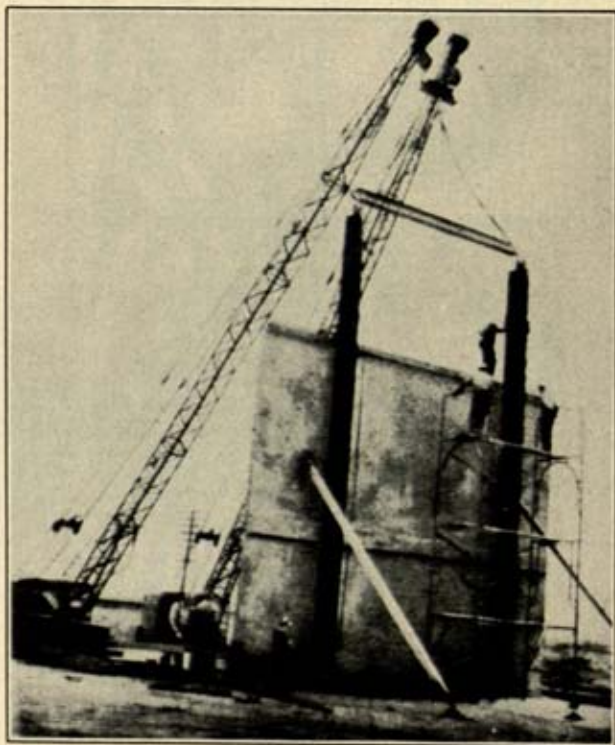


FIG. 18-13. Two cranes working together tilt a panel into position. (*The Constructor*.)

164 ft wide, with a wall 26 ft high, 4 weeks were required to place the concrete floor, which was used as a casting bed, cast the wall panels, allow them to cure, tilt the panels into position, and cast in place the supporting wall columns. The actual lifting of the wall panels required 26 hr [3].

The wall panels usually are cast on the concrete floor of the building, which is covered with a material to prevent the concrete in the panel from bonding to the floor. Sheet material, such as paper or canvas, may be used, but liquid compounds have proved more satisfactory for this purpose. Extreme care must be used to be sure that all the floor under a panel is treated with the compound.

The only forms required are placed around the edges of a panel. They may be wood or steel, wood being more popular. The reinforcing may be assembled in a mat and lifted into the panel form. Holes in the forms permit the reinforcing bars to project outside the edges of a panel. Frames for window, door, or other openings or utility conduits may be installed prior to placing the concrete. If a strongback is to be used in lifting a panel, nuts or coils should be installed before the concrete is

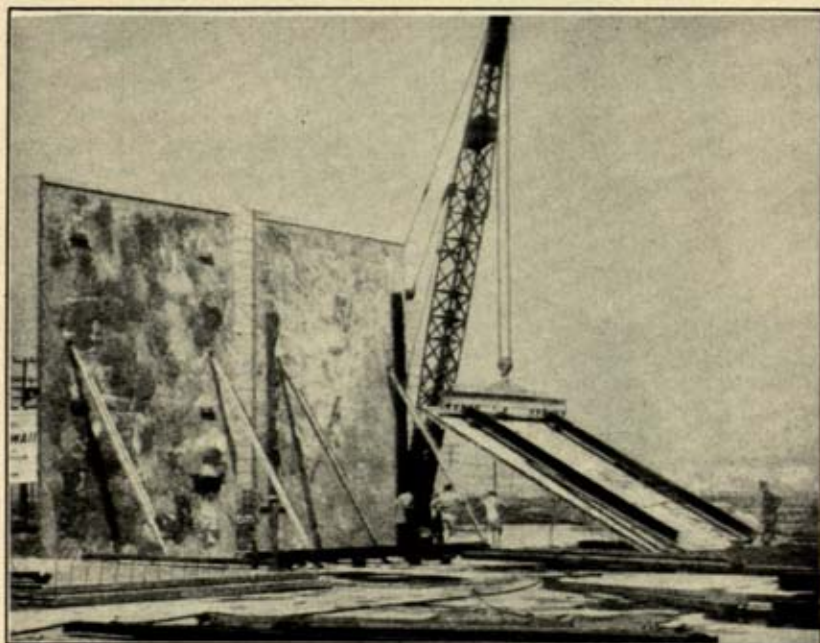


FIG. 18-14. Tilting wall panels 6 in. thick and 28 ft high into position. (*Western Construction.*)

placed. Bolts screwed into them will be used to attach the strongback to a panel.

When the panels have cured sufficiently, they are tilted or lifted into position, usually by one or more power cranes. Temporary bracing is attached, the panels are aligned and plumbed, then connected into an integral wall by means of cast-in-place concrete columns or pilasters.

The walls, which usually are about 6 in. thick, may be made of monolithic concrete, or they may be cast with a lightweight insulating concrete on the inside face. Also, a special insulating material may be sandwiched between two layers of ordinary concrete, if desired.

Figure 18-13 shows two cranes lifting a 32- by 30-ft panel, weighing approximately 35 tons. The slab was 6 in. thick, while the two vertical



and two horizontal beams were 12 in. thick. Strongbacks were attached to the vertical beams to reduce the danger of breaking a panel during the lifting operation. It required about 15 min to set a panel in position.

Each panel was cast with a beveled tongue along the lower edge, which slipped into a keyway cast into the supporting floor, to facilitate the alignment of the panels. The supporting base was covered with a cement

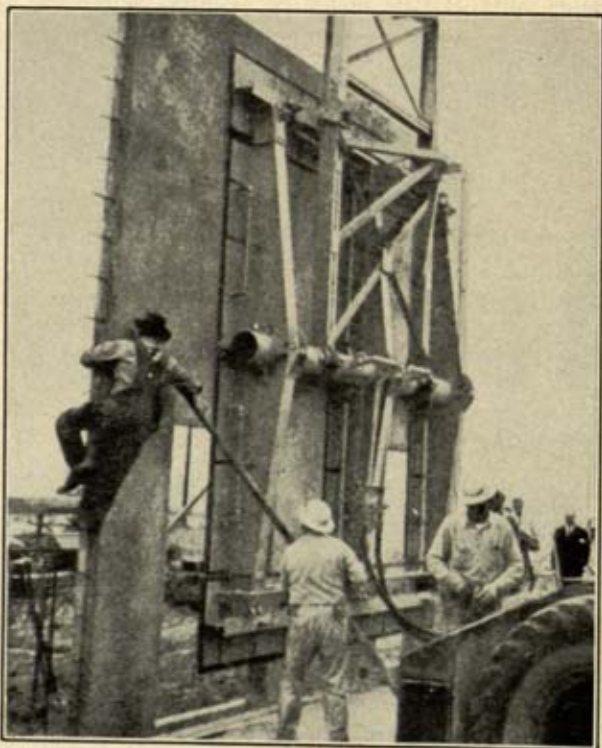


FIG. 18-15. Setting a wall panel in position with a vacuum lifter. (*Western Construction.*)

grout prior to placing the wall panels. Each of the two braces attached to the panel has an adjustable turnbuckle near the lower end to assist in plumbing the panel [4].

Figure 18-15 shows a vacuum lifter setting a wall panel in position. This lifter, which eliminates the use of strongbacks, is handled by a power crane. Note that the lifter can be rotated approximately  $180^\circ$  about a horizontal axis [5].

Figure 18-16 shows a six-man crew and a crane lowering a precast concrete wall panel into place. This is a modification of tilt-up construction.

**Slip-form Construction.** Slip-form construction is a method which permits the forms to be raised while the concrete is in a plastic state. By placing the concrete continuously for an entire structure, it is possible to eliminate construction joints, which is especially desirable when building

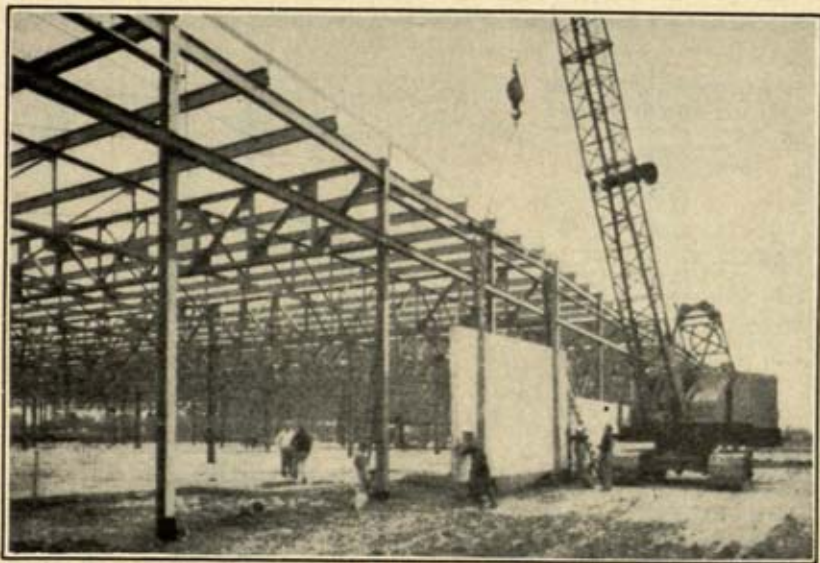


FIG. 18-16. Setting a precast wall panel in position with a six-man crew and a crane. (*Engineering News-Record.*)

structures that must be watertight. This method has several advantages compared with conventional construction, such as:

1. The elimination of construction joints
2. A reduction in form costs
3. Rapid construction

Projects which are especially suited to slip-form construction include silos, bins, tall rectangular buildings, bridge piers, chimneys, and other structures having reasonably high uniform cross sections. Under favorable conditions it is possible to erect a structure at a rate of 15 to 20 ft per day. A structure should be reasonably tall, usually 30 to 40 ft, or more, to justify the relatively high cost of preparing the forms for this type of construction.

**The Forms for Slip-form Construction.** For a structure consisting of walls it is necessary to assemble forms for the inside and outside walls in the correct position. The wall forms are made of vertical staves, such as 1- by 4-in. tongue and groove flooring, usually about 4 ft long, which serve as the sheathing. The sheathing is nailed to two or three wales,



made from 2-in.-thick lumber, cut to the correct shape for the inside or outside forms. The space between the forms is made equal to the thickness of the wall at the top, and about  $\frac{3}{8}$  in. thicker than the wall at the bottom to facilitate the movement of the forms as they are lifted.

The faces of the sheathing are held a uniform distance apart by hairpin yokes, made of heavy timber or steel sections, which are installed 6 to 8 ft

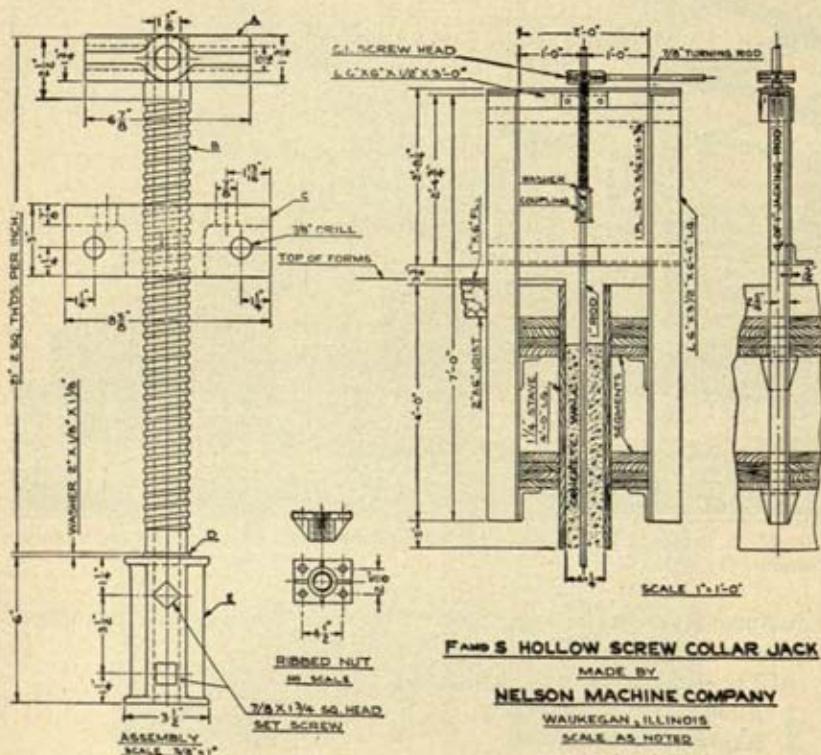


FIG. 18-17. Details of a jack and yoke assembly for lifting slip forms. (Nelson Machine Company.)

apart around the wall. The top portions of the yokes are well braced to assure rigidity. The lower portions of the yokes are fastened to the wales, as shown in Fig. 18-17. Thus, the yokes serve two purposes: they hold the forms in the correct position, and they transfer the lifting forces to the forms. The area inside the forms, except over the walls, is covered with decking to provide a platform for the workers and for the storage of necessary materials.

Figure 18-18 shows representative details of the sheathing, working platform, and scaffold assembly for slip forms.

**Lifting the Forms.** Several types of jacks are used to lift the forms. One method is to install a hollow screw jack with each yoke, as shown in Fig. 18-17. A smooth steel jackrod, whose bottom is embedded in the concrete, passes through the jackscrew. The base of the screw may be fastened to the jackrod with setscrews, or it may grip the rod to prevent the jack from slipping downward as the lifting force is applied. The screw is free to rotate in the base. As the screw is turned by means of a lever inserted in the head at the top, it forces a nut, which is attached to

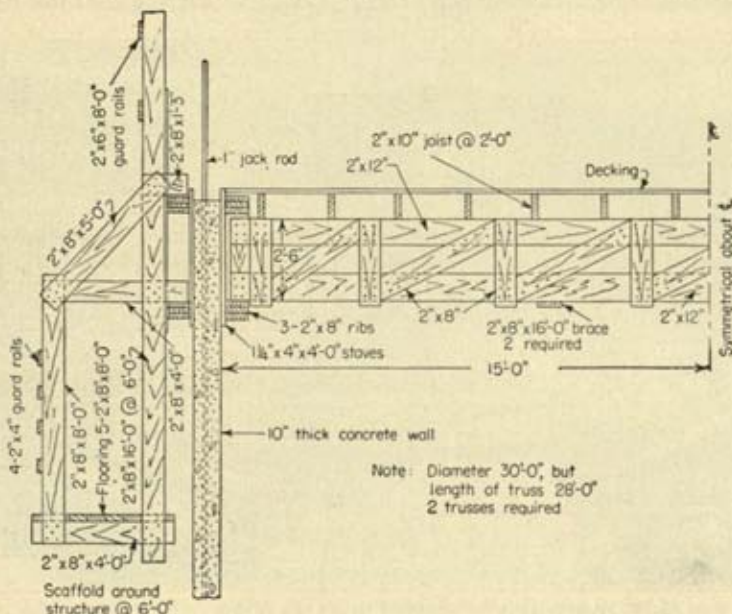


FIG. 18-18. Details of working floor and scaffold assembly for slip forms. (By permission from "Estimating Construction Costs," by R. L. Peurifoy. Copyright 1953, McGraw-Hill Book Company, Inc.)

the underside of the top cross member of the yoke, to move upward on the screw. This lifting force is transferred to the yoke and thence to the forms.

When a jack reaches the end of a rod, a new rod can be welded on top or the length can be extended with a sleeve and a new rod. The tops of the rods should be staggered in order that there will always be solid rods for some of the jacks.

One man should be able to handle about 36 jack-in. per hr. Thus, if the lifting rate is 9 in. per hr, a man can operate four jacks.

The rate of lifting the forms depends on the temperature of the concrete, which controls the rate of set. If the forms are lifted too rapidly,



the concrete will bulge as it leaves the bottom of the forms, which may endanger the structure. If the forms are lifted too slowly, the concrete will stick to the forms and increase the difficulty of lifting. The rate of lifting should be under the supervision of an inspector who tests the degree of set by thrusting a steel rod into the concrete from time to time.

The reinforcing steel and the concrete are placed as the forms move upward.

Care must be exercised to keep the wall vertical. One method of accomplishing this is to attach to each yoke a calibrated glass tube which

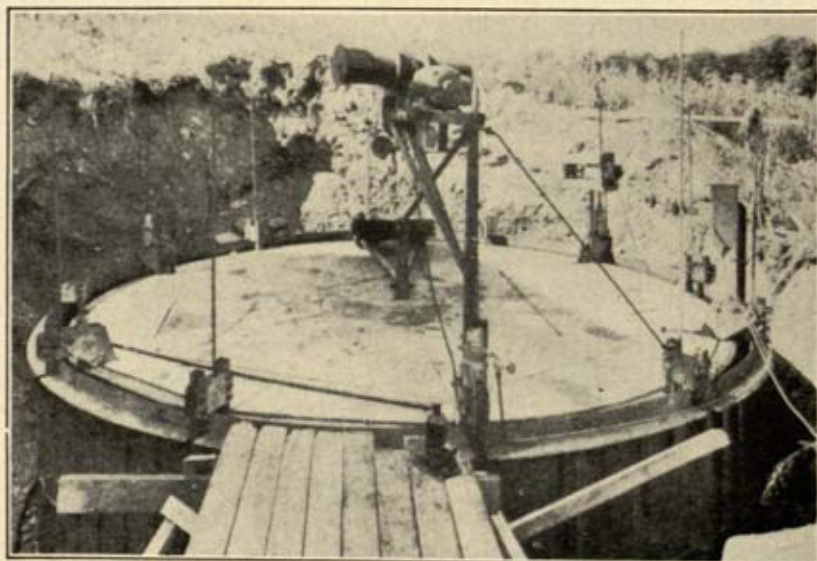


FIG. 18-19. Hydraulic jacks installed to lift slip forms. (B. M. Heede, Inc.)

is connected through a rubber hose to a common reservoir containing water. The jacks should be operated to keep the level of the water in each tube at a fixed position.

**Lifting with Hydraulic Jacks.** Within recent years a hydraulic jack has been developed which seems to be superior to the screw jack. The jacks are supplied with oil under a high pressure by a single electric-motor-driven pump. As the oil pressure is applied, the jacks are extended simultaneously at the same rate. When the pressure is reduced, each jack is automatically retracted by lifting the lower jaw pieces, while the top jaws grip the jackrod to prevent downward movement. With the reapplication of pressure the jacks resume the upward movement. This principle automatically assures that each jack will move upward at the same rate. One man can operate the pump and the jacks. The rate of

lift can be varied up to 20 in. per hr. The jacks are available in two sizes, with capacities of 3 and 6 tons.

Figure 18-19 shows a set of slip forms in place with some of the hydraulic jacks installed. Figure 18-20 shows a hydraulic jack in operation.

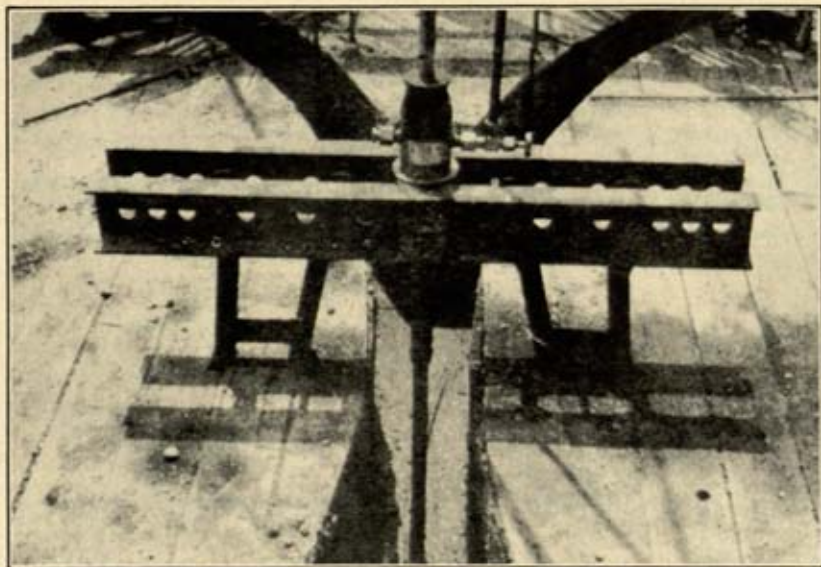


FIG. 18-20. Hydraulic jack in operation. (B. M. Heede, Inc.)

