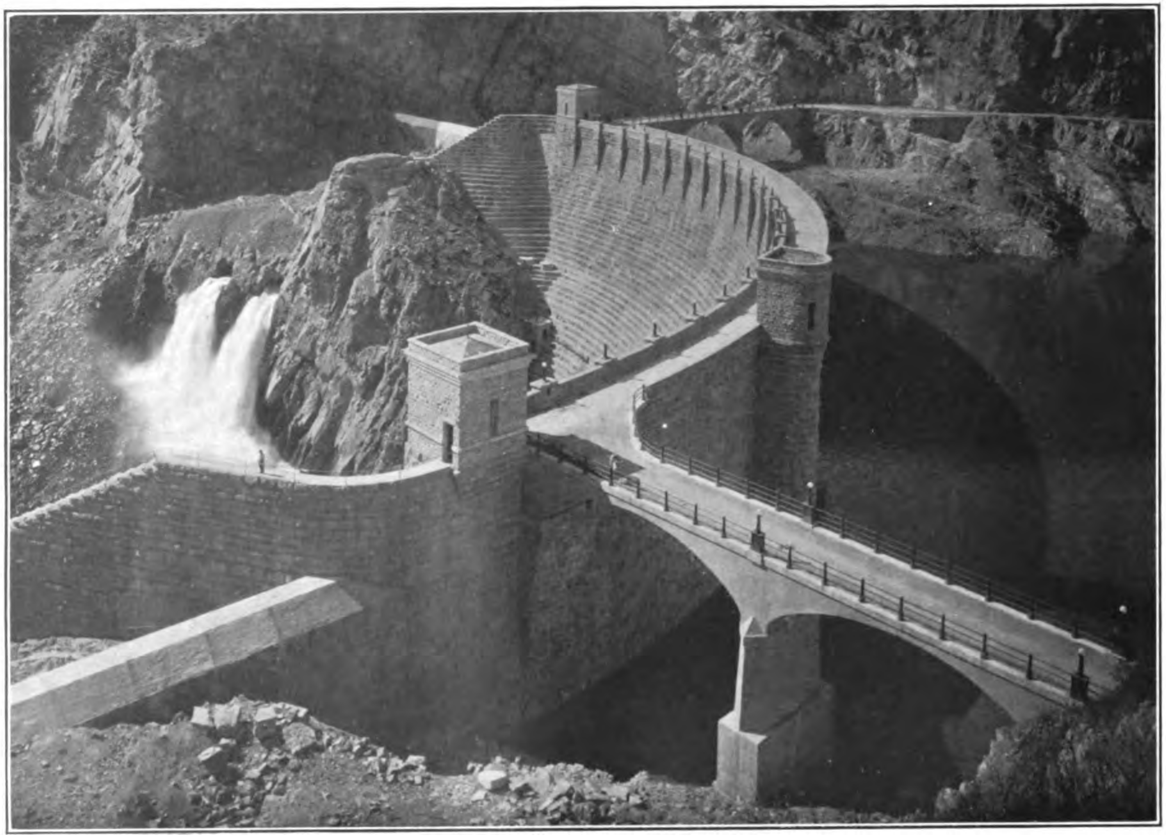
CONSTRUCTION OF MASONRY DAMS



Roosevelt Dam. (*Frontispiece.*)

CONSTRUCTION

OF

MASONRY DAMS

BY

CHESTER W. SMITH
CONSULTING ENGINEER

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PREFACE

This book has been written from the point of view of the constructing engineer. It discusses only those features of design which are or may be variable with each particular case, or which, from their nature, are better known and appreciated by the constructor as the work progresses, than by the designer before the work starts.

Many volumes have been written upon the design of masonry dams; the principal features and methods have been reduced to what may be called an accepted, standard practice. It is hoped that the present work will supply the details of construction and supervision which have not been adequately covered heretofore. For a number of years descriptions of particular dams have appeared in the engineering periodicals, but little attention has been given in these articles to the general principles of construction involved.

Within the limitations of this book it was not possible to cover the subject in all of its ramifications. Earth work, rock excavation, cement, pumping and many similar subjects have been passed over entirely either because adequate treatment was impossible in such a work as this, or because they are fully covered in other books. Other subjects, such as power, are touched upon in the most elementary manner and solely from the point of view of the man engaged in dam construction. Thus the material upon source and distribution of power are mere suggestions, while the subjects of power required, cost of power, etc., are treated somewhat more fully.