### PROBLEMS

**18-1.** Determine the maximum spacing of studs for 1-in.-nominal-thickness sheathing when the forms for a concrete wall will be filled at a rate of 3 ft per hr and the temperature of the concrete is 80 deg. The maximum permissible deflection of the sheathing is  $\frac{1}{8}$  in. Grade 1 southern pine will be used for the sheathing.

Limit the spacing to the value determined by deflection, bending stress, or shearing stress.

**18-2.** A concrete wall, 140 ft long, 12 ft high, and 10 in. thick, will be built in one pour. The forms will be filled at the rate of  $2\frac{1}{2}$  ft per hr. The temperature of the concrete will be 70 deg.

The sheathing will be 1 by 6 in., the studs 2 by 4 in., and the wales double 2 by 4 in., all S4S lumber. Form ties will be 3,000 lb allowable working stress. Use the information given in Table 18-4 to design the forms.

Prepare a bill of materials for the lumber, including braces on one side and form ties required for the forms. List each size of lumber as follows:

76 pc      2 × 4 in. × 16 ft      810 fbm

**18-3.** Wood forms will be used to support the concrete for a slab 6 in. thick. The clear width of the slab, between the beam sides, will be 14 ft 10 in. The decking will be 1-in.-nominal-thickness lumber. Determine the number of rows of stringers



required to support the joists, based on using 2- by 6-in. S4S and 2- by 8-in. S4S joists. Check the joists for bending and shearing stresses.

For each size joist determine the maximum spacing for commercial shores which will support a load of 4,500 lb each.

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## CHAPTER 19

### CONCRETE

**Introduction.** Concrete is basically cement, aggregate, and water which have been mixed together, deposited, and permitted to solidify. Sometimes admixtures are used for various purposes, such as to produce a desired color, improve the workability, entrain air, reduce the segregation, or accelerate setting and hardening.

The operations in the production of concrete will vary with the type of project requiring the concrete and the type of concrete produced. In general, the operations, which are represented graphically in Fig. 19-1, include the following:

1. Batching the materials
2. Mixing
3. Handling and transporting
4. Placing
5. Finishing
6. Curing

**Design of Concrete Mixtures.** The design of a concrete mixture involves determining the proper proportions of cement, aggregate, and water, plus any admixtures, to produce a concrete having the desired properties. Because of the many variables which affect the design of concrete for a given project it is impossible to include in this book specific information which can be used safely as a guide under all conditions.

However, some general information will apply to all projects.

The fine and coarse aggregate constitute about 75 to 80 per cent of the ultimate mass of concrete. They are joined together into a solid body by the solidification of a paste made of cement and water. The water serves two purposes. It causes the cement to hydrate into a solid mass, and it gives the fresh concrete sufficient plasticity to permit it to be formed into the desired shape. If an excess quantity of water is used, it

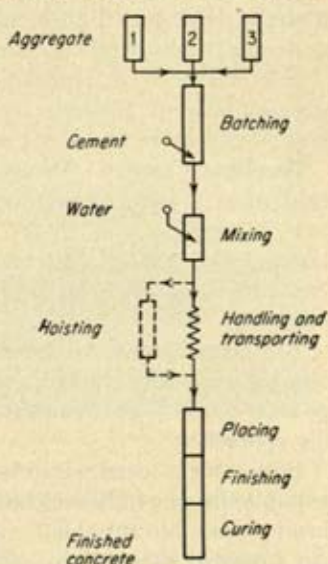


FIG. 19-1. Flow diagram showing the operations performed in constructing a concrete project.



dilutes the paste and weakens the concrete. If an insufficient quantity of cement is used, there will not be enough paste to join all the particles of aggregate together solidly and the strength of the concrete will be affected. Thus, it seems apparent that the quantity of water used in making concrete should be the minimum amount needed to give the concrete the required plasticity.

Economical concrete, having the required properties, can be produced by using the largest practical sizes of coarse aggregate and the smallest practical quantity of water. Large pieces of aggregate are already joined together by nature, and they require no cement for this purpose. If the water content is kept low, the strength of the cement paste will be high and a strong concrete can be produced with less cement.

**Handling and Batching Materials.** In order to produce concrete having the required properties, it is necessary to control the quantity of each material that goes into a batch. This is referred to as batching the materials. Although batching may be done by volume or by weight, the former method is so unreliable that it should not be used on any job where the properties of concrete are of importance. Weight batching is much more dependable and more commonly used than volume batching.

**Handling Cement.** Cement may be shipped to a job in paper bags, containing 1 cu ft and weighing 94 lb, or as bulk cement in special railroad cars, in boxcars, or by trucks. Bulk cement is cheaper than bag cement, but unless a job is large enough to justify the installation of facilities to handle bulk cement, it will be more satisfactory to use cement in bags.

Bag cement must be stored in a dry place and should be left in the original bags until used for concrete. If a batch of concrete requires one or more whole bags of cement, the use of bag cement simplifies the batching operation.

Bulk cement usually is unloaded from the cars or trucks and stored in a suitable silo or a fully enclosed overhead bin. Figure 19-2 shows an overhead cement bin suitable for storing one or more railroad cars of cement. An auxiliary silo may be added to increase the storage capacity. The cement flows from the bottom hopper of a railroad car into an undertrack screw conveyor to a bucket-type elevating conveyor and thence into the overhead storage bin.

A weighing hopper, suspended beneath the storage bin, is used to measure the correct amount of cement. Weighing may be done by means of a beam scale or a springless dial scale, the latter being more expensive but more dependable.

**Batching the Aggregate.** The specifications for a project may require that concrete be made with aggregate having two to six different size ranges. The quantity of material from each size range must be measured

carefully. It is the function of the batching equipment to perform this measuring operation.

If a project is not large enough to justify the use of more elaborate equipment, satisfactory batching may be obtained by using wheelbarrow scales. These scales, which are illustrated in Fig. 19-3, usually are equipped with a tare beam, to adjust for the empty weight of the wheelbarrow, plus three weight beams, generally of 500 lb capacity each, and a balance indicator. The net weight of each wheelbarrow of material, by

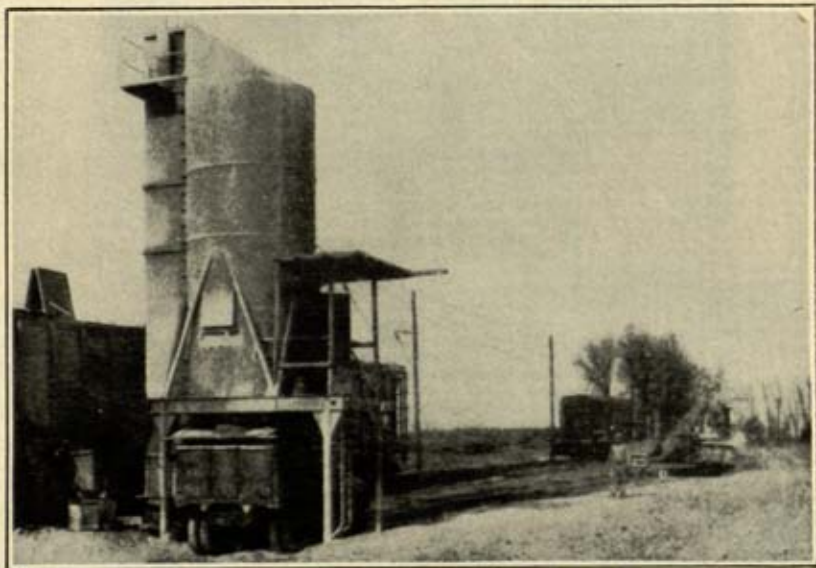


FIG. 19-2. Silo for the storage of bulk cement. (The C. S. Johnson Co.)

size, is set on an appropriate beam. Each beam may be operated independently of the other.

If a project is large enough to justify the additional investment in equipment for handling and batching aggregate, an elevated storage bin, equipped with a weighing batcher, should be used. It will be necessary to provide a clamshell, tractor-mounted scoop, or other suitable equipment to handle the aggregate from the stock pile to the bin. Figure 19-4 shows a trolley-type batching plant which may be filled with a tractor-mounted scoop. When the specified quantities of aggregate have flowed from the bins into the weighing hopper, the hopper is moved along the trolley to permit the aggregate to be discharged into the skip of the concrete mixer. Bins of this type have capacities varying from about 3 to 40 tons. The batchers have capacities varying from 1,000 to 4,000 lb





FIG. 19-3. Four-beam aggregate scales for use with wheelbarrows. (The C. S. Johnson Co.)

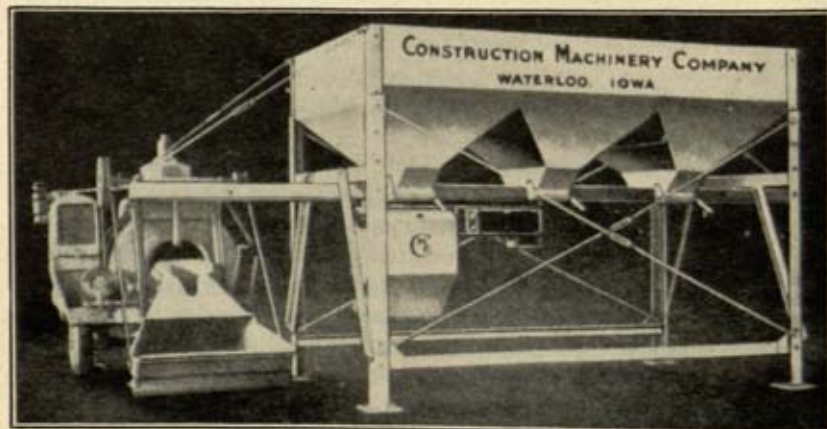


FIG. 19-4. Four-compartment trolley-type aggregate batching plant. (Construction Machinery Co.)

and are equipped with two, three, or four weighing beams, in addition to a tare beam.

For a large project it may be desirable to install one or more multiple-compartment overhead bins of the type illustrated in Fig. 19-5. This is a three-compartment storage bin equipped with a beam-type weighing batcher. Bins are available with two or more compartments, each compartment holding up to 50 tons of aggregate. The batched aggregate may be discharged into a truck, a transit mixer, or a chute and thence to a concrete mixer.



FIG. 19-5. Three-compartment aggregate batching bin. (The C. S. Johnson Co.)

The capacity of a batching bin is the sum of the capacities of the several compartments, expressed in tons or in cubic yards.

The capacity of the hopper of a weighing batcher should be at least  $1\frac{1}{3}$  times the rated capacity of the concrete mixer with which it is used.

Figure 19-7 shows the layout for storing, handling, and batching aggregates and for mixing and handling concrete for the Philpott Dam [1]. Note that a batch required four different sizes of aggregate, plus sand, two types of cement, flaked ice, and water.

**Measuring Water.** In view of the significant effect which the quantity of water has on the properties of concrete it is necessary to provide a method of accurately measuring the quantity of water per batch. Concrete mixers usually are equipped with water-measuring tanks, which may be adjusted to supply any reasonable amount of water per batch.



These tanks should be checked periodically to verify the amount of water supplied.

Other water-measuring devices include water meters and water-weighting tanks.

If the aggregate contains free surface water, such water should be included as part of the total quantity required for the concrete.

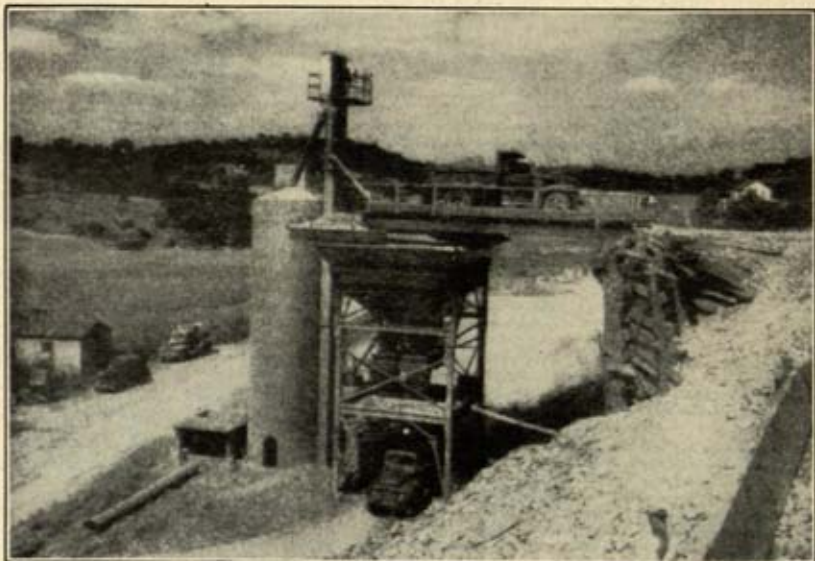


FIG. 19-6. Central batching plant for cement and aggregate. (The C. S. Johnson Co.)

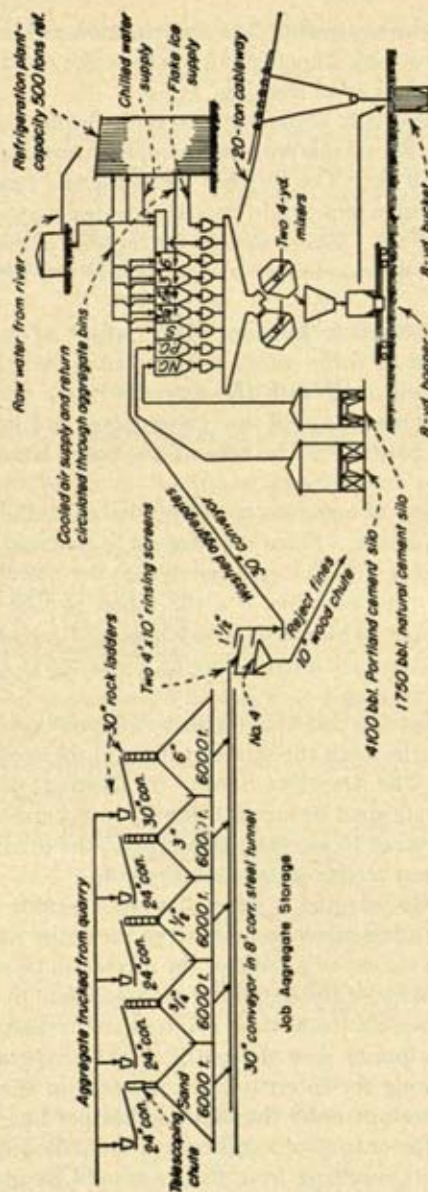
**Concrete Mixers.** For use in this book concrete mixers may be classified as

1. Construction mixers
2. Paving mixers
3. Transit mixers

The old practice of specifying the size of a mixer as one-bag, two-bag, etc., has been pretty well abandoned in favor of specifying the size by the nominal volume of concrete that can be mixed in a batch, expressed in cubic feet for construction and paving mixers and in cubic yards for transit mixers.

The Mixer Manufacturers Bureau of the AGC lists the various sizes of construction and paving mixers that shall be considered standard. Other sizes are available and may be entirely satisfactory.

For construction mixers with a single-compartment drum the standard sizes are 3½S, 6S, 11S, 16S, 28S, 56S, 84S, and 112S. The number indicates the nominal volume of mixed concrete in cubic feet, while the letter



**Fig. 19-7.** Flow diagram for the concrete-mixing plant at the Philpott Dam. (*Construction Methods and Equipment.*)



S designates that the equipment is a construction mixer. These mixers must be capable of mixing 10 per cent more than the rated capacities when they are operating in a level position.

For paving mixers with single-compartment drums the standard sizes are 27E and 34E. For mixers with two-compartment drums the standard sizes are 16E and 34E. The number indicates the nominal volume of mixed concrete in cubic feet, while the letter E designates that the equipment is a paving mixer. These mixers are capable of mixing 20 per cent more concrete than the rated capacities when they are operating on a level surface.

**Outputs of Construction Mixers.** The output of a concrete mixer usually is expressed in cubic yards of concrete mixed per hour. Obviously, the output will vary with the size of a mixer and the conditions under which it is operated. For any given mixer and job conditions the output will be the product of the volume per batch times the number of batches per hour.

The actual volume of concrete mixed per batch usually will not equal the rated size of a mixer. Since it is desirable to avoid the use of fractional bags of cement, the size of a batch may be more or less than the size of the mixer. For example, if a 16S mixer is used to mix concrete requiring six bags per cubic yard, a batch should include three bags of cement. This will produce a batch having a volume of 13.5 cu ft instead of 16 cu ft.

The number of batches mixed per hour will depend on the average time per cycle, which varies with the mixing time and the method of discharging the concrete. The American Society for Testing Materials (ASTM) specifies that concrete shall be mixed 1 min for mixer sizes through 1 cu yd and that for larger sizes 15 sec shall be added to the mixing time for each additional cubic yard of size or fraction thereof.

The method of discharging a mixer has considerable influence on the time per cycle. If a 16S mixer can discharge the entire batch into a single container such as a bucket or a hopper, the mixer can be emptied in 15 sec or less time. However, if the batch is discharged into five wheelbarrows, each requiring 10 sec, the total time required to discharge the mixer will be 50 sec. In the former case the mixer might produce 40 batches per hour without allowing for interruptions, whereas in the latter case the mixer might produce not more than 25 batches per hour.

In determining the output of a mixer over an extended period of time, any losses in output resulting from delays should be included by using an appropriate operating factor such as a 45- or 50-min hour.

**EXAMPLE.** Determine the quantities of materials required per batch and the probable output for a 16S construction mixer. The quantities of materials per cubic yard are

Cement, 5.6 bags  
 Sand, 1,438 lb  
 Gravel, 1,846 lb  
 Water, 39 gal

If the batch is 16 cu ft, the required volume of cement will be  $\frac{16 \times 5.6}{27} = 3.32$  bags. Instead of mixing 16 cu ft per batch, which would require a fractional bag of cement, reduce the quantity of cement to 3 bags, and the quantities of other materials in the same proportion. The volume per batch will be  $\frac{3 \times 27}{5.6} = 14.5$  cu ft. The quantities of materials per batch will be

Cement, 3 bags  
 Sand,  $\frac{14.5}{27} \times 1,438 = 771$  lb  
 Gravel,  $\frac{14.5}{27} \times 1,846 = 990$  lb  
 Water,  $\frac{14.5}{27} \times 39 = 20.9$  gal

If the mixer discharges the entire batch of concrete into a single bucket, the time per cycle should be about as follows:

Charging mixer = 0.25 min  
 Mixing concrete = 1.00 min  
 Discharging mixer = 0.25 min  
 Lost time = 0.10 min  
 Total time = 1.60 min

No. batches per hr,  $60 \div 1.60 = 37.5$

Output per hr,  $37.5 \text{ batches} \times 14.5 \text{ cu ft per batch} \div 27 = 20.1 \text{ cu yd}$

The output in a 50-min hr will be  $20.1 \times \frac{50}{60} = 16.7 \text{ cu yd}$

Table 19-1 gives representative ranges in the output of standard sizes of construction mixes.

TABLE 19-1. REPRESENTATIVE RANGES IN THE OUTPUTS FOR CONSTRUCTION MIXERS

Size mixer	Time per cycle, min		Batches per hr		Output,* cu yd per hr	
	Min	Max	Min	Max	Min	Max
3½S	1.5	2.25	27	40	3.5	5.2
6S	1.5	2.25	27	40	6.0	8.9
11S	1.5	2.5	24	40	9.8	16.3
16S	1.5	2.5	24	40	14.2	20.1
28S	1.75	2.75	22	34	22.6	35.3
56S	2.00	2.75	22	30	45.6	62.3
84S	2.25	3.00	20	27	62.2	84.0
112S	2.50	3.25	18	24	74.5	99.5

\* These values are based on a 60-min hour and should be adjusted to fit actual job conditions.



**Central Mixing Plants.** A central mixing plant may be installed to mix concrete for a large structure, such as a dam, or for sale to the public. Such a plant includes equipment for handling and storing aggregate and cement, batchers, and one to four construction-type concrete mixers in sizes from 28S to 112S. The mixers may be tilting or nontilting types. The mixed concrete may be discharged into buckets, agitator trucks, or dump trucks if air-entrained concrete is used.

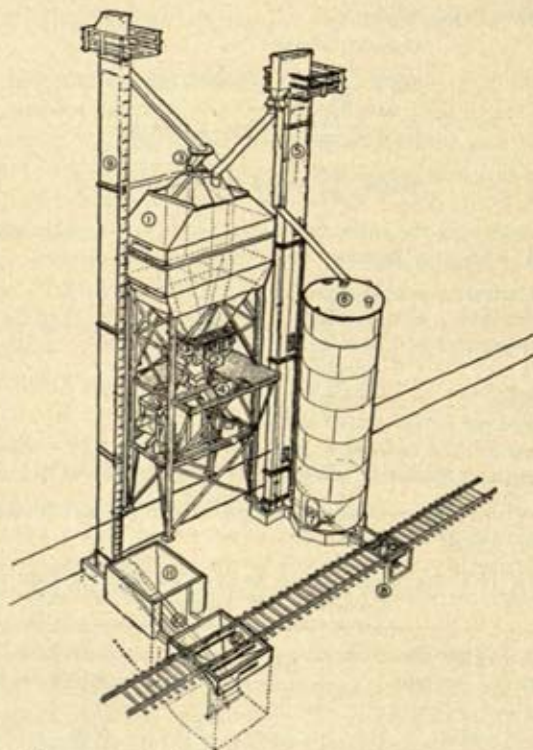


FIG. 19-8. Layout for a central mixing plant. (*Construction Methods and Equipment.*)

Figure 19-8 illustrates one arrangement for a central mixing plant, using a single nontilting concrete mixer. The plant layout can be revised to permit the delivery of cement and aggregate by truck or by any other desired method of transportation.

**Paving Mixers.** Paving mixers are used primarily to mix and place concrete for highways, streets, and airport runways. They are mounted on crawler tracks in order that they may move along with the placing of the concrete. Figure 19-10 illustrates a paving mixer in operation.

As previously stated under Concrete Mixers, the Mixer Manufacturers Bureau of the AGC specifies as standard the 27E and 34E single-drum and the 16E and 34E double-drum units. As illustrated in Fig. 19-11, the double-drum mixer has two compartments. The aggregate is charged into the first compartment, where it is premixed, following which it is transferred to the second compartment as soon as this compartment is

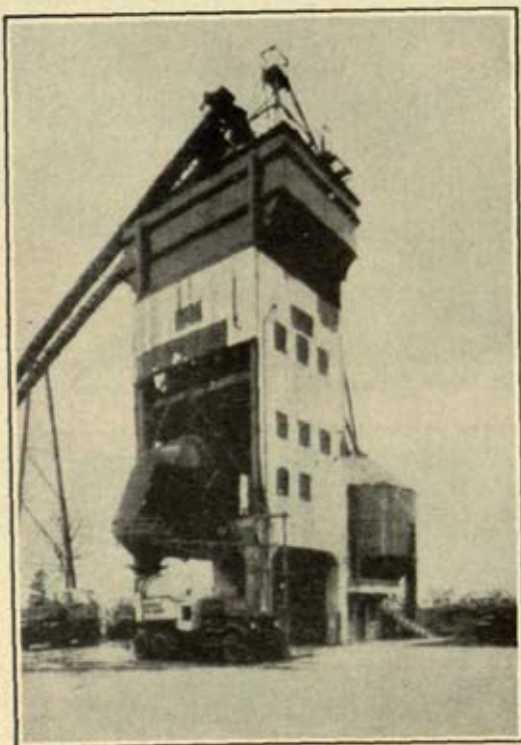


FIG. 19-9. Central mixing plant with tilt-type mixer. (The T. L. Smith Co.)

emptied. This operation permits a substantial increase in the output of a double-drum mixer compared with a single-drum unit.

The aggregate is hauled to a paving mixer in dump trucks whose beds are divided into two or three compartments, each compartment being large enough to hold one batch of aggregate.

Table 19-2 gives recommended sizes of aggregate bins, clamshell buckets, and cranes for batching plants for paving mixers [2].

The output of a paving mixer will vary with the size of the mixer, the number of compartments, and the nature of the job. Under favorable conditions a paving mixer can mix a 20 per cent overload of concrete.



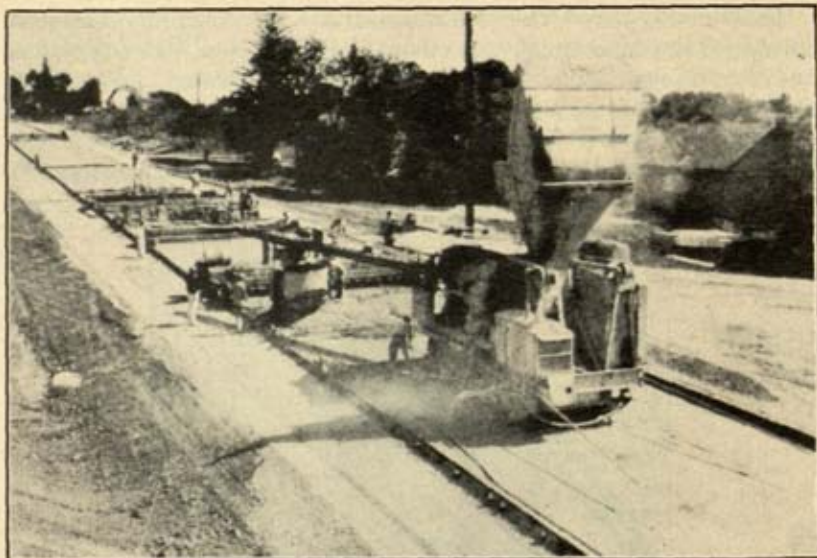


FIG. 19-10. Paving mixer in operation. (Koehring Co.)

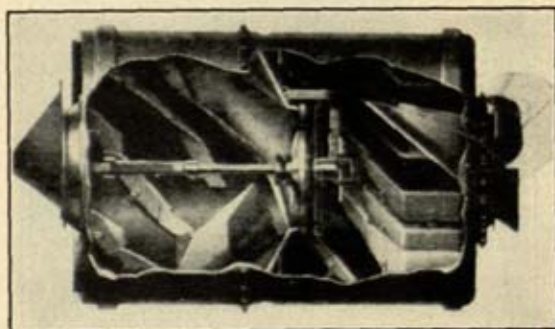


FIG. 19-11. Section through a double-drum paving mixer. (Chain Belt Co.)

TABLE 19-2. RECOMMENDED BATCHING-PLANT EQUIPMENT FOR PAVING MIXERS

Size mixer	Min size bin, tons	Size clamshell bucket, cu yd	Size crane, cu yd	Length of boom, ft
One 24E single-drum.....	75	$\frac{3}{4}$	$\frac{3}{4}$	45
One 34E single-drum.....	75	1	1	45
One 16E double-drum.....	50	$\frac{1}{2}$	$\frac{1}{2}$	40
One 34E double-drum.....	100	$1\frac{3}{4}$	$1\frac{1}{2}$	50
Two 34E double-drum.....	190	3	$2\frac{1}{2}$	60

The batch cycle for a single-drum mixer should run about 1.5 to 2 min and for a double-drum mixer about 0.8 to 1.25 min. While the lower cycle times are possible, it is not probable that they will be maintained over an extended period of time except under favorable conditions. For example, in paving city streets, requiring curbs and gutters, frequent driveway entrances, intersections, and other delay-producing operations, the actual operating factor may be as low as 0.5, corresponding to a 30-min hour.

**EXAMPLE.** Determine the probable output of a 34E double-drum paving mixer under various conditions.

If the highway is level and job conditions are favorable, it is possible to produce a batch of concrete in 50 sec.

Max size batch,  $34 \times 1.20 = 40.8$  cu ft

Batches per hr,  $60 \times 60 \div 50 = 72$

Max output per hr,  $72 \times 40.8 \div 27 = 109$  cu yd

Output for a 45-min hour,  $109 \times \frac{45}{60} = 81.6$  cu yd

Output for a 30-min hour,  $109 \times \frac{30}{60} = 54.5$  cu yd

Table 19-3 gives representative outputs for paving mixers operating on level ground. If the slope of the ground is as great as 6 per cent, the maximum capacity per batch will be 10 per cent greater than the rated size of the mixer. For such conditions the outputs given in the table should be reduced about 10 per cent.

TABLE 19-3. REPRESENTATIVE OUTPUTS FOR PAVING MIXERS

Size mixer	Time per cycle, min		Batches per hr		Output,* cu yd per hr	
	Min	Max	Min	Max	Min	Max
27E single.....	1.5	2.0	30	40	36.0	48.0
34E single.....	1.5	2.0	30	40	45.4	60.5
16E double.....	0.8	1.25	48	75	34.2	53.3
34E double.....	0.8	1.25	48	75	72.6	113.5

\* These values are based on a 60-min hour and should be adjusted to fit actual job conditions.

**Handling and Transporting Concrete.** The method used to handle and transport concrete should be selected to accomplish several objectives, including:

1. Economy
2. The prevention of segregation
3. Final placing before concrete attains initial set

Concrete may be handled and transported by several methods, such as buggies; buckets handled by cranes, hoisting towers, or cables; chutes;





FIG. 19-12. Power-driven concrete buggies. (Gar-Bro. Mfg. Co.)

belt conveyors; trucks, transit-mix or dump; pumps and pipe lines. Each method, which has advantages and disadvantages, is suitable for use under certain conditions. The method selected should permit the use of a concrete having the required properties, such as consistency, maximum-size aggregate, etc.

In order to reduce segregation, concrete should flow vertically downward as it is discharged into the forms or from one unit of equipment to another.

**Hand Buggies.** Hand buggies or carts, equipped with pneumatic tires, which are available in sizes of 6 to 11 cu ft, are suitable for use on many projects. The smaller size will haul about 4.5 cu ft and the larger size about 9 cu ft per load. They are superior to wheelbarrows because the two wheels provide a better balance for the load.

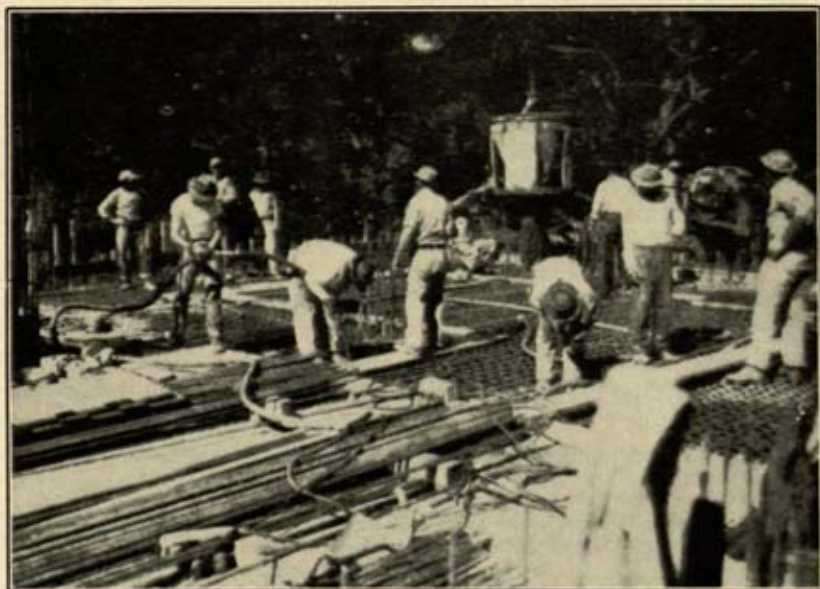


FIG. 19-13. Discharging concrete directly from bucket into forms. (Gar-Bro. Mfg. Co.)

**Power Buggies.** Within recent years power buggies have been used at increasing rates to haul concrete. They have capacities of  $\frac{1}{3}$  to  $\frac{1}{2}$  cu yd and speeds up to about 15 mph, can make a  $180^\circ$  turn in about 4 ft, and can climb grades up to about 20 per cent when loaded. On projects where they may be used advantageously, power buggies may pay for themselves in 1 to 6 months by economies which they can effect compared with the cost of transporting concrete with hand buggies.

**Buckets.** Buckets may be divided into two groups, those used with material towers, and those used with power cranes, cables, etc. The former, which are referred to as tower buckets, vary in size from about 8 to 36 cu ft, while the latter, which are referred to as concrete buckets, vary in size from about  $\frac{1}{2}$  to 8 cu yd.



Concrete buckets have bottom gates which may be opened in such a manner that the concrete will flow vertically downward. The gates on the smaller buckets are operated manually, while the gates on the larger buckets are operated by compressed air or by some other mechanical method. Gates should be designed so that they may be opened or closed at will to regulate the flow of the concrete.

Figure 19-13 shows a bucket discharging concrete directly into the forms of a structure.

**Hoisting Concrete with a Crane versus a Material Tower.** On some projects, such as multistory buildings, it may be possible to use a crane or a material tower to hoist the buckets of concrete. Each has advantages which may make it more suitable than the other under certain conditions.

The advantages of a crane and bucket are as follows:

1. Greater mobility permits the crane to deposit the concrete at different locations around the structure, provided there is access to the building, thus reducing the haul distance with buggies.
2. The crane may be used for other operations.
3. The cost of getting a crane ready to operate will be less than for a tower.

The advantages of a tower and bucket are as follows:

1. The investment in the tower and hoisting equipment will be less than for a crane.
2. The method requires less space in a congested location.

**EXAMPLE.** Compare the cost of using a crane with the cost of using a tower to hoist concrete for a building whose height will be 30 ft. The hoisting equipment will be used 150 hr. A  $\frac{3}{4}$ -cu-yd bucket will be required.

**Crane and bucket:**

Cost of crane, 8-ton, 10-ft radius	= \$15,600
Cost of concrete bucket	= 400
Total cost	= \$16,000
The cost per hour will be	
Crane	= \$ 5.10
Bucket	= 0.15
Crane operator	= 2.25
Crane oiler	= 1.50
Hopperman	= 1.25
Total cost per hr	= \$10.25

**Tower and bucket:**

Cost of tower, heavy, single, 50-ft	= \$ 1,100
Cost of hoisting unit, 15-hp	= 1,520
Cost of tower bucket	= 300
Cost of wire rope	= 200
Cost of setting up and taking down	= 150
Total cost	= \$ 3,270

The cost per hour will be		
Erecting tower, \$150 ÷ 150 hr	=	\$ 1.00
Tower	=	0.30
Hoisting unit with rope	=	0.75
Bucket	=	0.15
Hoist operator	=	2.00
Helper	=	1.25
Total cost per hr	=	\$ 5.45

While the tower is cheaper in investment and operating cost for this job, if it is required for only 10 hr, the cost of erecting the tower would make it more expensive than the crane.

**Chutes.** The use of chutes to transport concrete has been restricted considerably in recent years, primarily because of the tendency to segregate the concrete. Unless care is exercised to prevent it, segregation may occur along a chute or as the concrete flows from the lower end of the chute.

Chutes should be made of metal with round bottoms. The slope should be such that the concrete will flow at a uniform speed, with all materials flowing at the same speed, to eliminate segregation. Unless a chute can transport concrete without producing segregation, it should not be used.

**Belt Conveyors.** Under certain conditions belt conveyors are satisfactory for transporting concrete. The uniform flow and high capacity represent advantages, while the tendency to segregate the concrete at the discharge end represents a disadvantage. A suitable type ladder or down pipe should be installed at the discharge end to assure that the concrete will drop vertically. Usually it is necessary to install a belt cleaner at the discharge end to prevent a portion of the mortar from adhering to the belt.

**Transit-mixer and Agitator Trucks.** A transit-mixer or agitator truck is a truck on which there is mounted a concrete mixer. If the aggregate, including the cement, is charged into the mixer at a central batching plant, with mixing to be done en route to the job, the unit is called a transit mixer. If the unit is used to haul ready-mixed concrete, which requires agitation en route to the project only to prevent it from segregating, the unit is called an agitator.

Transit mixers are available in sizes varying from 1 to 7½ cu yd. If a unit is used as an agitator, the capacity will be considerably greater than when it is used as a transit mixer, because the concrete is premixed, and thus it occupies a volume less than that of the aggregates measured separately.

When concrete is delivered by transit-mixer or agitator trucks, the effect of mixing concrete for long periods may be questioned. Tests the



have been conducted over periods of several hours indicate that, when concrete is mixed for a long time, the slump will decrease and the compressive strength will increase for periods up to  $2\frac{1}{2}$  hr or longer [3]. The Standard Specifications for Ready-mixed Concrete (ASTM C94) requires



FIG. 19-14. Discharging concrete from a transit-mixer truck. (Chain Belt Co.)

that the concrete must be delivered and discharged from the truck mixer or agitator truck within  $1\frac{1}{2}$  hr after the introduction of the water to the cement and aggregate or the cement to the aggregate. Table 19-4 illustrates the effect of mixing time on the slump and strength of concrete.

TABLE 19-4. THE EFFECT OF MIXING TIME ON THE SLUMP AND STRENGTH OF CONCRETE\*

Time of mixing, min	Slump, in.	Compressive strength, psi		
		3-day	7-day	28-day
1	9.0	1,370	2,150	3,410
15	8.4	1,710	2,530	3,720
30	6.4	1,800	2,590	3,640
60	2.6	2,230	3,100	4,160

\* Courtesy Portland Cement Association.

**Transporting Concrete with Dump Trucks.** If air-entrained concrete is mixed at a central plant, it may be transported several miles, where the conditions are favorable, in dump trucks. The use of air-entraining cement will permit the incorporation of 3 to 6 per cent air, by volume, into the concrete. This air, which exists in very minute particles, reduces the tendency of concrete to segregate and thus permits it to be transported without agitation. Figure 19-15 illustrates a dumping hopper, mounted on a truck, which is specially designed to haul air-entrained concrete. These hoppers have rated capacities of 2, 3, and 4 cu yd of concrete.



FIG. 19-15. A special truck designed to haul air-entrained concrete. (Mazon Construction Co.)

**The Cost of Supplying Concrete by Transit-mixer Trucks versus Dump Trucks.** If concrete is transported by transit-mixer trucks, the central plant will require batching equipment only, whereas if the concrete is transported by dump trucks, it will be necessary to install a concrete mixer at the central batching plant. Other equipment costs, both at the batching plant and at the job, should be about the same for either type of transporting equipment. The quality of concrete should be the same, except that air-entrained concrete will be required if dump trucks are used, which should not affect the cost of the concrete. Only those costs which are affected by the selection need be considered.