It is probably true that for one dam actually built, several are projected and more or less thoroughly examined. The inception and promotion of such projects and their examination by financiers, result in a number of engineers being called upon for estimates of cost. Such estimates must often be prepared within a limited time and for a limited expenditure. To facilitate the rapid and reasonably accurate treatment of such estimates, a chapter has been added on this subject, and this, in fact, was one of the chief purposes of the book. It is hoped that the cost figures will prove a reliable and useful guide for estimates. Labor costs, as always, must be used with caution, and only after a careful comparison of conditions. The plant costs should be more generally applicable, as the possible margin of error in them would result in but an insignificant error in the total estimated cost of the work. When time permits, or the plant is especially large, additional figures are advisable, but in the majority of cases the figures presented will, it is hoped, be a useful guide.

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December, 1914

C. W. S.

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INTRODUCTION

Of all structures designed and built by engineers, a masonry dam is probably the one about which there is the least amount of exact knowledge. Lest there be a disposition to question this proposition, let us examine it.

Other structures may usually be resolved into their separate parts or members, and the function of each member may be analyzed to the end that its design may and does answer the purpose. Experiments may often be conducted upon actual full-sized members, the conditions of their construction may be accurately known and controlled and thus the adverse conditions may be eliminated.

A masonry dam is usually assumed and designed as an entirely homogeneous structure; in other words, as a structure of one member. The only tests are use and failure. Use, however long continued, is a negative test which conveys very little information; failure conveys only a sharp admonition to be more careful next time and usually leaves the specific cause unsatisfactorily determined. We do know that many masonry dams have stood for years, and that some have failed.

In the case of a failure a reason is usually assigned that satisfies our intelligence, but only because it seems to conform to other, more common and more appreciable experiences, and not because correct relative values can be assigned to the various possible contributing causes. Experiments under comparable conditions and on a working scale are impossible except as we regard all actual dam failures as experiments.

Now to take up the various features one by one. We assume the rock foundation to be unyielding, and such is probably the case as far as any effect upon the masonry is concerned; but it is probably often the case that the superimposed weight of masonry produces new conditions in the rock and in the possible channels for leakage. The function of other foundations is simply to sustain weight, and the determination of their adequacy is a relatively simple matter compared with the problems which arise in connection with the foundation for a dam. In the latter case the foundation must be tight enough to prevent any significant amount of leakage under the dam, in addition to sustaining the weight. The obscurity of many of the conditions, the fact that the foundation must be prepared and accepted once for all with very limited possibilities for future inspection, repairs or reinforcement combine to render this a problem calling for the most experienced judgment.

Though we may know the strength of mortar briquettes or concrete cubes manufactured under conditions insuring a high degree of uniformity, we can obviously know very little about the behavior of masses of masonry comparable in size to the structure under consideration. Temperature conditions will vary widely during construction, and even if it were possible to make a sufficient number of observations, still the effect of the changes could not be accurately determined.

The point of least uncertainty is the load that the dam has to carry. We may know within very close limits the weight per cubic foot of the masonry that resists the weight of the water, but immediately we are in the midst of an argument as to whether to reduce that figure by 10 per cent., 20 per cent. or 30 per cent., to allow for an unknown but possible uplift pressure.

We assume that a mass of masonry may resist by a certain amount any tendency to slide, when every observation we have as to coefficient of friction was made under circumstances so widely different as to be absolutely inapplicable. We make no allowance for ability to resist shearing stress, when we know that a sliding failure is inconceivable without shear in a vertical plane.

We proportion a dam to resist overturning, and assume for the purpose no tensile strength in the masonry, though an actual overturn would be resisted by tension and also by shear.

We assume an ice thrust equal to the crushing strength of the ice, without any reason to suppose that the ice can exert any such pressure. We regard the masonry as homogeneous, when every large dam is constructed under conditions such as to produce initial temperature stresses. Under certain conditions a curved dam recommends itself to our reason and intelligence as being more stable for the same amount of masonry, or of equal stability for less masonry, yet when we try to analyze the structure we are immediately involved in a mass of assumption which may with equal justice be made to yield widely differing results.

When we can assign but one limit to the range of values possible for any factor entering into the design, we work close to that limit; when both limits are known we are cautious and work nearer the more conservative one, trusting that the margin of safety will cover the conditions of unknown value. Thus the limits for value of tension and shear are zero and some unknown finite quantity, and we assume zero. The amount of uplift pressure is between zero and full head applied to full area, hence we are cautious and work near the latter.