**EXAMPLE.** Compare the cost of mixing and transporting concrete by transit-mixer trucks versus dump trucks. The job requires 48 cu yd of concrete per hour. The



concrete can be supplied by a 56S mixer and six 4-cu-yd dump trucks, each making two trips per hour, or it can be supplied by six 4½-cu-yd truck mixers, each carrying 4 cu yd per trip and making two trips per hour.

The costs and other information should be about as follows:

**Using truck mixers:**

Weight of truck and mixers	= 16,000 lb
Weight of concrete, 4 cu yd $\times$ 4,150 lb	= 16,600 lb
Total weight, loaded	= 32,600 lb
Cost of complete unit, \$13,500	
The cost per hour to mix and haul concrete will be	
Truck mixers, 6 $\times$ \$6.10	= \$36.60
Truck drivers, 6 $\times$ \$1.50	= 9.00
Total cost per hr	= \$45.60
Cost per cu yd, \$45.60 $\div$ 48	= 0.95

**Using dump trucks:**

Weight of truck and hopper	= 12,500 lb
Weight of concrete, 4 cu yd $\times$ 4,150 lb	= 16,600 lb
Total weight, loaded	= 29,100 lb
Cost of complete unit, \$7,900	
The cost per hour to mix and haul concrete will be	
Concrete mixer	= \$ 3.60
Trucks, 6 $\times$ \$3.50	= 21.00
Truck drivers, 6 $\times$ \$1.50	= 9.00
Total cost per hr	= \$33.60
Cost per cu yd, \$33.60 $\div$ 48	= 0.70

**Concrete Pumps.** Sometimes concrete is pumped through a steel pipe line. This method is particularly advantageous in lining tunnels or when it is necessary to place concrete in locations which are not easily accessible to conventional mixing, handling, or delivery equipment. The equipment includes a storage hopper, with an agitator to prevent segregation, mounted over a single-acting horizontal piston-type pump, plus steel pipe and accessories. Pumps are available in three sizes, a single-cylinder having a capacity of 15 to 20 cu yd per hr, a single-cylinder having a capacity of 25 to 33 cu yd per hr, and a double-cylinder having a capacity of 50 to 65 cu yd per hr. The pipe is 6, 7, and 8 in. in diameter. The maximum-size aggregate that can be pumped is 3 in.

The concrete can be pumped up to about 1,200 ft horizontally, depending on the size of the pump and pipe and the slump of the concrete. One foot of vertical distance is equivalent to about 8 ft of horizontal distance. A 45° bend is equivalent to 20 ft of horizontal pipe. Concrete having any reasonable slump can be pumped, but the best results are obtained in pumping concrete with a 3-in. slump.

**Placing Concrete.** If concrete is placed on earth, the earth should be moistened sufficiently to prevent it from robbing the concrete of its water. If fresh concrete is to be placed on or adjacent to concrete that has set,

the surface of the old concrete should be cleaned thoroughly, preferably with a high-pressure air and water jet and steel-wire brushes. The surface should be wet, but there should be no standing water. A small quantity of cement grout should be brushed over the entire area, then followed immediately with the application of a  $\frac{1}{2}$ -in. layer of mortar. The fresh concrete should be placed on or against the mortar.

In order to reduce the segregation resulting from movement after it is placed, concrete should be placed as nearly as practicable in its final location. It should be placed in layers whose thickness will permit uniform compaction. The time lapse between the placing of layers should

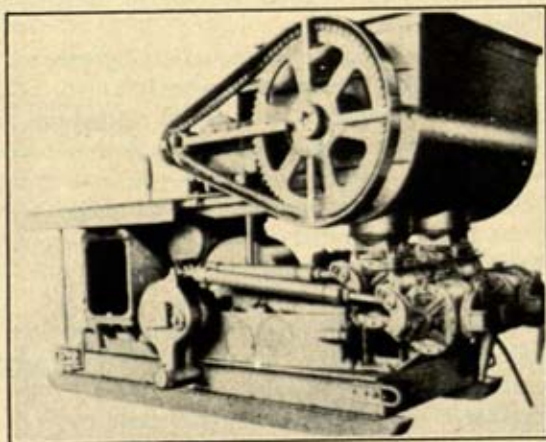


FIG. 19-16. Model 200 double Pumperete machine. (Chain Belt Co.)

be limited to assure perfect bond between the fresh and previously placed concrete.

In placing concrete in deep forms, a tremie should be used to limit the free fall to not over 3 or 4 ft, in order to prevent segregation. A tremie is a pipe made of lightweight metal, having adjustable lengths and attached to the bottom of a hopper into which the concrete is deposited. As soon as the forms are filled, sections of the pipe may be removed.

Immediately after concrete is placed, it should be compacted by hand puddling or a mechanical vibrator to eliminate voids. The vibrator should be left in one position only long enough to reduce the concrete around it to a plastic mass; then the vibrator should be moved, or segregation of the aggregate will occur. In general, the vibrator should not be permitted to penetrate concrete in the prior lift.

The primary advantage of vibrating is that it permits the use of a drier concrete, which has a higher strength because of the reduced water content. Among the advantages of vibrating concrete are the following:



1. The reduced water permits a reduction in the cement and fine aggregate because less cement paste is needed.
2. The lower water content reduces shrinkage and voids.
3. The drier concrete reduces the cost of finishing the surface.
4. Mechanical vibration can replace three to eight hand puddlers.
5. The lower water content increases the strength of the concrete.
6. The drier mix permits the removal of some forms more quickly, which may reduce the cost of forms.

**Curing Concrete.** If concrete is to attain its maximum strength and other desirable properties, it should be cured with adequate moisture and at a favorable temperature. Failure to provide these conditions may result in an inferior concrete.

The initial moisture in concrete is adequate to hydrate all the cement, provided it is not permitted to evaporate before it is used. Curing should prevent the loss of initial moisture, or it should replace the moisture that does evaporate. This may be accomplished by several methods, such as leaving the forms in place, keeping the surface wet, or covering the surface with a liquid curing compound, which forms a watertight membrane that prevents the escape of the initial water. Curing compounds may be applied by brushes or pressure sprayers. A gallon will cover 200 to 300 sq ft.

Concrete should be placed at a temperature not less than 40 or more than 80°F. A lower temperature will reduce the rate of setting, while a higher temperature will reduce the ultimate strength.

**Placing Concrete in Cold Weather.** When concrete is placed during cold weather, it usually is necessary to preheat the water, the aggregate, or both in order that the initial temperature will assure an early set and gain in strength. Preheating the water is the most effective method of providing the required temperature. For this purpose a water reservoir should be equipped with pipe coils through which steam can be passed, or steam may be discharged directly into the water, several outlets being used to give better distribution of the heat.

When the temperatures of the ingredients are known, the chart in Fig. 19-17 may be used to determine the temperature of concrete. A straight line across all three scales, passing through any two known temperatures, will permit the determination of the third temperature. If the sand is surface-dry, the solid lines of the scales giving the temperature of concrete should be used. However, if the sand contains about 3 per cent moisture, the dotted lines should be used.

Specifications frequently require that freshly placed concrete shall be maintained at a temperature of not less than 70°F for 3 days or 50°F for 5 days after it is placed. Some suitable method must be provided to maintain the required temperature when cold weather is anticipated.

**Placing Concrete in Hot Weather.** When concrete is placed during hot weather, it may be necessary to precool the concrete in order to keep the temperature within the prescribed limits. For a massive concrete structure such as a dam, this may be accomplished by installing an ice-manufacturing plant near the concrete-mixing plant, as was done at the Philpott

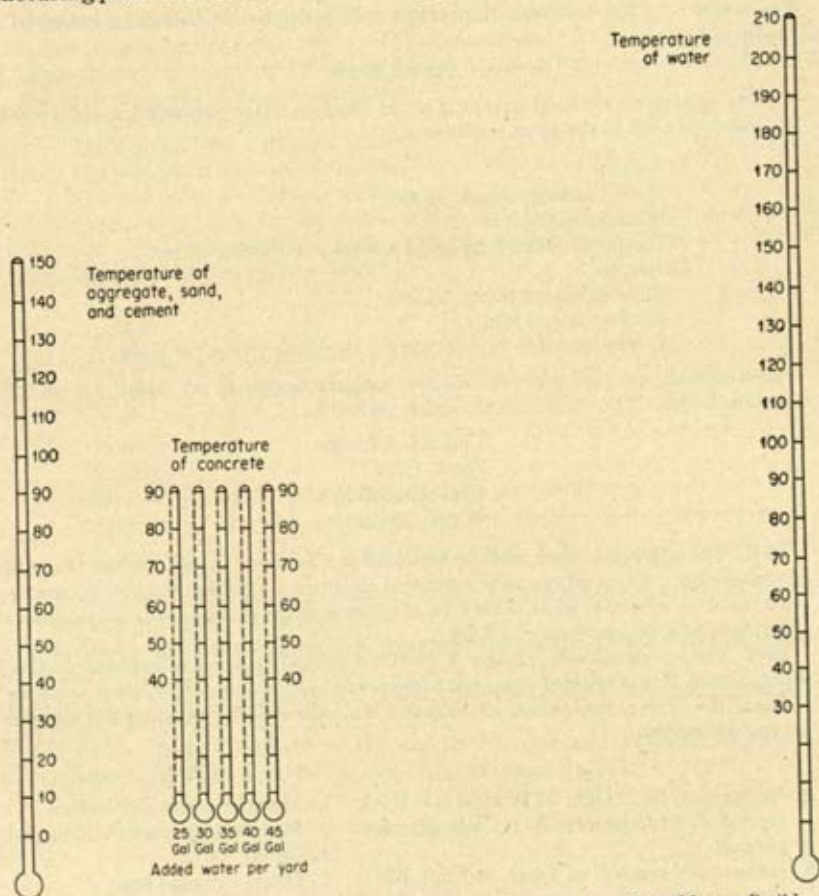


FIG. 19-17. Chart for determining the temperature of concrete. (*The Master Builders Co.*)

Dam, illustrated in Fig. 19-7. Flaked ice was added to the concrete as part of the water.

The specifications for the Pine Flat Dam required that the concrete be placed at a temperature not over 50°F [4]. In order to cool the concrete, it was necessary to use as much as 350 lb of flaked ice in a 4-cu-yd batch. With a maximum output of 136 cu yd of concrete per hour the peak ice demand was 6 tons per hr.



**Placing Concrete in Water.** When it is necessary to place concrete in water, the concrete should be dropped through a pipe long enough to reach to the bottom of the water. As the concrete rises in the forms, the bottom of the pipe should be kept continuously below the surface of the concrete in order that the fresh concrete will not come in contact with the water. This method of placing will reduce the loss of mortar to a minimum.

### PROBLEMS

**19-1.** Determine the ideal output of a 16S concrete mixer, expressed in cubic yards per hour, for each of the given conditions.

**Condition 1**

Time to charge mixer,  $\frac{1}{2}$  min

Mixing time, 1 min

Discharge entire batch into a hopper, requiring 20 sec

**Condition 2**

Time to charge mixer,  $\frac{1}{2}$  min

Mixing time, 1 min

Discharge batch into 5 buggies, requiring 10 sec per buggy

**19-2.** Determine the quantity of each material required per batch for an 11S concrete mixer. The quantities per cubic yard are

Cement, 5.2 bags

Sand, 1,324 lb

Gravel, 1,948 lb

Water, 34 gal

**19-3.** The aggregate, sand, and cement used in a concrete have an initial temperature of 50 deg. The sand contains 3 per cent moisture. If 30 gal of water is used per cubic yard of concrete, what should be the temperature of the water to produce a concrete with a temperature of 70 deg?

**19-4.** The information in column *A* describes several types of concrete structures, while column *B* lists types of equipment frequently used to handle and place concrete. Indicate the type of equipment which seems most suitable for handling the concrete for each structure.

<i>A</i>	<i>B</i>
a. Bridge pier, 32 ft high, 24 ft wide, 6 ft thick, erected in dry channel, 20 ft from dry, firm ground	1. Material tower and bucket
b. Bridge pier erected on rough terrain, 200 ft from level ground adjacent to a paved road	2. Mobile power crane and bucket
c. Slab on ground, size 90 × 120 ft	3. Chute
d. Retaining wall, 16 ft high, 460 ft long, 14 in. thick, with firm, open ground on one side	4. Pump and pipe line
e. Multistory office building, 80 ft wide, 120 ft deep, adjacent to a street, surrounded on 3 sides by other buildings	5. Power buggies
f. Lining for a tunnel 840 ft long	
g. Massive foundations for heavy machines, in a pit 20 ft deep, surrounded by dry, firm ground	

**19-5.** A reinforced-concrete retaining wall, 13 ft 6 in. high, 12 in. thick, and 600 ft long, is to be erected. An 11S mixer will be used to mix the concrete for the job. Construction joints will be installed to limit the volume of concrete per pour to 75 cu yd. It is planned to pour one section of the wall every 5 days.

By using prefabricated form panels a carpenter can erect 22 sq ft of forms per hour. The forms will be stripped the next day after the concrete is placed, by carpenter's helpers, and moved to another section of the wall immediately.

The reinforcing will weigh 662 lb per ft of wall length. A handyman can place the reinforcing at the rate of 160 lb per hr. The erection of the forms and the placing of the reinforcing will proceed simultaneously.

Determine the following:

- The length of the wall per section
- The number of carpenters required
- The number of steel setters required

Prepare a time chart for the entire project, showing a 5-day week starting with Monday. Using the indicated form, show the operations that will be performed each day the project is under construction.

Day	Operations		
	Build forms	Place reinforcing	Place concrete
Monday.....			
Tuesday.....			
Etc.....			

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## CHAPTER 20

### SAFETY ENGINEERING

**Introduction.** Did you ever see a healthy, happy construction worker performing his duties, then a few minutes later see him lying dead, the victim of an unnecessary accident? It is an experience which saddens the most calloused workers. But even more saddening is to have to witness the sorrow of the wife and children of the accident victim. It is too late to ask the question "Was this accident necessary?" The answer is "no," but it will be of little help to the victim or his family.

It has been said of a certain section of a railroad that every tie laid represents the loss of one human life, the life of a worker on the gang that built the railroad.

During the earlier years of construction it was common practice to assume that accidents would claim one life for each:

1. Two floors of a building
2. Million dollars of general construction
3. One-half mile of tunnel

The nation is no longer willing to accept such statistics as inevitable or the occurrence of these accidents as a necessary price to pay for construction. Accident prevention should be an essential part of the program of any contractor who expects to enjoy a successful career in this activity.

**Why an Accident-prevention Program?** An accident-prevention program should be inaugurated on every project in order to reduce the cost of construction measured in terms of:

1. Human lives sacrificed
2. Temporary and permanent injuries to workers
3. Loss of materials resulting from accidents
4. Loss of or damage to equipment
5. The cost of workmen's compensation insurance
6. Loss of time because of accidents

The financial justification for a safety program was illustrated by John McLeod, president of the AGC, in addressing a safety school, when he said [1]:

We have one yardstick which we use in our company to measure our accident costs. Our insurance is written under the retrospective rating plan which tells

us at the end of each policy year just what our insurance has cost us. The frequency and severity rates of industrial accidents govern the amount we pay. It so happens that between the minimum and the maximum of our premiums there is a potential savings in excess of \$200,000. We are doing everything we can to save that sum.

You know and I know that in these days \$200,000 is the profit on a lot of work. We also know that for every dollar we save on those insurance premiums, which represent direct accident losses, we have saved several dollars on indirect or hidden costs.

I believe that no one segment of your industrial operations will pay greater dividends with less investment than a good safety program.

**Workmen's Compensation Insurance.** Employers are required to carry insurance for the protection of workers who are injured on a job. Claims paid under this insurance include medical bills, hospital bills, a portion of the employee's wages during the time he is absent from work because of an injury, and death benefits.

The cost of such insurance, which must be paid by the employer, is set by the state in which a project is constructed. A designated state agency is charged with the responsibility of administering the insurance program. Construction work is divided into a number of classifications, such as steel erection, excavation, masonry, etc., based on the hazards involved, and a manual premium rate is established for each classification. In order to establish a rate for a given classification, the state agency will collect from insurance companies or from other sources detailed information showing payrolls, insurance premiums paid, and losses for each classification of hazards. Also from the insurance companies the agency will obtain reports showing the cost to the insurance companies of conducting their businesses. From this information it is possible to determine what rate should be set for any given hazard. The information obtained may cover a period of 1 or more previous years.

Assume that a premium rate is to be established for general concrete construction, based on experiences covering the 2 previous years. For the 2 years all employers spent \$21,654,792 in payrolls for concrete con-



FIG. 20-1. This hard hat saved a life.  
(Mine Safety Appliance Co.)



struction. The losses paid by the insurance companies for accidents during the same period amounted to \$416,450. The average loss paid for each \$100.00 of payroll was \$1.92. This is the loss part of the manual rate. An analysis of the expense records of the insurance companies, which are filed with the state agency, shows an expense factor of 46.4 per cent. Assume that the insurance carriers are allowed a profit margin of 2 per cent. Thus, the expense and profit factor will be 48.4 per cent, leaving a loss factor of 51.6 per cent, which is represented by \$1.92 per \$100.00 of payroll. The manual rate for premiums will be  $\$1.92 \div 0.516 = \$3.72$ . A contractor who experiences an average accident record will pay insurance premium at the rate of \$3.72 per \$100.00 of payroll for all work done under this classification.

A contractor whose accident record is lower than the average, as determined at the end of his policy year, is entitled to a credit in the form of a discount on the premium rate, while a contractor whose accident record is higher than the average will have an additional charge assessed against him for insurance premium. The extent of the credit or charge will vary with the degree to which his accident record is lower or higher than the average for the state. Thus, the actual cost of insurance premium for a given contractor is based on the manual rate, plus or minus the charge or credit, respectively, as determined from his accident experience during a policy year or during some other designated period of time. Formulas for determining the credit or charge are established by the agency for a given state.

**EXAMPLE.** The effect which a credit or a charge for insurance premium may have on the cost of constructing a project is illustrated in this example. Assume that the estimated payroll for a concrete building will be \$100,000. The manual rate for this class of work is \$3.72. The nominal cost of insurance, based on this rate, will be \$3,720. Contractor A enjoys a 24 per cent credit, while contractor B must pay a 26 per cent charge.

The ultimate cost of insurance for each contractor will be

For contractor A	
Cost from manual rate	= \$3,720
Deduct 24 % credit	= 893
Net cost	= \$2,827
For contractor B	
Cost from manual rate	= \$3,720
Add 26 % charge	= 967
Net cost	= \$4,687
Difference in cost of insurance	
	= \$1,860

Thus, other costs being the same, contractor A has an \$1,860 advantage over contractor B.

**The Indirect Costs of Accidents.** While insurance may cover certain losses to an injured worker, no insurance will cover all possible losses sus-

tained by a contractor as the result of an accident. Those losses which are not covered by insurance are defined as indirect costs and include the following:

1. Cost of lost time of injured employee
2. Cost of time lost by other employees who stop work because of an accident
3. Cost of time lost by supervisory staff to
  - a. Assist the injured employee
  - b. Investigate the cause of the accident
  - c. Arrange for someone to replace the injured employee
  - d. Prepare an accident report
4. Cost due to damaged equipment or other property
5. Cost due to spoilage of materials
6. Cost due to delayed progress on the project
7. Cost of paying wages to the injured employee during the period of injury
8. Cost of lost production resulting from the slowing down of other employees for a while following an accident

Surveys made on a nationwide basis indicate that the average direct cost of an accident on construction which results in time lost from a job is \$750. Such surveys further indicate that the average indirect cost to a contractor is at least four times the direct cost. Thus, the average indirect cost of a lost-time accident is at least \$3,000 [2].

**Definitions of Accident Terms.** Two terms which are used to indicate the extent of injuries resulting from accidents are *injury-frequency rate* and *injury-severity rate*.

The injury-frequency rate is defined as the number of disabling injuries per 1,000,000 man-hours worked. A disabling injury is an injury which causes a loss of working time beyond the day, shift, or turn during which the injury was received. The rate is expressed by the equation

$$\text{Injury-frequency rate} = \frac{\text{no. disabling injuries} \times 1,000,000}{\text{no. man-hr worked}} \quad (20-1)$$

It should be noted that the injury-frequency rate does not take into account the time lost because of an injury.

**EXAMPLE.** A contractor who employs an average of 200 men 40 hr per week for 50 weeks has 6 disabling injuries. What is his injury-frequency rate? Applying formula (20-1), we get

$$\text{Injury-frequency rate} = \frac{6 \text{ injuries} \times 1,000,000}{200 \times 40 \text{ hr} \times 50 \text{ wk}} = 15$$

The injury-severity rate is defined as the number of days of lost time because of injuries per 1,000 man-hours worked. The American Standard



Scale for lost time resulting from death or permanent disability is given in Table 20-1. If an injury should cause the death or permanent disability of a worker, the number of days of lost time used for that worker should be taken from this table. Even though a worker who loses several fingers in an accident later returns to work, the schedule given in Table 20-1 for the extent of disability should be used. The injury-severity rate, which indicates the severity of injuries, is expressed by the equation

$$\text{Injury-severity rate} = \frac{\text{no. days lost} \times 1,000}{\text{no. man-hr worked}} \quad (20-2)$$

EXAMPLE. For the contractor in the previous example, the 6 disabling injuries resulted in a total of 86 days lost from work. What is the injury-severity rate?

$$\text{Injury-severity rate} = \frac{86 \times 1,000}{200 \times 40 \text{ hr} \times 50 \text{ wk}} = 0.215$$

If 1 of the 6 injuries, involving a time loss of 16 days, resulted in the loss of an arm below the elbow, Table 20-1 indicates a time charge of 3,600 days. The total equivalent time lost for the 6 accidents will be  $86 - 16 + 3,600 = 3,670$  days. What is the injury-severity rate?

$$\text{Injury-severity rate} = \frac{3,670 \times 1,000}{400,000} = 9.18$$

TABLE 20-1. AMERICAN STANDARD SCALE OF TIME CHARGES FOR DEATH OR PERMANENT PARTIAL DISABILITY

Nature of injury	Time charge, days
Death.....	6,000
Permanent total disability.....	6,000
Loss of member or function:	
Arm, at or above elbow.....	4,500
Arm below elbow.....	3,600
Hand.....	3,000
Thumb.....	600
Any 1 finger.....	300
2 fingers on same hand.....	750
3 fingers on same hand.....	1,200
4 fingers on same hand.....	1,800
Thumb and 1 finger on same hand.....	1,200
Thumb and 2 fingers on same hand.....	1,500
Thumb and 3 fingers on same hand.....	2,000
Thumb and 4 fingers on same hand.....	2,400
Leg, at or above knee.....	4,500
Leg, below knee.....	3,000
Foot, at ankle.....	2,400
Great toe.....	300
2 great toes.....	600
1 eye, loss of sight.....	1,800
Both eyes, loss of sight.....	6,000
1 ear, loss of hearing.....	600
Both ears, loss of hearing.....	3,000

**Accident Prevalence in the Construction Industry.** Although many contractors have adopted safety programs as a part of their construction operations, the frequency and severity of accidents in construction are too high, more than twice as high as the national average for all industries.

For 1949 the reports of the National Safety Council and the Bureau of Labor Statistics of the U.S. Department of Labor for injury-frequency and injury-severity rates for the construction industry were as follows:

Source	No. units reporting	Injury frequency	Injury severity
National Safety Council.....	449	19.48	2.15
Bureau of Labor Statistics.....	4,443	39.8	3.9

During 1950 there were 2,300 deaths and 205,000 disabling injuries as the result of accidents on construction. For that year the records were as follows:

Industry	Deaths per 100,000 workers	Injuries per 100,000 workers
Construction.....	93	8,300
Average of all industries.....	27	3,360

For 1953 the National Safety Council reported 2,500 deaths and 218,000 disabling injuries in the construction industry, with a loss of about 3,500,000 man-days.

**Classification of Construction Accidents.** Extensive surveys indicate that the causes of construction accidents may be grouped as follows [3]:

1. Uncontrollable contact between men and equipment or between men and materials, such as cranes, trucks, and material storage
2. Failure of temporary structures, such as forms, scaffolds, ramps, ladders, cofferdams, sheeted cuts, etc.
3. Inherent engineering hazards, such as the use of explosives, presence of injurious gases, toxic dusts, etc.
4. Unsafe practices of individual workers or personal hazards resulting from the carelessness of workers

An analysis of the causes given above indicates that most of the accidents could be avoided through the application of an effective safety program.

**Accident Prevention in the Construction Industry.** There is no justified reason why accidents should be so prevalent in the construction industry. Numerous surveys of the causes of accidents reveal that it would be relatively easy to reduce the number of accidents to less than



50 per cent of the current rate. Contractors who maintain safety programs are able to accomplish this reduction. The saving to the industry, through reductions in the direct and indirect costs, would greatly exceed the cost of maintaining safety programs, as illustrated by the cases which follow.

In 1944 the accident record for steel workers in Michigan was not good. The manual rate for insurance was \$28.95 per \$100.00 of payroll. During that year representatives of the steel and ironworkers' unions and engineers from an insurance company decided to establish a safety school in the Detroit area. Some 1,400 men attended the school, where the engineers taught them the principles of safety [4]. As a result of the reduction in accidents that followed, the basic insurance rate was reduced to \$17.57 in 1948, and it was further reduced to \$12.12 in 1951. On the basis of a payroll of \$3,000,000 for steel erection in the state, this reduction in insurance cost produced an annual saving of \$405,000 in the state.

On a \$6,500,000 project in the Boston area an analysis of the cost of insurance revealed a difference of \$60,000 on the estimated payroll between the low bidder and the fourth bidder [2]. This difference is equivalent to a 28 per cent credit for the low bidder and a 27 per cent charge for the high bidder. Such a difference in the cost of insurance places a careless contractor at a great disadvantage when he must bid competitively against a contractor who maintains an effective safety program.

The contractor on a \$16,000,000 project for the United States Army Engineers employed an average of more than 1,200 men. The adoption of a safety program enabled him to operate during the first 5 months, involving 716,745 man-hours of work, with only 5 delaying accidents, which resulted in a loss of 61 days. This record gave an injury-frequency rate of 6.97 and an injury-severity rate of 0.084, compared with a national average of 19.34 and 2.15, respectively.

During the 4 years prior to 1950 the Denver District, Corps of Engineers, supervised construction costing more than \$25,000,000, involving approximately 5,500,000 man-hours of work. Specified safety requirements were written into the construction contracts. Supervision of the work was directed toward accident prevention. As a result, the injury-frequency rate was 5.84, and the injury-severity rate was 0.21 [5].

A highway contractor in Virginia had his insurance canceled because of a bad record. When he obtained insurance from another company, it was agreed that a safety engineer from this company would assist him in developing a safety program. Through this assistance and the effectiveness of the safety program, the frequency of accidents was reduced substantially. As the insurance premiums were paid on a retrospective basis rather than a manual rate, the reduction in accidents resulted in a



saving in insurance cost of \$7,000 the first year, \$11,000 the second year, and \$10,000 the third year after the safety program was inaugurated. The cost of the safety program was a fraction of the value of the savings.

**A Safety Program for Construction.** A safety program cannot be applied to the construction industry as a whole. It must be made an integrated part of the operations of each construction company. Such a program must receive the full support of an entire organization, beginning with top management and continuing down through the ranks to include the project superintendents, foremen, and workers. It is the responsibility of management to inaugurate the program and to contribute the continuing support necessary to keep it operating effectively.

The scope of this book does not permit a detailed coverage of all the steps that should be taken to inaugurate and maintain a safety program for the construction industry. While many of the fundamentals of safety apply to all kinds of construction, an effective program must be developed to fit the particular operations, such as steel erection, pile driving, excavation, drilling and blasting, form erection, tunneling, etc. Each operation has its own hazards, and a safety program should be developed to cope with the particular hazards. Fortunately, there are a number of sources to which one can go for detailed information to assist him in developing a program suitable for his operations. The bibliography at the end of this chapter lists some of the sources of information. Many of the insurance carriers are prepared to furnish safety engineers to assist contractors with their safety problems.

The following program, when vigorously promoted by management, has been effective in reducing the accident rate on construction [6]:

1. *Secure the Full Support of Top Management.* The employees in an organization cannot be expected to maintain an interest in a program unless management is willing to promote such an interest. After all, management will derive the greatest benefit from an effective safety program, and it should be willing to assume a leadership in promoting the program.

2. *Designate Someone in the Organization to Direct the Safety Program.* Any program as important as safety should be placed under the direction of a capable person. The safety director should be given the full support of management. He should be responsible for all safety training and should have authority to inspect all operations to assure that adequate safety practices are adopted.

3. *Publicize Your Safety Program.* Let each employee know that you have a safety program. Tell him how he can contribute to its success. Tell him how he will benefit from a reduction in accidents. When an accident does occur, let the employees know how it occurred and how it could have been prevented.



4. *Develop a Safety Program for Each Job.* Since each job has its own safety hazards, it is not possible to develop a standardized safety program that will operate effectively on all types of construction. Safety practices that are highly satisfactory for constructing a concrete building may be of little value when applied to drilling and blasting operations. Prior to starting construction on a project, the safety director should analyze the operations with the superintendent and foremen to determine what hazards will exist, and together they should develop a safety program for the project. Then it should be the responsibility of the superintendent and foremen to see that the program is put into effect.

5. *Install a Safety Program on a Competitive Basis.* The creation of competition between key personnel, with appropriate rewards for outstanding performance, has produced excellent results in many activities. It will produce results in promoting a safety program. Establish a system of awards to those supervisors who produce the best safety records. Cash awards are very good. Publicize the names of the winners.

6. *Indoctrinate New Employees.* Give all new employees a medical examination prior to employment. Let the employee know immediately that he will be required to observe safety practices. Inform him of the hazards of his work, and explain how he can reduce the danger of accidents to himself and to other workers.

7. *Make Safety Practices Effective.* Have each foreman hold a short safety session with his gang as often as conditions warrant, possibly at the beginning of work each day. Give the employees an opportunity to participate in the discussions. Sometimes the most effective method of emphasizing safety practices is by demonstration.

Replace any employees who persist in causing accidents. It is too dangerous to keep them.

8. *Promote Good Housekeeping.* A clean job is a safe job. On a construction job there should be a place for materials, tools, and equipment, and they should be in their places when not in use. Waste material should be removed from the area of operations immediately. When workers must climb over or go around piles of trash or discarded materials, tools, or equipment, the danger of accidents is increased unnecessarily. This danger should be eliminated.

9. *Maintain Adequate First-aid Facilities.* Many minor injuries can be treated satisfactorily at the job site if first-aid facilities are maintained. Some member of the staff should be prepared to provide this treatment, and each employee should know how to secure it when it is needed.

10. *Seek Assistance from the Insurance Carrier.* Many insurance carriers will assist contractors in developing and maintaining their safety programs. This service is furnished through safety engineers, whose

training and experience qualify them to suggest safety measures that will reduce accidents.

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## APPENDIX A

### COST OF OWNING AND OPERATING CONSTRUCTION EQUIPMENT

The costs given in this table are intended to indicate the approximate costs per hour for ownership, fuel, and other expenses and the total ownership and operating costs per working hour for construction equipment.

The cost per hour for ownership includes depreciation, major overhauling and repairs, interest, taxes, insurance, and storage.

The annual cost of depreciation is based on the estimated economical life of the equipment. The straight-line method of determining the annual cost of depreciation is used. Thus the annual cost of depreciation for equipment having an estimated life of 4 years is 25 per cent of the original cost of the equipment.

The annual cost of major repairs is based on experience obtained from the operation of the equipment and will vary with the equipment and the conditions under which it is used. This cost includes major repairs only. The cost of minor repairs, which are usually made in the field, is included under fuel and other expenses.

The annual cost of interest, taxes, insurance, and storage is assumed to be constant and is based on the following: interest 6 per cent, taxes 2 per cent, insurance and storage 2 per cent of the cost of the equipment. The total is 10 per cent per year.

The hours used per year are assumed for average conditions and will vary between jobs, with owners, and with geographical locations. Estimators should modify the number of hours used per year, if necessary, to represent more nearly actual conditions under which the equipment will be used. If equipment is used more than the number of hours given in the table, the ownership cost per hour should be reduced, while if it is used fewer hours than given in the table, the ownership cost per hour should be increased. The ownership cost per hour is obtained by multiplying the cost to the owner by the total per cent of cost, as given in column (4), and dividing the product by the hours used per year.

The cost per working hour for fuel and other expense includes fuel, lubricating oil, greasing, filters, air cleaners, minor repairs, etc., for equipment powered by internal-combustion engines. For equipment powered by electric motors the cost in column (8) includes the cost of electrical energy and minor repairs.

The costs given in column (8) for equipment powered by diesel engines are obtained as follows:

Cost of diesel fuel, \$0.15 per gal, tax included

Fuel consumed per brake hp-hr at full load, 0.04 gal

Assume that equipment operates at an average of 66⅔% of the rated hp

Fuel consumed per rated hp-hr,  $\frac{2}{3} \times 0.04 = 0.027$  gal

Fuel cost per rated hp-hr,  $\$0.15 \times 0.027$

= \$0.004

Cost per rated hp-hr for oil, grease, filters, minor repairs, etc.

= 0.003

Total cost per rated hp-hr

= \$0.007

The costs given in column (8) for equipment powered by gasoline engines are obtained as follows:

Cost of gasoline, \$0.25 per gal, tax included	
Fuel consumed per brake hp-hr at full load, 0.06 gal	
Assume that equipment operates at an average of 66 $\frac{2}{3}$ per cent of the rated hp	
Fuel consumed per rated hp-hr, $\frac{2}{3} \times 0.06 = 0.04$ gal	
Fuel cost per rated hp-hr, $\$0.25 \times 0.04$	= \$0.010
Cost per rated hp-hr for oil, greasing, filters, minor repairs, etc.	= 0.004
Total cost per rated hp-hr	= \$0.014

The costs given in column (8) for equipment powered by electric motors are obtained as follows:

Cost of electrical energy, \$0.02 per kwhr	
Energy consumed per hp-hr, based on 75% efficiency, 1 kwhr	
Cost of electrical energy per hp-hr	= \$0.02
Additional costs per hp-hr	= 0.002
Total cost per hp-hr	= \$0.022

The costs given in column (8) for equipment which has no direct power unit, such as self-loading scrapers, include greasing and minor repairs.

The total operating cost per working hour, as given in column (9), is the sum of the costs given in columns (7) and (8). *The total operating cost does not include the wages of the operator. It does not include the cost of transporting the equipment to the job or setting it up for operation.* These costs must be included in the total estimate for the job, as separate items.

The following example illustrates the method used in estimating the hourly ownership and operating cost of equipment:

Equipment, crawler tractor, 130-hp diesel	
Purchase price, f.o.b. factory	\$14,500
Freight, factory to owner	500
Total cost to owner	\$15,000
Annual costs:	
Depreciation, 20% of \$15,000	\$3,000
Repairs, 15% of \$15,000	2,250
Interest, etc., 10% of \$15,000	1,500
Annual ownership cost	\$6,750
Cost per hr @ 2,000 hr per yr	= \$3.38
Fuel and other expense, 130 hp @ \$0.007 per hp	= 0.91
Total estimated hourly cost	= \$4.29

The costs given for construction equipment are based on 1951 prices. The estimator must modify the information given in the table to allow for future changes in prices of equipment.



## 512 CONSTRUCTION PLANNING, EQUIPMENT, AND METHODS

## ESTIMATED HOURLY OWNERSHIP AND OPERATING COST OF CONSTRUCTION EQUIPMENT

Equipment	Average annual expense, per cent of cost				Cost to owner	Hr used per year	Cost per working hr		
	De- pre- cia- tion	Major re- pairs	Inter- est, taxes, insur- ance	Total per cent of cost			Own- ership	Fuel and other ex- pense	Total oper- ating
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Air compressors, portable, free air at 100 lb:									
Gasoline engine									
60 cfm.....	25	15	10	50	\$ 1,540	1,200	\$ 0.65	\$0.28	\$ 0.93
75 cfm.....	25	15	10	50	1,920	1,200	0.80	0.35	1.15
105 cfm.....	25	15	10	50	2,875	1,200	1.20	0.50	1.70
125 cfm.....	25	15	10	50	3,065	1,200	1.28	0.56	1.84
185 cfm.....	25	15	10	50	4,225	1,200	1.76	0.80	2.56
250 cfm.....	20	15	10	45	5,340	1,200	2.00	1.05	3.05
315 cfm.....	20	15	10	45	7,190	1,200	2.69	1.20	3.89
Diesel engine									
105 cfm.....	25	15	10	50	3,890	1,200	1.62	0.25	1.87
125 cfm.....	25	15	10	50	4,140	1,200	1.73	0.30	2.03
185 cfm.....	25	15	10	50	5,300	1,200	2.21	0.42	2.63
250 cfm.....	20	15	10	45	6,650	1,200	2.50	0.50	3.00
365 cfm.....	20	15	10	45	8,390	1,200	3.15	0.72	3.87
600 cfm.....	20	15	10	45	11,800	1,200	4.40	1.10	5.50
Air compressors, stationary, no power unit or air tank, free air at 100 lb:									
60 cfm.....	25	15	10	50	1,075	1,200	0.45	0.05	0.50
105 cfm.....	25	15	10	50	1,470	1,200	0.61	0.06	0.67
160 cfm.....	25	15	10	50	1,745	1,200	0.73	0.07	0.80
210 cfm.....	20	15	10	45	2,485	1,200	0.93	0.09	1.02
315 cfm.....	20	15	10	45	3,035	1,200	1.14	0.10	1.24
Air tools, no hose or steel:									
Drifters									
Light.....	25	10	10	45	625	1,200	0.24	.....	0.24
Medium.....	25	10	10	45	695	1,200	0.26	.....	0.26
Heavy.....	25	10	10	45	845	1,200	0.32	.....	0.32
Jackhammers									
Light.....	33	10	10	53	295	1,200	0.13	.....	0.13
Medium.....	33	10	10	53	315	1,200	0.14	.....	0.14
Heavy.....	33	10	10	53	335	1,200	0.15	.....	0.15
Paving breakers									
60 lb.....	33	10	10	53	295	1,200	0.13	.....	0.13
80 lb.....	33	10	10	53	330	1,200	0.15	.....	0.15
Wagon drills									
Light.....	15	10	10	35	1,585	1,400	0.40	.....	0.40
Medium.....	15	10	10	35	1,865	1,400	0.47	.....	0.47
Heavy.....	15	10	10	35	2,630	1,400	0.66	.....	0.66
Air hose, 50 ft with couplings:									
½ in., 2 braid.....	50	10	10	70	25	1,200	0.02	.....	0.02
¾ in., 2 braid.....	50	10	10	70	29	1,200	0.02	.....	0.02
1 in., 2 braid.....	50	10	10	70	38	1,200	0.02	.....	0.02
1¼ in., 2 braid.....	50	10	10	70	50	1,200	0.03	.....	0.03
1½ in.....	50	10	10	70	75	1,200	0.04	.....	0.04