For ice thrust we assume a fair value as its crushing strength, and a maximum thickness. We apply it all, knowing that the truth is probably much less. Finally, we trust that the margin of safety is sufficient allowance for the factors of foundation, temperature and settlement cracks and initial stress, to whose limits we can assign no values though they may appear small. Though such procedure is usually sound, an attempt will be made to show that certain fears are groundless or much exaggerated, and that certain alternate measures are entirely adequate.

Very many dams (and this is especially true of large dams) are designed by one person and constructed by another. However each may be assisted or consulted by the other the fact remains that the two functions are in different hands, and there are occasional indications that they are imperfectly correlated. This condition may manifest itself in several ways as follows:

First.—In the specification of expensive details or methods to secure some real or fancied effect which the constructor could sufficiently approximate at less cost.

Second.—The designer, not knowing the constructor, or perhaps being chary about trusting to his experience and fidelity, not unnaturally produces a conservative design. It is one intended to survive considerable laxity or departure from high standards of construction—and the foolproof design must be expensive.

Third.—Certain features or methods of construction seem to be overlooked in some designs because the designer often possesses a very inadequate appreciation of their adequacy as compared with that possessed by the constructor.

The past few years have seen revolutionary changes in both the design and construction of masonry dams, and the next few will see further changes in construction methods if not so much in design.

The older dams were of rubble masonry, containing 60 per cent. to 70 per cent. of stone, laboriously laid up by hand with mortar and spalls. The upstream faces were of cut stone with comparatively thin well-pointed joints. Although some leakage or seepage is almost invariably present, it is probably under such conditions that no harmful pressures exist, and it is rarely sufficient in amount to present a scandalous appearance.

The present type of dam is of cyclopean concrete containing far less large stone (down to 25 per cent. or less) with concrete or concrete block faces. The resulting prominent cracks are cared for by expansion joints; and the leakage (whether more or less than in the rubble type) is intercepted and removed by a drainage system.

In future dams the stone will be frankly discarded on account of the plant cost of equipping to handle it. The process will be reduced to the very simple one of manufacturing a certain amount of concrete and transporting it a certain distance. Much higher rates of progress and much lower unit costs will be realized.

Changes in design will probably consist largely in the development of the expansion joint and drainage systems; also in making greater use of the possibilities of concrete for architectural effect.

CONSTRUCTION OF MASONRY DAMS

CHAPTER I

EXPLORING THE SITE

In advance of any construction work, a thorough knowledge of the character of the foundation is necessary. This may be obtained by a study of surface indications, together with test pits, wash borings and core borings into the rock.

While a casual examination of the surface may indicate that a masonry dam is feasible, still a thorough exploration is necessary in order

To Determine:

First.—Whether or not rock exists over the entire site of the dam.

Second.—Whether the rock is hard and sound enough to serve as a foundation, and free from seams or joints that would permit an objectionable amount of leakage.

Third.—The depth:

(a) Of earth or loose material overlying the rock.

(b) Of disintegrated or unsatisfactory rock to excavate in order to reach sound, tight bottom.

Fourth.—The character, amount and location of the supply of stone and sand to be used in the masonry.

Answers to Nos. 1 and 2 above answer directly the question whether a masonry dam is possible.

The answer to No. 3 determines the amount of material (excavation and masonry) to be handled, and besides bearing on the design, is a question of cost which in fact may determine the practicability of the structure.

No. 4 is obviously of vital importance as affecting the cost.

As a general proposition a masonry dam should be founded upon rock. Under certain circumstances some slight departures from the general rule are allowable. While somewhat less rigid practice might possibly be permissible in some cases, such an admission would still be a very long way from condoning the construction in several recent cases of dam failures.

This is all prefatory to saying that while the depth and character of the material overlying the rock should be determined by borings, the borings are primarily to determine depth. The character of the material, while most interesting and useful in connection with the questions of cost of excavation, design of temporary works, amount of pumping, etc., is not required for such a primary purpose as judgment of its adequacy as a foundation.

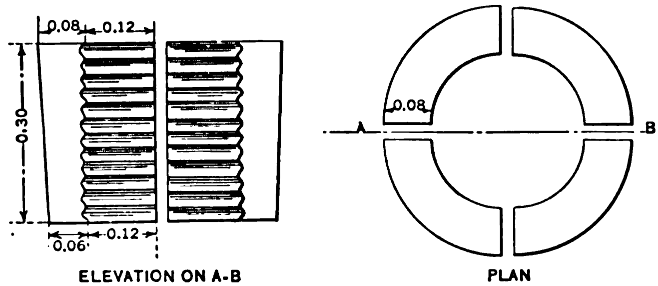


FIG. 1.—Wedges used with clamp to pull casing.

Wash Drill Borings.—In order to determine whether rock exists and the depth to the rock (questions Nos. 1 and 3a) the procedure is to cover the site with wash drill borings. Briefly the method is to

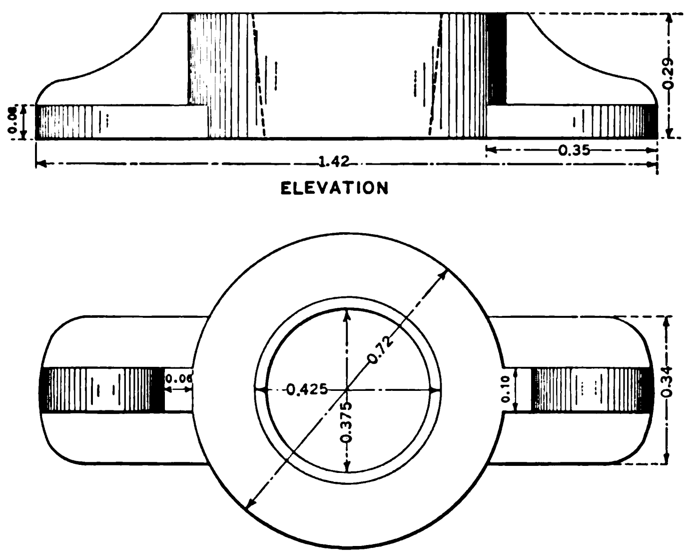


FIG. 2.—Clamp to pull casing used with pries or jacks.

drive down to rock an iron pipe of suitable size, say $2\frac{1}{2}$ in. inside diameter. Inside of this casing, so called, and accompanying it as it is being driven, is a smaller pipe called the drill rod furnished with a chisel point and supplying a stream of water. The function of the inside pipe is to loosen the material which is being driven through and wash it to the surface, thus facilitating the driving and furnishing samples of the material.

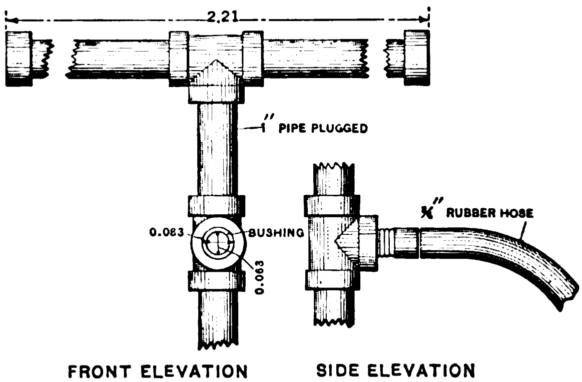


FIG. 3.—Upper end of drill rod, showing handle and the hose supplying wash water.

For doing this work there are several different rigs which, however, vary only in minor particulars. Figs. 1 to 9 illustrate the details of the outfit very satisfactorily employed on the Boston Metropolitan

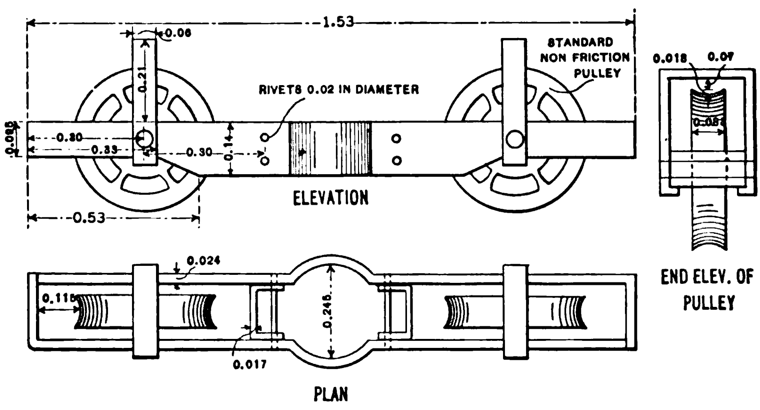


FIG. 4.—Yoke.