ESTIMATED HOURLY OWNERSHIP AND OPERATING COST OF CONSTRUCTION EQUIPMENT  
(Continued)

Equipment	Average annual expense, per cent of cost				Cost to owner	Hr used per year	Cost per working hr		
	De- pre- cia- tion	Major re- pairs	Inter- est, taxes, insur- ance	Total per cent of cost			Owner- ship	Fuel and other ex- pense	Total oper- ating
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
<b>Batching equipment, for con- crete aggregates:</b>									
Batcher only									
Weigher, 2 mat., 1 cu yd....	25	17	10	52	\$ 1,600	1,200	\$ 0.70	.....	\$0.70
Weigher, 2 mat., 2 cu yd....	25	17	10	52	1,960	1,200	0.80	.....	0.80
Weigher, 3 mat., 1 cu yd....	25	17	10	52	1,800	1,200	0.75	.....	0.75
Weigher, 3 mat., 2 cu yd....	25	17	10	52	2,190	1,200	0.95	.....	0.95
Bins, steel, portable									
1 compartment, 15 tons....	20	15	10	45	860	1,200	0.32	.....	0.32
1 compartment, 27 tons....	20	15	10	45	1,050	1,200	0.40	.....	0.40
2 compartments, 35 tons....	20	15	10	45	1,300	1,200	0.50	.....	0.50
2 compartments, 50 tons....	20	15	10	45	1,800	1,200	0.68	.....	0.68
3 compartments, 60 tons....	20	15	10	45	2,090	1,200	0.80	.....	0.80
3 compartments, 100 tons....	20	15	10	45	2,660	1,200	1.00	.....	1.00
<b>Bituminous equipment:</b>									
Distributor, gasoline engine									
600 gal, no truck.....	20	17	10	47	3,800	1,600	1.10	\$1.25	2.35
1,000 gal, no truck.....	20	17	10	47	4,900	1,600	1.45	1.25	2.70
1,500 gal, no truck.....	20	17	10	47	5,200	1,600	1.55	1.25	2.80
600 gal, with truck.....	20	17	10	47	6,100	1,600	1.80	2.50	4.30
1,000 gal, with truck.....	20	17	10	47	8,100	1,600	2.40	2.50	4.90
1,500 gal, with truck.....	20	17	10	47	8,600	1,600	2.52	2.80	5.32
Paver, complete									
Crawler, 100 tons per hr, gas.	25	15	10	50	13,500	1,600	4.25	0.60	4.85
Crawler, 100 tons per hr, diesel.....	20	15	10	45	15,400	1,600	4.40	0.35	4.75
Rotary broom									
6 ft, without power.....	20	15	10	45	1,230	1,600	0.35	.....	0.35
6 ft, with power.....	20	15	10	45	1,800	1,600	0.50	0.25	0.75
9 ft, with power.....	20	15	10	45	2,700	1,600	0.75	0.30	1.05
Spreader, gasoline engine									
5 to 12 ft, crawler.....	25	15	10	50	6,000	1,600	1.90	0.53	2.43
5 to 12 ft, tires.....	25	17	10	52	5,600	1,600	1.75	0.53	2.28
7 to 13 ft, crawler.....	25	15	10	50	6,400	1,600	2.00	0.56	2.56
7 to 13 ft, tires.....	25	17	10	52	5,900	1,600	1.91	0.56	2.47
Spreader, box									
8 to 11 ft, mounted.....	20	15	10	45	1,180	1,600	0.35	.....	0.35
<b>Carts, concrete:</b>									
Steel wheels									
6 cu ft.....	33	20	10	63	56	1,400	0.03	.....	0.03
9 cu ft.....	33	20	10	63	74	1,400	0.03	.....	0.03
11 cu ft.....	33	20	10	63	83	1,400	0.04	.....	0.04
Pneumatic tires									
6 cu ft.....	33	25	10	68	65	1,400	0.03	.....	0.03
9 cu ft.....	33	25	10	68	85	1,400	0.04	.....	0.04
11 cu ft.....	33	25	10	68	95	1,400	0.05	.....	0.05



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ESTIMATED HOURLY OWNERSHIP AND OPERATING COST OF CONSTRUCTION EQUIPMENT.  
(Continued)

Equipment	Average annual expense, per cent of cost				Cost to owner	Hr used per year	Cost per working hr		
	De- pre- cia- tion	Major re- pairs	Inter- est, taxes, insur- ance	Total per cent of cost			Own- ership	Fuel and other ex- pense	Total oper- ating
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Power-driven, with tires									
9 cu ft.....	33	25	10	68	\$ 565	1,400	\$ 0.28	\$0.09	\$0.37
11 cu ft.....	33	25	10	68	650	1,400	0.32	0.11	0.43
13 cu ft.....	33	25	10	68	875	1,400	0.43	0.13	0.56
Cement gun machines, with no- zle and 50-ft hose:									
1½ in., skids, no power.....	25	15	10	50	1,885	1,600	0.59	.....	0.59
1½ in., tires, gas engine.....	25	15	10	50	2,100	1,600	0.66	0.21	0.87
1½ in., tires, air-compressor, gas engine.....	25	15	10	50	4,725	1,600	1.47	0.70	2.17
1½ in., tires, gas engine.....	25	15	10	50	3,520	1,600	1.10	0.42	1.52
1½ in., tires, air-compressor, gas engine.....	25	15	10	50	8,925	1,600	2.80	0.90	3.70
1½ in., tires, air-compressor diesel engine.....	25	15	10	50	10,200	1,600	3.20	0.40	3.60
Column clamps, adjustable:									
36 in.....	20	10	10	40	72,000		Per use		0.15
48 in.....	20	10	10	40	82,000		Per use		0.15
60 in.....	20	10	10	40	92,000		Per use		0.15
Concrete buckets:									
Bottom dump									
¾ cu yd.....	20	15	10	45	360	1,200	0.13	.....	0.13
1 cu yd.....	20	15	10	45	400	1,200	0.15	.....	0.15
1½ cu yd.....	20	15	10	45	580	1,200	0.22	.....	0.22
2 cu yd.....	20	15	10	45	600	1,200	0.23	.....	0.23
3 cu yd.....	20	15	10	45	800	1,200	0.30	.....	0.30
4 cu yd.....	20	15	10	45	1,150	1,200	0.43	.....	0.43
Tower hoist									
8 cu ft.....	20	15	10	45	170	1,200	0.07	.....	0.07
12 cu ft.....	20	15	10	45	215	1,200	0.08	.....	0.08
16 cu ft.....	20	15	10	45	255	1,200	0.10	.....	0.10
18 cu ft.....	20	15	10	45	275	1,200	0.11	.....	0.11
27 cu ft.....	20	15	10	45	305	1,200	0.12	.....	0.12
36 cu ft.....	20	15	10	45	360	1,200	0.14	.....	0.14
Concrete hoppers, floor:									
Single gate									
14 cu ft.....	20	20	10	50	180	1,200	0.08	.....	0.08
27 cu ft.....	20	20	10	50	200	1,200	0.08	.....	0.08
40 cu ft.....	20	20	10	50	225	1,200	0.09	.....	0.09
54 cu ft.....	20	20	10	50	275	1,200	0.12	.....	0.12
68 cu ft.....	20	20	10	50	320	1,200	0.13	.....	0.13
Double gate.									
54 cu ft.....	20	20	10	50	370	1,200	0.16	.....	0.16
68 cu ft.....	20	20	10	50	425	1,200	0.18	.....	0.18
84 cu ft.....	20	20	10	50	500	1,200	0.21	.....	0.21
108 cu ft.....	20	20	10	50	590	1,200	0.25	.....	0.25
135 cu ft.....	20	20	10	50	650	1,200	0.27	.....	0.27

**ESTIMATED HOURLY OWNERSHIP AND OPERATING COST OF CONSTRUCTION EQUIPMENT.**  
(Continued)

Equipment	Average annual expense, per cent of cost				Cost to owner	Hr used per year	Cost per working hr		
	De- pre- cia- tion	Major re- pairs	Inter- est, taxes, insur- ance	Total per cent of cost			Owner- ship	Fuel and other ex- pense	Total oper- ating
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
<b>Concrete mixers, construction:</b>									
Gasoline engine									
3½8 tilt.....	50	12	10	72	\$ 565	1,600	\$ 0.25	\$0.06	\$0.31
3½8 nontilt.....	50	12	10	72	825	1,600	0.37	0.06	0.43
6S.....	40	12	10	62	1,775	1,600	0.70	0.12	0.82
11S.....	40	12	10	62	2,350	1,600	0.92	0.23	1.15
16S.....	33	12	10	55	3,135	1,600	1.08	0.28	1.36
28S, on skids.....	25	12	10	47	5,850	1,600	1.75	0.48	2.23
<b>Concrete mixers, paving:</b>									
Diesel engine									
27E single.....	25	12	10	47	21,000	1,400	7.05	0.48	7.53
27E double.....	25	12	10	47	27,500	1,400	9.20	0.55	9.75
34E single.....	20	12	10	42	25,000	1,400	7.50	0.55	8.05
34E double.....	20	12	10	42	31,500	1,400	9.50	0.70	10.20
<b>Concrete mixers, truck type:</b>									
Gasoline engine, no truck									
2 cu yd.....	25	12	10	47	4,700	2,000	1.10	0.38	1.48
3 cu yd.....	25	12	10	47	5,400	2,000	1.25	0.82	2.07
4½ cu yd.....	25	12	10	47	6,400	2,000	1.50	0.92	2.42
5½ cu yd.....	25	12	10	47	7,450	2,000	1.75	1.05	2.80
Including truck, gas engine									
1 cu yd.....	33	15	10	58	5,400	2,000	1.55	1.45	3.00
2 cu yd.....	25	15	10	50	8,500	2,000	2.20	2.00	4.20
3 cu yd.....	25	15	10	50	9,500	2,000	2.40	2.30	4.70
4 cu yd.....	25	15	10	50	12,000	2,000	3.00	2.60	5.60
<b>Concrete pumps, complete gaso- line engine:</b>									
Single, 15 to 20 cu yd per hr, 800-ft pipe.....	25	15	10	50	14,000	1,400	5.00	0.42	5.42
Single, 25 to 33 cu yd, per hr, 1,000-ft pipe.....	20	15	10	45	22,000	1,400	6.75	0.87	7.62
Double, 50 to 65 cu yd per hr, 1,000-ft pipe.....	20	5	10	45	30,000	1,400	9.65	1.08	10.73
<b>Concrete screeds and finishers, gasoline engine, no screeds:</b>									
10 to 15 ft, adjustable.....	25	15	10	50	5,500	1,600	1.72	0.32	2.04
20 to 25 ft, adjustable.....	25	15	10	50	5,750	1,600	1.76	0.32	2.08
<b>Concrete screeds, vibrating, ad- justable:</b>									
6 ft, gasoline engine.....	25	15	10	50	430	1,600	0.13	0.03	0.16
8 ft, gasoline engine.....	25	15	10	50	440	1,600	0.14	0.03	0.17
10 ft, gasoline engine.....	25	15	10	50	450	1,600	0.14	0.03	0.17
12 ft, gasoline engine.....	25	15	10	50	460	1,600	0.14	0.03	0.17
16 ft, electric motor.....	25	12	10	47	575	1,600	0.16	0.05	0.21



## 516 CONSTRUCTION PLANNING, EQUIPMENT, AND METHODS

ESTIMATED HOURLY OWNERSHIP AND OPERATING COST OF CONSTRUCTION EQUIPMENT  
(Continued)

Equipment	Average annual expense, per cent of cost				Cost to owner	Hr used per year	Cost per working hr		
	De- pre- cia- tion	Major re- pairs	Inter- est, taxes, insur- ance	Total per cent of cost			Own- ership	Fuel and other ex- pense	Total oper- ating
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
<b>Concrete vibrators, internal:</b>									
Electric motor									
2 hp, 7-ft shaft.....	25	10	10	45	\$ 275	1,600	\$ 0.08	\$0.05	\$0.13
2½ hp, 3-ft shaft.....	25	10	10	45	300	1,600	0.08	0.06	0.14
2½ hp, 7-ft shaft.....	25	10	10	45	315	1,600	0.09	0.06	0.15
2½ hp, 14-ft shaft.....	25	10	10	45	340	1,600	0.10	0.06	0.16
2½ hp, 21-ft shaft.....	25	10	10	45	390	1,600	0.11	0.06	0.17
2½ hp, 28-ft shaft.....	25	10	10	45	425	1,600	0.12	0.06	0.18
Gasoline engine									
2 hp, 7-ft shaft.....	33	12	10	55	175	1,600	0.06	0.03	0.09
2 hp, 14-ft shaft.....	33	12	10	55	200	1,600	0.07	0.03	0.10
4 hp, 7-ft shaft.....	33	12	10	55	320	1,600	0.11	0.06	0.17
4 hp, 14-ft shaft.....	33	12	10	55	360	1,600	0.12	0.06	0.18
4 hp, 21-ft shaft.....	33	12	10	55	405	1,600	0.14	0.06	0.20
<b>Cranes, complete:</b>									
Crawler, gasoline engine									
4 tons, 10-ft radius.....	25	10	10	45	10,500	1,600	2.95	0.56	3.51
6 tons, 10-ft radius.....	25	10	10	45	12,200	1,600	3.45	0.63	4.08
8 tons, 10-ft radius.....	25	10	10	45	15,600	1,600	4.40	0.70	5.10
16 tons, 10-ft radius.....	20	10	10	40	20,100	1,600	5.00	1.25	6.25
8 tons, 12-ft radius.....	20	10	10	40	16,800	1,600	4.20	0.85	5.05
12 tons, 12-ft radius.....	20	10	10	40	20,100	1,600	5.00	1.10	6.10
20 tons, 12-ft radius.....	20	10	10	40	24,300	1,600	6.15	1.25	7.40
30 tons, 12-ft radius.....	17	10	10	37	27,000	1,600	7.15	1.75	8.90
10 tons, 15-ft radius.....	17	10	10	37	20,100	1,600	5.30	1.25	6.55
Crawler, diesel engine									
4 tons, 10-ft radius.....	25	10	10	45	12,200	1,600	3.40	0.25	3.65
6 tons, 10-ft radius.....	20	10	10	40	13,600	1,600	3.40	0.27	3.67
8 tons, 10-ft radius.....	20	10	10	40	17,000	1,600	4.25	0.30	4.55
8 tons, 12-ft radius.....	20	10	10	40	17,800	1,600	4.45	0.36	4.81
12 tons, 12-ft radius.....	20	10	10	40	22,300	1,600	5.60	0.48	6.08
20 tons, 12-ft radius.....	17	10	10	37	27,800	1,600	6.45	0.55	7.00
35 tons, 12-ft radius.....	17	10	10	37	40,100	1,400	10.60	0.76	11.36
30 tons, 15-ft radius.....	17	10	10	37	40,100	1,400	10.60	0.76	11.36
40 tons, 15-ft radius.....	17	10	10	37	54,300	1,400	14.40	0.80	15.20
25 tons, 20-ft radius.....	17	10	10	37	54,300	1,400	14.40	0.80	15.20
15 tons, 30-ft radius.....	17	10	10	37	54,300	1,400	14.40	0.80	15.20
10 tons, 40-ft radius.....	17	10	10	37	54,300	1,400	14.40	0.80	15.20
Truck-mounted, gas engine									
4 tons, 10-ft radius.....	25	15	10	50	12,000	1,600	3.75	0.85	4.60
6 tons, 10-ft radius.....	25	15	10	50	13,300	1,600	4.15	0.95	5.10
8 tons, 10-ft radius.....	20	15	10	45	16,500	1,600	4.65	1.05	5.70
12 tons, 10-ft radius.....	20	15	10	45	22,800	1,400	7.40	1.50	8.90
16 tons, 10-ft radius.....	20	12	10	42	33,100	1,400	9.90	1.90	11.80

**ESTIMATED HOURLY OWNERSHIP AND OPERATING COST OF CONSTRUCTION EQUIPMENT**  
(Continued)

Equipment	Average annual expense, per cent of cost				Cost to owner	Hr used per year	Cost per working hr		
	De- pre- cia- tion	Major re- pairs	Inter- est, taxes, insur- ance	Total per cent of cost			Owner- ship	Fuel and other ex- pense	Total oper- ating
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
<b>Truck-mounted, diesel</b>									
8 tons, 10-ft radius.....	20	15	10	45	\$18,600	1,600	\$ 5.25	\$0.50	\$5.75
16 tons, 10-ft radius.....	17	10	10	37	34,600	1,400	9.15	0.70	9.85
12 tons, 12-ft radius.....	17	10	10	37	34,600	1,400	9.15	0.70	9.85
35 tons, 12-ft radius.....	17	10	10	37	55,400	1,400	14.60	1.15	15.75
20 tons, 20-ft radius.....	17	10	10	37	55,400	1,400	14.60	1.15	15.75
<b>Derricks, steel, including load blocks:</b>									
Guy, 55-ft boom, 5 tons.....	8	12	10	30	4,400	1,400	0.95	.....	0.95
Guy, 55-ft boom, 15 tons.....	8	12	10	30	5,100	1,400	1.10	.....	1.10
Guy, 70-ft boom, 15 tons.....	8	12	10	30	6,000	1,400	1.30	.....	1.30
Guy, 70-ft boom, 25 tons.....	8	12	10	30	8,700	1,400	1.85	.....	1.85
Guy, 100-ft boom, 10 tons.....	8	12	10	30	6,900	1,400	1.50	.....	1.50
Guy, 100-ft boom, 25 tons.....	8	12	10	30	9,700	1,400	2.10	.....	2.10
Stiff leg, 50-ft boom, 5 tons.....	8	15	10	33	6,600	1,400	1.55	.....	1.55
Stiff leg, 50-ft boom, 15 tons.....	8	15	10	33	8,100	1,400	1.90	.....	1.90
Stiff leg, 70-ft boom, 5 tons.....	8	15	10	33	7,900	1,400	1.85	.....	1.85
Stiff leg, 70-ft boom, 15 tons.....	8	15	10	33	9,250	1,400	2.20	.....	2.20
<b>Draglines, including rope and bucket:</b>									
<b>Crawler, gasoline</b>									
1½ cu yd.....	25	15	10	50	13,200	2,000	3.30	1.05	4.35
¾ cu yd.....	25	15	10	50	18,900	2,000	4.25	1.20	5.45
1 cu yd.....	20	15	10	45	23,500	2,000	5.30	1.35	6.65
1½ cu yd.....	17	15	10	42	31,500	1,600	8.25	1.75	10.00
2 cu yd.....	17	15	10	42	44,000	1,600	11.50	2.50	14.00
<b>Crawler, diesel</b>									
1½ cu yd.....	25	15	10	50	15,000	2,000	3.75	0.45	4.20
¾ cu yd.....	20	15	10	45	20,400	2,000	4.60	0.55	5.15
1 cu yd.....	20	15	10	45	25,400	2,000	5.70	0.60	6.30
1½ cu yd.....	17	15	10	42	36,500	1,600	9.60	0.75	10.35
2 cu yd.....	17	15	10	42	52,000	1,600	13.60	1.10	14.70
<b>Forms, steel:</b>									
Curb, 6 in. high, 1,000 lin ft....	25	15	10	50	1,100	1,400	Per use		6.40
Road, per 1,000 lin ft.....									
Depth 7 in., base 7 in.....	25	15	10	50	1,850	1,400	Per use		10.50
Depth 8 in., base 8 in.....	25	15	10	50	1,960	1,400	Per use		11.20
Depth 8 in., base 9 in.....	25	15	10	50	2,100	1,400	Per use		12.00
<b>Graders, hydraulic control:</b>									
8-ft blade, gasoline.....	25	15	10	50	4,150	2,000	1.05	0.42	1.47
10-ft blade, gasoline.....	25	15	10	50	7,500	2,000	1.90	0.63	2.53
12-ft blade, diesel.....	20	15	10	45	11,500	2,000	2.60	0.45	3.05
13-ft blade, diesel.....	20	15	10	45	12,800	2,000	2.90	0.60	3.50