Water Works. The casing and drill rod are introduced, as required, in lengths of about 6 ft.; the casing is inserted just below the drive head (see Fig. 9), and the drill rod just below the hose connection.

The hammer slides up and down on a length of casing screwed into

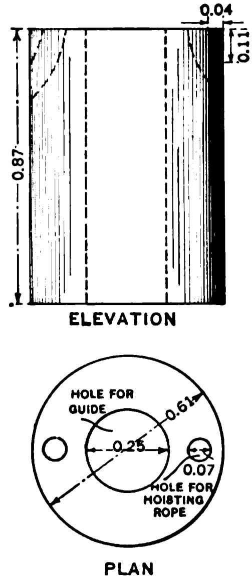


Fig. 5.—Light hammer.

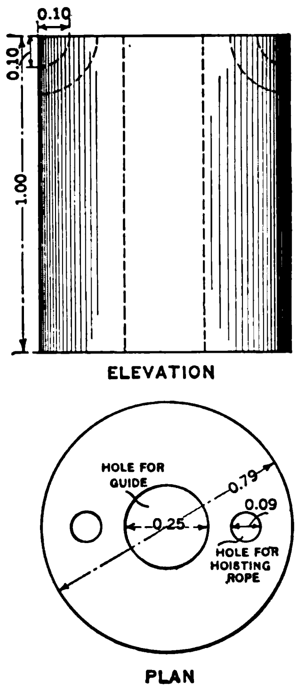


Fig. 6.—Heavy hammer.

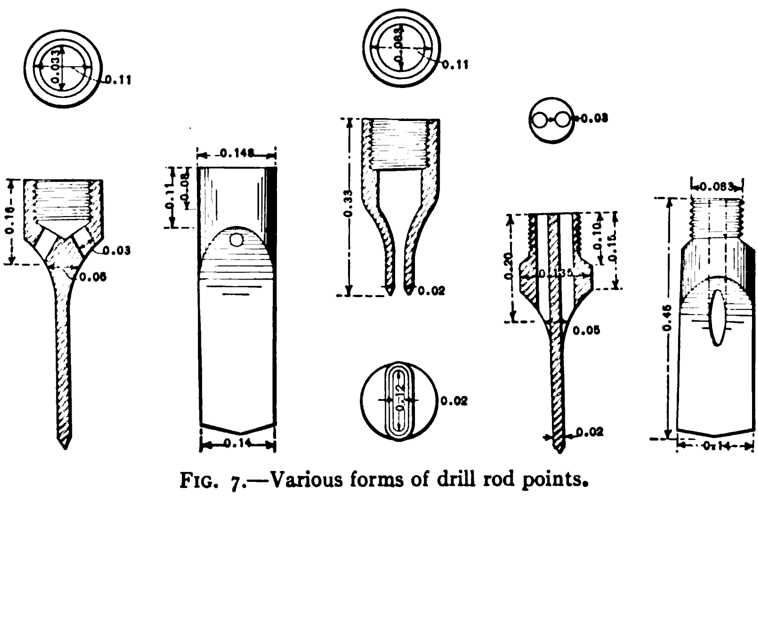


Fig. 7.—Various forms of drill rod points.

the top of the drive head; ropes from the hammer go over the pulleys in the yoke. The yoke is mounted on a coupling 2 in. or 3 in. below the top of the same length of casing.

Two or four men are employed on the ropes according to the weight of the hammer. One or two men, according to length and weight of drill rod in use, stand upon planks supported at any desired height by two wooden horses and operate the drill rod.

Though the casing as used on the Metropolitan Water Works had outside couplings it has since been amply demonstrated that inside couplings are much to be preferred, on account of the lesser resistance from the material penetrated.

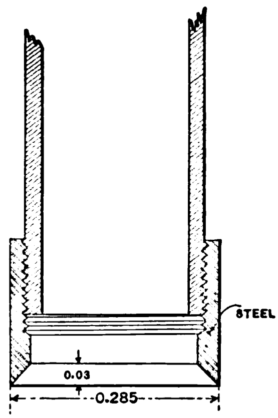


FIG. 8.—Shoe for bottom of casing.

These borings should cover the entire site and some consideration should be bestowed upon the order in which they are taken, so that each boring as made may settle the largest possible amount of remaining uncertainty; that is to say, the first few holes should properly be broadly scattered in order to supply general information, leaving the details to be filled in later. The results from each hole assist in determining the number and location of further adjacent holes which may be necessary.

Difficulties and Cost of Wash Borings.—This method is intended only to locate the surface of the ledge, as it is not adapted to penetrating rock as rock is generally classified. The most difficult material to go through is gravel or hardpan with many boulders. The smaller boulders may very often, by persistent drilling with the drill rod, be broken, or rolled aside sufficiently so that the casing may be driven through or by them. An abundant stream of water will assist materially in forming a lateral cavity into which the boulder may be pushed out of the way.

For boulders large enough to resist this treatment it is often necessary to employ dynamite. This explosive may be lowered through the casing until it rests on the top of the boulder; or it may be necessary even to drill a hole in the boulder. Before exploding the dynamite the casing should be drawn up 2 ft. or 3 ft. to save it from damage.

In some kinds of loose material it is necessary that the casing be kept driven several inches in advance of the drill rod in order that the material may be washed up; that is to say, if the drill rod were in advance so much water might be lost in the loose ground that none

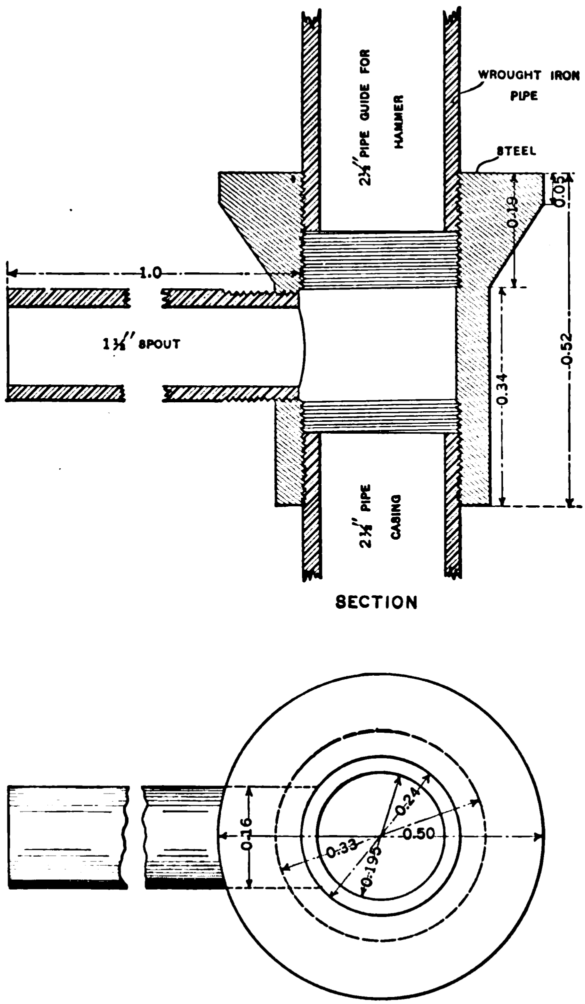


FIG. 9.—Drive-head.

would come up. On the other hand, in fine sand containing sufficient clay binder it is often possible to run the drill rod far and rapidly in advance of the casing. In some borings under the observation of the writer the drill rod was run as much as 175 ft. as fast as the workmen could couple on the lengths. In fact, many borings after entering such material were run down to ledge at similar depths in a small fraction of the time required to drive the casing.

A gang for making wash drill borings should consist of a foreman who can take samples and record all measurements intelligently, also take any other notes that will assist in interpreting the results. There should be (depending on the length of casing being driven, and consequent weight of hammer) from two to four laborers on the hammer and one or two on the drill rod. This is based on the assumption that the water is supplied under pressure; it may be necessary to use a hand force pump and take water from a nearby pool or stream, in which case about two more laborers are required.

If the borings are to continue for some time, and several gangs are employed, it may be desirable to install a power-driven pump. For several gangs working not too far apart one foreman will be sufficient, in which case the most intelligent laborer should handle the drill rod and be given some coaching so that he may do it intelligently.

It is obvious that the costs of such work may vary widely depending

(a) Very largely on the character of the material encountered, the ease or difficulty of driving the casing; also whether or not the casing must be driven to the full depth of each hole.

(b) Somewhat on whether the holes are scattered, or bunched, this affecting the cost of moving not only the boring apparatus, but the also possible taking up and relaying of the water-supply line.

(c) Somewhat upon the cost of the water supply.