## 518 CONSTRUCTION PLANNING, EQUIPMENT, AND METHODS

ESTIMATED HOURLY OWNERSHIP AND OPERATING COST OF CONSTRUCTION EQUIPMENT  
(Continued)

Equipment	Average annual expense, per cent of cost				Cost to owner	Hr used per year	Cost per working hr		
	De- pre- cia- tion	Major re- pairs	Inter- est, taxes, insur- ance	Total per cent of cost			Owner- ship	Fuel and other ex- pense	Total oper- ating
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
<b>Hoisting units, including power unit:</b>									
<b>Gasoline engine</b>									
Single drum, 4 hp.....	25	15	10	50	\$ 600	1,600	\$ 0.20	\$0.06	\$0.26
Single drum, 10 hp.....	20	15	10	45	1,200	1,600	0.35	0.14	0.49
Single drum, 15 hp.....	20	15	10	45	1,520	1,600	0.45	0.21	0.66
Double drum, 20 hp.....	17	10	10	37	2,200	1,600	0.52	0.28	0.80
Double drum, 40 hp.....	17	10	10	37	3,100	1,600	0.72	0.56	1.28
Double drum, 70 hp.....	17	10	10	37	4,300	1,600	1.00	0.98	1.98
Three drums, 40 hp.....	17	10	10	37	4,000	1,600	0.95	0.56	1.51
Three drums, 60 hp.....	17	10	10	37	5,100	1,600	1.20	0.84	2.04
<b>Electric motor</b>									
Single drum, 3 hp.....	20	15	10	45	650	1,600	0.20	0.07	0.27
Single drum, 10 hp.....	17	10	10	37	1,200	1,600	0.30	0.22	0.52
Double drum, 10 hp.....	17	10	10	37	1,570	1,600	0.35	0.22	0.57
Double drum, 20 hp.....	13	10	10	33	2,950	1,600	0.60	0.44	1.04
Double drum, 40 hp.....	13	10	10	33	4,400	1,600	0.90	0.88	1.78
Three drums, 40 hp.....	13	10	10	33	5,300	1,600	1.10	0.88	1.98
Three drums, 60 hp.....	13	10	10	33	6,650	1,600	1.40	1.32	2.72
<b>Loaders:</b>									
<b>Bucket, gasoline engine</b>									
60 cu yd per hr.....	20	15	10	45	4,650	1,400	1.50	0.42	1.92
90 cu yd per hr.....	20	15	10	45	5,500	1,400	1.75	0.65	2.40
120 cu yd per hr.....	20	15	10	45	6,500	1,400	2.10	0.85	2.95
150 cu yd per hr.....	20	15	10	45	7,400	1,400	2.40	1.25	3.65
<b>Elevating grader, earth</b>									
9-ft 6-in. blade, 190-hp diesel	20	15	10	45	33,000	2,000	7.50	1.20	8.70
9-ft 6-in. blade, 245-hp diesel	20	15	10	45	36,000	2,000	8.10	1.45	9.55
<b>Tractor, front end</b>									
½ cu yd.....	33	20	10	63	2,150	1,600	0.80	.....	0.80
¾ cu yd.....	33	20	10	63	2,700	1,600	1.10	.....	1.10
1 cu yd.....	25	20	10	55	5,400	1,600	1.85	.....	1.85
2 cu yd.....	25	20	10	55	8,000	1,600	2.75	.....	2.75
<b>Pile hammers, standard:</b>									
<b>Single-acting steam</b>									
3,600 ft-lb.....	20	12	10	42	3,250	1,400	1.00	.....	1.00
7,500 ft-lb.....	20	12	10	42	4,500	1,400	1.35	.....	1.35
15,000 ft-lb.....	17	12	10	39	5,600	1,400	1.55	.....	1.55
25,000 ft-lb.....	17	12	10	39	8,500	1,400	2.40	.....	2.40
30,000 ft-lb.....	17	12	10	39	8,800	1,200	2.85	.....	2.85
<b>Double-acting steam</b>									
3,600 ft-lb.....	20	12	10	42	3,450	1,400	1.05	.....	1.05
7,500 ft-lb.....	20	12	10	42	4,900	1,400	1.50	.....	1.50
15,000 ft-lb.....	17	12	10	39	6,200	1,400	1.75	.....	1.75
25,000 ft-lb.....	17	12	10	39	9,400	1,400	2.60	.....	2.60
36,000 ft-lb.....	17	12	10	39	19,500	1,200	6.40	.....	6.40

ESTIMATED HOURLY OWNERSHIP AND OPERATING COST OF CONSTRUCTION EQUIPMENT  
(Continued)

Equipment	Average annual expense, per cent of cost				Cost to owner	Hr used per year	Cost per working hr		
	De- pre- cia- tion	Major re- pairs	Inter- est, taxes, insur- ance	Total per cent of cost			Owner- ship	Fuel and other ex- pense	Total oper- ating
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Power plants, portable, gasoline engine:									
1½ kw, mounted.....	15	15	10	40	\$ 450	1,600	\$ 0.15	\$0.06	\$0.21
2½ kw, mounted.....	15	15	10	40	600	1,600	0.20	0.09	0.29
5 kw, mounted.....	15	10	10	35	1,200	1,600	0.30	0.18	0.48
5 kw, skids.....	15	10	10	35	1,100	1,600	0.25	0.18	0.43
Pumps, centrifugal:									
Electric, portable									
1½ in., 4,000 gal per hr	17	15	10	42	242	1,200	0.10	0.03	0.13
2 in., 7,000 gal per hr low head.....	17	15	10	42	300	1,200	0.10	0.06	0.16
2 in., 7,000 gal per hr, high head.....	17	15	10	42	380	1,200	0.15	0.09	0.24
3 in., 12,000 gal per hr, low head.....	17	15	10	42	350	1,200	0.15	0.11	0.26
3 in., 20,000 gal per hr, high head.....	17	15	10	42	500	1,200	0.20	0.20	0.40
4 in., 30,000 gal per hr, low head.....	17	15	10	42	700	1,200	0.25	0.48	0.73
4 in., 40,000 gal per hr, high head.....	17	15	10	42	900	1,200	0.30	0.66	0.96
Gasoline, portable									
1½ in., 4,000 gal per hr.....	20	20	10	50	130	1,200	0.05	0.02	0.07
2 in., 10,000 gal per hr.....	20	20	10	50	360	1,200	0.15	0.07	0.22
3 in., 20,000 gal per hr.....	20	20	10	50	520	1,200	0.25	0.13	0.38
4 in., 40,000 gal per hr.....	20	20	10	50	1,100	1,200	0.45	0.42	0.87
6 in., 90,000 gal per hr.....	20	20	10	50	1,800	1,200	0.75	0.56	1.31
Rollers:									
Sheep's-foot, tamping									
4 ft wide.....	25	15	10	50	900	2,000	0.25	.....	0.25
8 ft wide.....	25	15	10	50	1,800	2,000	0.45	.....	0.45
12 ft wide.....	25	15	10	50	2,700	2,000	0.70	.....	0.70
Tandem wheel, variable weight									
3 to 5 tons, gasoline.....	15	12	10	37	3,200	2,000	0.60	0.35	0.95
5 to 8 tons, gasoline.....	15	12	10	37	5,300	2,000	1.00	0.67	1.67
8 to 10 tons, gasoline.....	15	12	10	37	5,800	2,000	1.10	0.82	1.92
8 to 12 tons, gasoline.....	15	12	10	37	6,300	2,000	1.15	0.82	1.97
10 to 14 tons, gasoline.....	15	12	10	37	6,800	2,000	1.25	0.95	2.20
5 to 8 tons, diesel.....	15	12	10	37	6,600	2,000	1.20	0.30	1.50
8 to 10 tons, diesel.....	15	12	10	37	7,200	2,000	1.35	0.43	1.78
8 to 12 tons, diesel.....	15	12	10	37	7,500	2,000	1.40	0.43	1.83
10 to 14 tons, diesel.....	15	12	10	37	7,900	2,000	1.45	0.43	1.88



## 520 CONSTRUCTION PLANNING, EQUIPMENT, AND METHODS

ESTIMATED HOURLY OWNERSHIP AND OPERATING COST OF CONSTRUCTION EQUIPMENT  
(Continued)

Equipment	Average annual expense, per cent of cost				Cost to owner	Hr used per year	Cost per working hr		
	De- pre- cia- tion	Major re- pairs	Inter- est, taxes, insur- ance	Total per cent of cost			Owner- ship	Fuel and other ex- pense	Total oper- ating
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Three wheels									
6 tons, gasoline.....	15	12	10	37	\$ 6,000	2,000	\$ 1.10	\$0.66	\$1.76
8 tons, gasoline.....	15	12	10	37	6,900	2,000	1.30	0.66	1.96
10 tons, gasoline.....	15	12	10	37	7,800	2,000	1.45	1.00	2.45
12 tons, gasoline.....	15	12	10	37	8,300	2,000	1.55	1.00	2.55
6 tons, diesel.....	15	12	10	37	7,100	2,000	1.30	0.25	1.55
8 tons, diesel.....	15	12	10	37	7,900	2,000	1.45	0.25	1.70
10 tons, diesel.....	15	12	10	37	9,000	2,000	1.65	0.43	2.08
12 tons, diesel.....	15	12	10	37	9,500	2,000	1.75	0.43	2.18
Saws:									
Chain, gasoline									
18-in. cut.....	33	15	10	58	300	1,200	0.15	0.05	0.20
24-in. cut.....	33	15	10	58	400	1,200	0.20	0.07	0.27
36-in. cut.....	33	15	10	58	415	1,200	0.20	0.07	0.27
48-in. cut.....	33	15	10	58	425	1,200	0.20	0.07	0.27
60-in. cut.....	33	15	10	58	450	1,200	0.25	0.10	0.35
Hand, electric									
4-in. blade.....	33	15	10	58	50	1,400	0.05	0.02	0.07
6-in. blade.....	33	15	10	58	60	1,400	0.05	0.02	0.07
8-in. blade.....	33	15	10	58	100	1,400	0.06	0.02	0.08
Tilting table, electric									
8-in. blade.....	25	10	10	45	175	1,600	0.08	0.02	0.10
10-in. blade.....	25	10	10	45	260	1,600	0.10	0.03	0.13
12-in. blade.....	25	10	10	45	420	1,600	0.15	0.04	0.19
14-in. blade.....	25	10	10	45	640	1,600	0.20	0.05	0.25
Tilting table, gasoline									
8-in. blade.....	25	15	10	50	210	1,600	0.10	0.02	0.12
10-in. blade.....	25	15	10	50	325	1,600	0.10	0.03	0.13
12-in. blade.....	25	15	10	50	510	1,600	0.20	0.03	0.23
14-in. blade.....	25	15	10	50	750	1,600	0.25	0.04	0.29
Scrapers, self-loading, with pneu- matic tires:									
Capacity, cu yd									
Struck Heaping									
3.6 4.5 4 wheels....	20	15	10	45	3,600	2,000	0.80	0.20	1.00
6.0 8.0 4 wheels....	20	15	10	45	6,000	2,000	1.35	0.25	1.60
9.0 11.0 4 wheels....	20	15	10	45	8,000	2,000	1.80	0.30	2.10
9.0 11.0 2 wheels....	20	15	10	45	6,000	2,000	1.35	0.30	1.65
15.0 20.0 4 wheels....	20	15	10	45	13,000	2,000	2.90	0.40	3.30
15.0 20.0 2 wheels....	20	15	10	45	11,500	2,000	2.60	0.40	3.00
17.0 20.0 4 wheels....	20	15	10	45	13,200	2,000	2.95	0.40	3.35

**ESTIMATED HOURLY OWNERSHIP AND OPERATING COST OF CONSTRUCTION EQUIPMENT**  
(Continued)

Equipment	Average annual expense, per cent of cost				Cost to owner	Hr used per year	Cost per working hr		
	De- pre- cia- tion	Major re- pairs	Inter- est, taxes, insur- ance	Total per cent of cost			Owner- ship	Fuel and other ex- pense	Total oper- ating
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	
Shores, adjustable, floor:									
4½ to 7½ ft. ....	25	15	10	50	\$ 14	.....	Per use	\$0.30	
6½ to 11½ ft. ....	25	15	10	50	15	.....	Per use	0.30	
7 to 13 ft. ....	25	15	10	50	15	.....	Per use	0.30	
8 to 15 ft. ....	25	15	10	50	16	.....	Per use	0.35	
Shovels, power, complete:									
Crawler, diesel engine									
½ cu yd. ....	25	15	10	50	15,000	2,000	\$ 3.75	\$0.45 4.20	
¾ cu yd. ....	25	15	10	50	19,000	2,000	4.75	0.55 5.30	
1 cu yd. ....	20	15	10	45	22,000	2,000	5.00	0.60 5.60	
1½ cu yd. ....	20	15	10	45	42,000	2,000	9.50	0.75 10.25	
2 cu yd. ....	17	15	10	42	56,000	1,600	14.75	1.10 15.85	
Crawler, gasoline engine									
½ cu yd. ....	25	15	10	50	14,000	2,000	3.50	1.05 4.55	
¾ cu yd. ....	25	15	10	50	17,800	2,000	4.45	1.20 5.65	
1 cu yd. ....	20	15	10	45	20,500	2,000	4.60	1.35 5.95	
1½ cu yd. ....	20	15	10	45	40,000	1,600	11.25	1.75 13.00	
Tower, elevator, tube steel:									
Light, single, 25 ft. ....	20	10	10	40	570	1,600	0.15	..... 0.15	
Light, single, 50 ft. ....	20	10	10	40	850	1,600	0.20	..... 0.20	
Light, single, 100 ft. ....	20	10	10	40	1,300	1,600	0.35	..... 0.35	
Light, double, 25 ft. ....	20	10	10	40	1,050	1,600	0.25	..... 0.25	
Light, double, 50 ft. ....	20	10	10	40	1,500	1,600	0.40	..... 0.40	
Light, double, 100 ft. ....	20	10	10	40	2,200	1,600	0.55	..... 0.55	
Heavy, single, 50 ft. ....	20	10	10	40	1,100	1,600	0.30	..... 0.30	
Heavy, single, 100 ft. ....	20	10	10	40	1,700	1,600	0.40	..... 0.40	
Heavy, double, 50 ft. ....	20	10	10	40	1,750	1,600	0.40	..... 0.40	
Heavy, double, 100 ft. ....	20	10	10	40	2,750	1,600	0.70	..... 0.70	
Tractors:									
Crawler, diesel engine, drawbar hp									
30-35. ....	25	15	10	50	4,800	2,000	1.20	0.25 1.45	
40-45. ....	25	15	10	50	6,800	2,000	1.70	0.27 1.97	
60-70. ....	20	15	10	45	8,300	2,000	1.85	0.42 2.27	
80-90. ....	20	15	10	45	11,000	2,000	2.50	0.54 3.04	
120-130. ....	20	15	10	45	15,000	2,000	3.40	0.78 4.18	
Four wheels, diesel, belt hp									
100-125. ....	20	15	10	45	13,000	2,000	2.90	0.75 3.65	
200-225. ....	20	15	10	45	17,000	2,000	3.80	1.35 5.15	
250-300. ....	20	15	10	45	20,000	2,000	4.50	1.80 6.30	



## 522 CONSTRUCTION PLANNING, EQUIPMENT, AND METHODS

ESTIMATED HOURLY OWNERSHIP AND OPERATING COST OF CONSTRUCTION EQUIPMENT  
(Continued)

Equipment	Average annual expense, per cent of cost				Cost to owner	Hr used per year	Cost per working hr		
	De- pre- cia- tion	Major re- pairs	Inter- est, taxes, insur- ance	Total per cent of cost			Owner- ship	Fuel and other ex- pense	Total oper- ating
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Two-wheel tractors and scrapers									
Tractor, hp, Scraper, cu yd									
100-125 6- 7.....	20	15	10	45	\$17,000	2,000	\$ 3.80	\$0.75	\$4.55
170-200 12-14.....	20	15	10	45	25,000	2,000	5.65	1.20	6.85
200-225 14-17.....	20	15	10	45	32,000	2,000	7.20	1.35	8.55
250-300 15-20.....	20	15	10	45	33,000	2,000	7.40	1.80	9.20
200-225 18-24.....	20	15	10	45	52,000	2,000	11.70	1.35	13.05
Tractor attachments:									
Angle dozers, cable									
13-ft blade.....	20	15	10	45	2,100	1,600	0.55	.....	0.55
13-ft 8-in. blade.....	20	15	10	45	2,300	1,600	0.65	.....	0.65
14-ft 6-in. blade.....	20	15	10	45	3,100	1,600	0.85	.....	0.85
Bulldozers, cable									
8-ft blade.....	20	15	10	45	1,350	2,000	0.30	.....	0.30
9-ft 6-in. blade.....	20	15	10	45	1,500	2,000	0.35	.....	0.35
10-ft blade.....	20	15	10	45	1,600	2,000	0.35	.....	0.35
11-ft 6-in. blade.....	20	15	10	45	2,000	2,000	0.45	.....	0.45
Bulldozers, hydraulic									
6-ft 8-in. blade.....	20	15	10	45	1,875	2,000	0.45	.....	0.45
8-ft blade.....	20	15	10	45	1,900	2,000	0.45	.....	0.45
9-ft 6-in. blade.....	20	15	10	45	2,200	2,000	0.50	.....	0.50
11-ft 6-in. blade.....	20	15	10	45	2,800	2,000	0.65	.....	0.65
14-ft blade.....	20	15	10	45	4,000	2,000	0.90	.....	0.90
Ripper, cable									
For 80- to 100-hp tractor	20	10	10	40	2,500	1,400	0.70	.....	0.70
For 100- to 125-hp tractor...	20	10	10	40	3,500	1,400	1.00	.....	1.00
Tractor and bottom-dump wag- ons with pneumatic tires:									
20 tons, 200 hp, diesel.....	20	15	10	45	25,500	2,000	5.75	1.20	6.95
38 tons, 240 hp, diesel.....	20	15	10	45	34,200	2,000	7.70	1.45	9.15
40 tons, 300 hp, diesel.....	20	15	10	45	38,000	2,000	8.55	1.80	10.35
Trench braces, per dozen:									
1½-in. screw, 30 in.....	20	10	10	40	70	.....	Per use	.....	0.25
1½-in. screw, 48 in.....	20	10	10	40	85	.....	Per use	.....	0.35
2-in. screw, 46 in.....	20	10	10	40	150	.....	Per use	.....	0.60
2-in. screw, 58 in.....	20	10	10	40	160	.....	Per use	.....	0.65
2-in. screw, 70 in.....	20	10	10	40	170	.....	Per use	.....	0.70
Trenching machines:									
Ladder type, diesel engine.....									
5-ft 6-in. depth.....	25	20	10	55	14,000	2,000	3.85	0.30	4.15
8-ft 6-in. depth.....	25	20	10	55	15,000	2,000	4.10	0.35	4.45
11-ft 0-in. depth.....	25	20	10	55	17,000	2,000	4.65	0.40	5.05
12-ft 6-in. depth.....	25	20	10	55	18,000	2,000	4.95	0.45	5.40
15-ft 0-in. depth.....	20	20	10	50	22,000	1,600	6.90	0.55	7.45
17-ft 0-in. depth.....	20	20	10	50	23,000	1,600	7.20	0.60	7.80