(d) Very little upon the cost of the apparatus, as that is not much.

(e) Somewhat upon the total amount of the work.

(f) Very little upon season, or temperature conditions.

Roughly it may be said that the cost will range between 25 cents and $1 per lin. ft.; although lower costs have been recorded under particularly favorable conditions, and much higher costs might apply in certain localities, namely, to holes in the most obstinate material.

Interpretation of Results.—The water together with all the material washed up should be caught in a tub; the samples should be preserved in small bottles with identifying labels, the location of the holes be properly shown upon topographical plans and the material encountered at various depths in each hole be properly noted upon profiles or sections.

It is obvious that gravel or stones too large to be washed up between the drill rod and the casing will not be represented in the sample. The sample is also often deficient in that some of the finer material is usually not allowed to settle in the tub, but is lost in the overflow. Nothing short of preserving the entire effluent until the finest material had completely settled would entirely obviate this latter difficulty; even then the settled material would be a redistribution (through the process of settling through water) of the actual material encountered.

It has been attempted without much success to obtain a continuous (though of course inverted) sample; by diverting (through a hole in the bottom of the spout to the tub) a continuous certain per cent. of the effluent. The material thus diverted caught in a long narrow box constructed, for observation, with one glass side and having corresponding depths marked at intervals as the sample grows. After all much depends upon the honesty, experience and intelligence of the man directly in charge and upon the completeness of his notes. The "feel" of the drill rod, the color and amount of the effluent, whether or not the drill rod can be advanced beyond the casing, etc., are all instructive to the drill runner and to the man who interprets the results.

The most illuminating note that can be made is as to the amount of the effluent compared with the water supplied. If all the water is recovered it indicates an impervious material and the effluent will undoubtedly show considerable clay; on the other hand, if all the water is lost when the drill rod is advanced, and much of it even with the casing in advance, quite open sand or gravel is indicated, and the effluent will be correspondingly clear.

Boulders, unless very large, can be differentiated from ledge by the rebound and ring of the drill rod pounding on it because the ledge gives a more springy and ringing rebound. By driving the casing to whatever rock is encountered, and persisting with the drill rod, easily identified rock chips can almost always be obtained to assist in the diagnosis of boulder or ledge.

After the boring is finished the casing should be left in such holes as it is desired to extend down into rock by means of a diamond drill. In many cases it will be of value to observe the elevation and fluctuations of the ground water in the casing.

Core Borings in the Rock.—The site of any important dam should be carefully studied by a competent geologist, who should not only be conversant with the geological history of the entire region, but should be afforded time and opportunity to study the immediate vicinity more minutely.

It is well to have in mind that there is usually a geological reason for the location of the stream, *i.e.*, why it is where it is instead of somewhere else. This reason may exist in the rock as a longitudinal fault, or as a stratum of more easily eroded rock, and is much more likely to be uncovered in the bottom of the foundation than it is part way up the side. Stratification far from the horizontal indicates that water must travel by a much longer and more devious course in order to escape under the dam.

It may be taken almost as an axiom that the rock under a river bed is in better condition than the rock exposed above, and will in all probability require less depth of stripping to arrive at a suitable foundation. The rock below water level, besides being more recently exposed by erosion, is protected from the disintegrating influences that are at work above; in fact, under a stream that is actively at work on its rock bed it may be expected that little excavation will be required.

In a large number of cases the rock upon the two sides of the river is different, as when there has been an intrusion of igneous rock through the sedimentary bed rock of the region. The contact of the two rocks is likely to show a zone of weakness, which zone, as it was easily eroded, was searched out and followed by the drainage, thereby resulting in the location of the stream and the formation of the valley or canyon.

A reassuring condition is where the rock upon both sides of the river is the same in kind and position, particularly in the absence of serious distortions or faults. Nevertheless it cannot be too strongly urged that outcroppings afford no safe basis for final conclusions; at best, and with the most intelligent and conservative interpretation, they should be supplemented by core borings and tests of the holes. The geologist should determine the number, location and depth of the borings; and, to facilitate his analysis of results, he should have adequate topographical plans and sections of the site.

The core boring is so called because the cutting is done at the edge of a revolving cylinder, such that a rod or core of rock is preserved inside of the cylinder and can be drawn out with it.

Different Kinds of Drills.—Possessing this feature in common are three distinct types of machines. The Brandt drill is a very satisfactory one, used, however, more often in Europe than in this country. The cutting is done by chilled steel teeth revolving very slowly (5 to 8 r.p.m.) and forced against the rock under very heavy pressure.