ESTIMATED HOURLY OWNERSHIP AND OPERATING COST OF CONSTRUCTION EQUIPMENT  
(Continued)

Equipment	Average annual expense, per cent of cost				Cost to owner	Hr used per year	Cost per working hr		
	De- pre- cia- tion	Major re- pairs	Inter- est, taxes, insur- ance	Total per cent of cost			Own- ership	Fuel and other ex- pense	Total oper- ating
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Ladder type, gasoline engine									
5-ft 6-in. depth.....	25	20	10	55	\$13,000	2,000	\$3.60	\$0.65	\$4.25
8-ft 6-in. depth.....	25	20	10	55	13,500	2,000	3.70	0.80	4.50
11-ft 0-in. depth.....	25	20	10	55	15,500	2,000	4.25	0.90	5.15
12-ft 6-in. depth.....	25	20	10	55	16,000	2,000	4.40	1.00	5.40
15-ft 0-in. depth.....	20	15	10	45	20,500	1,600	5.75	1.25	7.00
17-ft 0-in. depth.....	20	15	10	45	21,000	1,600	5.90	1.35	7.25
Trucks:									
Dump, diesel engine									
5 tons, 3½ cu yd.....	20	15	10	45	9,000	2,000	2.05	0.55	2.60
10 tons, 7 cu yd.....	20	15	10	45	12,700	1,800	3.20	0.75	3.95
15 tons, 10 cu yd.....	20	12	10	42	19,500	1,800	4.55	1.00	5.55
20 tons, 15 cu yd.....	20	12	10	42	28,500	1,600	7.50	1.80	9.30
35 tons, 20 cu yd.....	20	12	10	42	43,500	1,600	11.40	2.30	13.70
Dump, gasoline engine									
2 cu yd.....	25	15	10	50	5,400	2,000	1.35	0.85	2.20
3½ cu yd.....	20	15	10	45	7,500	2,000	1.70	1.00	2.70
5 cu yd.....	20	15	10	45	9,300	2,000	2.10	1.25	3.35
9 cu yd heavy duty.....	20	12	10	42	17,500	1,600	4.60	2.10	6.70
Stake, gasoline engine.....									
½ ton.....	33	15	10	58	1,600	2,000	0.50	0.70	1.20
1 ton.....	33	15	10	58	2,100	2,000	0.60	0.85	1.45
2 tons.....	25	12	10	47	5,000	2,000	1.20	1.00	2.20
3½ tons.....	25	12	10	47	6,000	2,000	1.40	1.10	2.50
5 tons.....	20	12	10	42	7,000	2,000	1.50	1.25	2.75
Welding equipment, electric arc, with cable, holders, clamps, etc.:									
Electric drive, portable.....									
200 amp d.c.....	20	12	10	42	280	2,000	0.06	0.24	0.30
300 amp d.c.....	20	12	10	42	540	2,000	0.12	0.30	0.42
400 amp d.c.....	20	12	10	42	610	2,000	0.13	0.37	0.50
600 amp d.c.....	20	12	10	42	850	2,000	0.18	0.52	0.70
Diesel-engine drive, portable									
300 amp d.c.....	20	12	10	42	2,190	2,000	0.46	0.30	0.76
400 amp d.c.....	20	12	10	42	2,500	2,000	0.53	0.38	0.91
600 amp d.c.....	20	12	10	42	2,900	2,000	0.61	0.59	1.20
Gas-engine drive, portable									
200 amp d.c.....	20	15	10	45	950	2,000	0.22	0.33	0.55
300 amp d.c.....	20	15	10	45	1,390	2,000	0.32	0.68	1.00
400 amp d.c.....	20	15	10	45	1,575	2,000	0.35	0.90	1.25
600 amp d.c.....	20	15	10	45	1,810	2,000	0.41	1.34	1.85
Transformers, a.c., stationary									
220 to 440 volts primary.....									
200 amp, 30 volts.....	20	10	10	40	250	2,000	0.05	0.20	0.25
300 amp, 40 volts.....	20	10	10	40	400	2,000	0.08	0.37	0.45
500 amp, 40 volts.....	20	10	10	40	625	2,000	0.13	0.57	0.70



## APPENDIX B

### CONSTRUCTION EQUIPMENT DEPRECIATION SCHEDULE

The information given in this table was published in the *Engineering News-Record* on Mar. 29, 1951, and is included through the courtesy of the *Engineering News-Record*.

The January, 1942, revision by the U.S. Bureau of Internal Revenue of its 1931 Schedule F of probable useful life and depreciation rates allowable for income tax purposes is still in effect. Annual depreciation rates are computed on a straight-line basis. For example, for a probable useful life of 10 years, annual depreciation is 10 per cent of the original cost; for a probable useful life of 20 years, annual depreciation is 5 per cent of the original cost. The probable useful life of construction equipment is given below:

Equipment	Life in years	Equipment	Life in years
Automobiles:		Blowers, mechanical	10
Light	2	Boilers:	
Medium	3	Upright	7
Heavy	5	Locomotive	15
Backfillers, power:		Stationary	20
Light	3	Borers, wood, portable	3
Medium	5	Boring apparatus, test	10
Heavy	6	Boxes, mortar and batching	3
Tractor	5	Brakes:	
Barges:		Bending	10
Steel	30	Cornice, sheet metal	22
Wood	25	Buckets:	
Batcher plants:		Cableway	6
All steel, demountable	10	Clamshell	6
Steel frame, wood bin	10	Concrete	5
Stationary	14	Elevator	5
Wood frame and bin	7	Orange-peel	6
Batch, measuring devices	4	Bail, pivot turnover	5
Benders, bar	5	Scraper or dragline	6
Bending blocks	10	Buggies:	
Bending machines:		Concrete	3
Angle	15	Timber	3
Pipe and rail	10	Bulldozers:	
Bins:		Gradebuilders	8
Steel and concrete	6	Tractor	4
Steel	12	Burner equipment, gas and oil	12
Wood	8	Cables, wire	4
Bin frames, steel	6	Cableways, cable only	3
Blacksmith shop, portable	4	Cableway carriage	5
Blocks, pulley, differential	6	Camping equipment	3

Equipment	Life in years	Equipment	Life in years
Capstans, electric	10	Chain, portable	6
Cars:		Portable	5
Ballast spreader	10	Scraper	6
Batch box, steel	5	Cranes:	
Boarding and tool	20	Bridge and cantilever	20
Concrete	8	Crawler, electric:	
Derrick, bridge	10	2½-5 tons	5
Dump, steel	8	10-15 tons	7
Dump, wood	6	20 tons and over	9
Flat, steel	12	Crawler, gasoline:	
Flat, wood	10	2½-5 tons	5
Hand, hopper, scale	10	10-15 tons	9
Skip hoist	10	20 tons and over	12
Tank	20	Locomotive, gasoline	7
Carts, concrete	3	Steam:	
Carts, tool, steel	4	2½-5 tons	6
Cement gun machines	4	10-15 tons	10
Chains:		20 tons and over	12
Hawsers and lines	6	Locomotive	10
Power, transmissions	5	Dragline	10
Channelers, rock	6	Universal (gasoline 2½-5 tons)	
Chipping and calking tools,		mounted on 10-ton truck	6
pneumatic	3	Dock or wharf, travel	20
Chutes, concrete, gravity	2	Craneways:	
Clamps, column forms	5	Steel	15
Cleaning machines for exterior of		Wood	10
buildings, steam or sand	15	Crushers, rock:	
Compressors:		Portable	8
Belt driven	10	Stationary	10
Electric, portable	8	Cutters:	
Gasoline, portable	6	Bar, power	5
Motor-truck unit	5	Corrugated iron, hand	10
Steam, portable	6	Cutting and welding outfits	4
Concrete machines, pneumatic	5	Derricks:	
Concrete mixers:		Boat	10
Electric	5	Circle swing, hand	8
Gasoline, 3½S, 5S, 7S	3	Crab, hand	16
Gasoline, 10S, 14S,	4	Crab, power	10
Gasoline, 21S, 28S	5	Guy, steel	12
Paving, gasoline, steam	8	Guy, wood	8
Truck-mounted	5	Stiffleg, steel	12
Controllers, motor	12	Stiffleg, wood	8
Conveyors:		Diggers, clay, pneumatic	3
Belt elevating, portable	3	Draglines:	
Belt elevating, stationary	6	Electric:	
Buckets		½-¾ cu yd	6
Cable-drag	6	1-1½ cu yd	8
Monorail	15	2 cu yd and over	10



Equipment	Life in years	Equipment	Life in years
Gasoline:		Extinguishers, fire	3
$\frac{1}{2}$ - $\frac{3}{4}$ cu yd	5	Fans, exhaust	15
1-1 $\frac{1}{2}$ cu yd	9	Finishing machines	4
2 cu yd and over	12	Floats, bridge, steel	5
Steam:		Forges, gas or oil	10
$\frac{1}{2}$ - $\frac{3}{4}$ cu yd	6	Forms:	
1-1 $\frac{1}{2}$ cu yd	10	Concrete, metal pans	5
2 cu yd and over	12	Concrete supports, adjustable	4
Dredges:		Pavements, steel	4
Clamshell	16	Pipes, steel	3
Dipper	8	Walls, steel	5
Hydraulic	20	Tunnels and conduit, steel	4
Pipe	10	Furnaces, metal melting:	
Drill boats	12	Coal fired	10
Drill points, well	5	Electric	12
Drills:		Gas or oil fired	7
Airdrifter	3	Generator sets:	
Electric or pneumatic, hand, for wood or metal	5	Steam engine	12
Hand, electric	3	Turbine, headlight or floodlight	4
Rock, electric	3	Gin poles, steel	10
Jackhammer	3	Graders:	
Steam	5	Blade, road:	
Traction, well	7	7-8 ft blade	4
Tripod	7	9-10 ft blade	5
Tunnel carriage	5	Over 10 ft blade	8
Well	10	Elevating	8
Elevators:		Form subgrade planers	6
Bucket, stationary	6	Rooters, wheel	5
Cage, steel tower	5	Grinders:	
Engines:		Metal surfacing	15
Blowing	12	Saw filers and setters	14
Fire	7	Surface, concrete	4
Gasoline	10	Hammers:	
Marine	20	Electric	3
Oil	20	Pneumatic riveting	3
Plumbing	14	Heaters, asphalt, tar, and pitch kettles	4
Steam	11	Helmets, gas and diving suits, and equipment	10
Excavators:		Hoists:	
Cableway, complete	4	Chain	6
Trench, gasoline:		Air, electric or steam	8
Depth 7-12 ft	6	Electric monorail or post	5
Depth 18 ft	8	Gasoline	6
Trench, steam:		Hand power	8
Depth 7-12 ft	8	Slew, electric	8
Depth 18 ft	10	Steam	12
Trench, vertical boom	5		
Wheel or ladder type	5		

Equipment	Life in years	Equipment	Life in years
<b>Hose:</b>		<b>Motors:</b>	
Fire, linen or rubber lined		Electric, small	8
cotton	5	Electric, medium	10
Metal, flexible	10	Electric, large	12
Oil	5	Hydraulic	5
Reel or cart	10	Pneumatic	5
Rubber, air, steam or water	10	Mowers, right of way	5
Inundators, batch	4	<b>Pile drivers:</b>	
<b>Jacks:</b>		Railroad	10
Hydraulic	8	Barge	8
Rail	25	Track	12
Ratchet	8	Steam, on skids	10
Screw	5	<b>Pile hammers, steam or air:</b>	
Push and pull	3	Light	4
Jibs, steam	17	Medium	5
Jointers, bench, electric, steam or		Heavy	10
gasoline	5	<b>Pipe:</b>	
Ladders, steel	3	Black or galvanized	4
Ladles, metal	7	Wood	5
<b>Lathes:</b>		Wood and steel combination	6
Metal working	15	<b>Pipe lines and fittings for floating</b>	
Wood working	17	dredges	10
<b>Levee construction:</b>		Pit and quarry plants	6
Draglines	8	<b>Planers:</b>	
Shovels	8	Metalworking	15
Tower excavators	12	Woodworking	20
Light plant	4	<b>Plows:</b>	
Lighters	22	Furrow	3
<b>Loaders, bucket:</b>		Rooter	6
Crawler and portable	5	Presses, drill	12
Stationary	6	<b>Pumping units:</b>	
<b>Locomotive batteries</b>	4	Electric:	6
<b>Locomotives, industrial:</b>		Gasoline	6
Diesel	10	Highway contractor's pump	4
Electric	16	Piston	5
Gasoline, up to 10 tons	8	Steam centrifugal	10
Gasoline, 10-20 tons	15	<b>Pumps:</b>	
Gasoline, over 20 tons	20	Air lift	10
Steam, up to 10 tons	8	Centrifugal, humdinger, impulse	6
Steam, 10-20 tons	18	Hydraulic	15
Steam, over 20 tons	20	Oil	10
<b>Locomotives, standard gauge</b>	30	Steam piston unit	6
<b>Magnets, lifting</b>	15	Testing for pipe lines	15
<b>Mixers:</b>		Punches, hydraulic	20
Portable mortar	3	Punches for steel, power	15
Less than $\frac{1}{2}$ cu yd	6	Rails, steel	10
Over $\frac{1}{2}$ cu yd	8	Razing equipment for bldgs	8
Caterpillar	8	Reamers, electric or pneumatic	3



Equipment	Life in years	Equipment	Life in years
Riveters, pneumatic	5	Switches:	
Rollers:		Portable	4
Concrete finishing, steel	10	Stationary	5
Road, gasoline or steam	10	Tampers, backfill, pneumatic	3
Sandblast outfits	10	Tamping machines	10
Sawmills, portable	10	Tanks, gasoline, storage, relay	6
Saws:		Tanks, water or air storage, steel	10
Band, cutoff and rip, power	10	Thread cutting machines, pipe	10
Hand, electric and pneumatic	3	Tongs, chain	4
Saws and woodworkers:		Towers, cableway:	
Steel frames	10	Steel	6
Wood frames	5	Wood	3
Scales, large track and wagon	20	Steel boom with counterweights	5
Scarifiers:		Tractors:	
Attachments	4	Electric:	
Blocks, steerable	5	3 tons	3
Drag, all steel	4	5 tons	5
Grader type	4	10 tons	6
Scrapers:		20 tons	8
Blade, carryall	6	Gasoline or steam:	
Fresno	4	3 tons	4
Slip	2	5 tons	6
Wheel	5	10 tons	8
Screens and bunkers, for gravel pits only	5	20 tons	10
Screens, revolving	6	Trailers:	
Sharpeners, drill	8	Dump, steel or wood	10
Shears, for steel, hand	10	Platform, wood	4
Shores, adjustable	4	Drop platform, heavy duty	5
Shovel attachments	6	Transformers, car	10
Shovels:		Trucks, automobile, dump:	
Electric or gas, crawler, or wheel:		$\frac{1}{2}$ - $\frac{3}{4}$ cu yd	3
$\frac{1}{2}$ - $\frac{3}{4}$ cu yd	5	1-1 $\frac{1}{2}$ cu yd	5
1-1 $\frac{1}{2}$ cu yd	6	2 cu yd and over	8
2 cu yd and over	8	Tugs, screw-propelled, steam or gasoline	25
Steam, crawler or wheel:		Vises	5
$\frac{1}{2}$ - $\frac{3}{4}$ cu yd	7	Wagons:	
1-1 $\frac{1}{2}$ cu yd	8	Dump, steel or wood	6
2 cu yd and over	10	Road oilers, steel tank	10
Railroad, steam	10	Tank or sprinkler:	
Tunnel	4	Steel	10
Spouting plants, concrete	4	Wood	8
Spraying equipment, paint	12	Washers, gravel	3
Spreaders, stone:		Welders, acetylene or electric	10
Hopper wagon	5	Wheelbarrows	2
Steel box	5	Winches, electric or pneumatic	10
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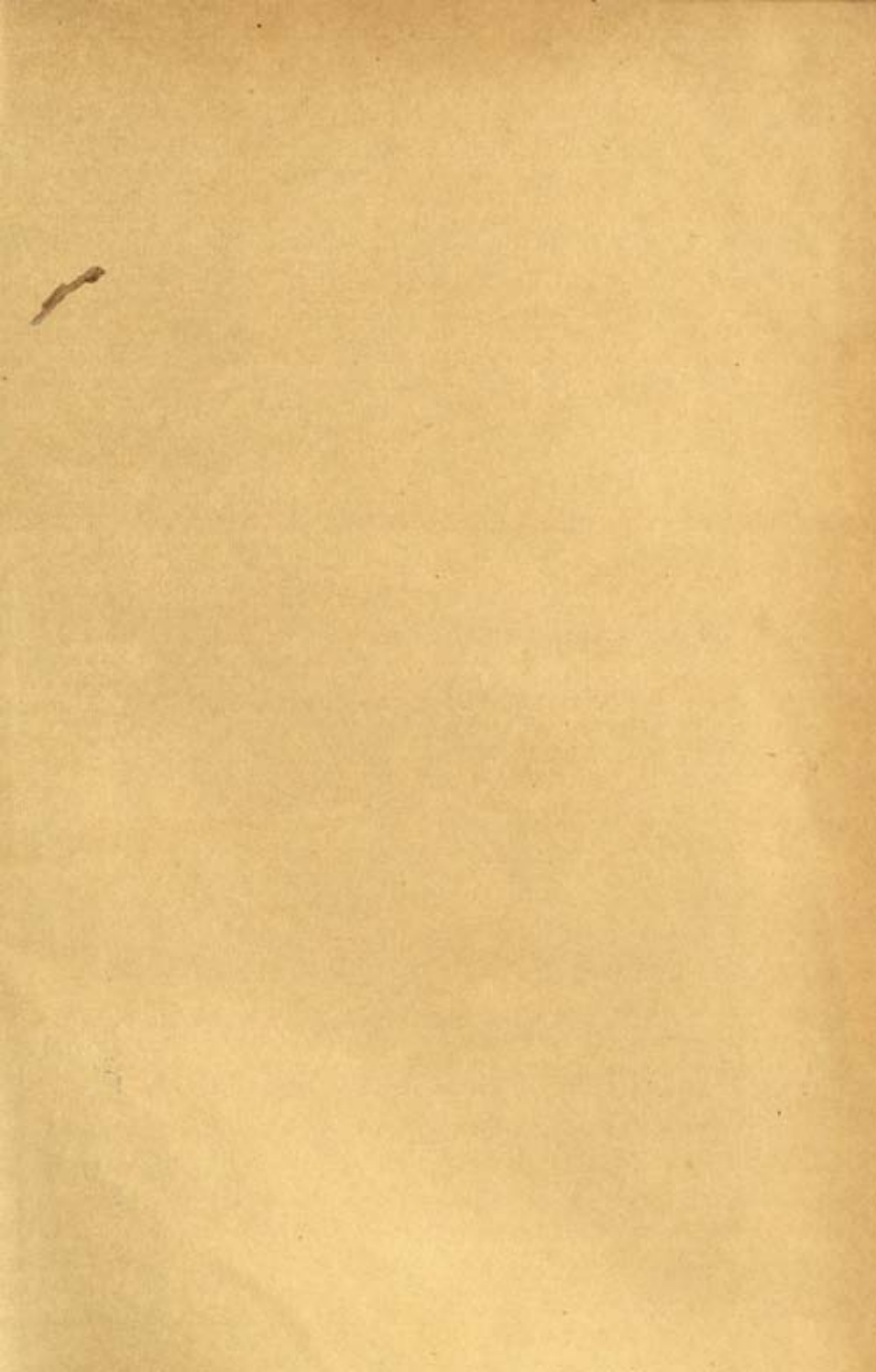
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