In a diamond drill carbons or black diamonds are set in a soft steel bit which is screwed to the end of the revolving cylinder or core barrel. In a bit for a common size ($1\frac{3}{4}$ in. diam.) core eight diamonds are usually set, four projecting slightly outside and four inside. They must be set with just enough projection to give a thickness of cut such that the tool will not bind; and the process of setting them should be undertaken only by an experienced man.

In a shot machine the cutting is done by chilled steel shot poured in as required and ground against the rock by a soft steel cutting bit. Sometimes instead of round shot granular fragments are used. In a diamond or shot machine the pressure on the bit is comparatively small, usually not more than the weight of the revolving barrel; the speed is much higher than with the Brandt drill, being 200 r.p.m. to 400 r.p.m.

In any type of machine a stream of water is supplied through the revolving core barrel to the cutting edge to carry the débris away between the core barrel and the wall of the hole. When the rock is overlaid by earth or loose material it is necessary to sink a casing through such material after the manner described under Wash Borings, clean it out, and introduce the boring tools through it.

It is not the purpose here to discuss all of the advantages and disadvantages of the different methods, nor the various means of overcoming all the troubles encountered in difficult rock.

The most exhaustive treatise imaginable would not render it less necessary to employ a skilled runner to do the work. However, a few of the broader comparisons between diamond and shot machines may be made. A diamond drill will drill holes at any angle of inclination, while with a shot machine the holes must be very nearly vertical in order that the shot may stay at the cutting edge. In ordinary borings for exploring the foundation of a dam this limitation on the shot machine would count for very little, as it is rarely necessary to drill other than vertical holes. When an open seam is encountered the shot or granular fragments are often lost in the seam, in which case the drill runner may then cut teeth on the end of the bit and proceed until again in rock that will retain the shot.

With a diamond drill the maximum diameter of hole is about 15 in., and with a shot machine 30 in. or more. It is obvious that the large sizes are for the purpose of wells, elevator plungers or column foundations; strictly exploration purposes would be as well served by a 2-in. or 3-in. hole as they would by anything larger.

It should be remembered that in a very deep hole it may be necessary to step down in diameter, as with a common percussion drill on shorter holes, and allowance should be made for such a possibility. Both types of drill have very light rigs for use in mountainous country or in places where transportation is a serious question compared with the amount of boring required. However, for a number of holes at a site not too difficult of access a heavier machine run by a steam or gas engine is certainly advisable. There seems to be no marked difference between the two types as regards cost per foot, rate of progress or ability to recover satisfactory cores.

Cost of Core Drilling.—The cost of core drilling varies between much wider limits than almost any other kind of work. For large holes for wells or elevator foundations it may often be the case that the contractor has, from adjacent work, a quite accurate knowledge of the rock which will be encountered; further, the site is usually more convenient and accessible and the conditions are better, so that bids may be received at a reasonable price per foot. For strictly exploration purposes it is usually altogether more satisfactory for everybody concerned to arrange for such work on a basis of cost plus either a percentage or a fixed sum. Such holes are usually shallower, more scattered, at sites more expensive of access, may be in deep or running water requiring rafts, crib work or even suspended staging to support the drill; in fact, every condition is not only more expensive but more uncertain. A price per foot must include all the uncertainties, and nine times in ten it will much more than cover the cost.

In rock not too hard, free from seams and with all conditions exceptionally favorable, the cost per lineal foot might be as low as $1, or even slightly less. Several recent large holes in New York City cost not including plant and overhead charges as follows:

Depth		
2000 ft., 10-in. diam. hole	In Fordham, gneiss and granite conditions rather difficult.	cost per foot $5.56
14 ft., 30-in. diam. hole	In trap-rock concrete containing steel I-beams.	cost per foot $5.60
775 ft., 6-in. diam. hole	In mica schist and quartz	cost per foot $2.65
550 ft., 8-in. diam. hole	In mica schist and quartz	cost per foot $2.80
50 ft., 20-in. diam. hole	In mica schist and quartz, five holes each 50 ft. deep	cost per foot $4.50

On the other hand, the following extremely high cost is due to the most adverse conditions possible.

Conditions: remote Western river, borings made in the bed of a river 160 ft. wide; drill worked on suspended staging for which timbers and guys had to be imported; temperature below freezing and for much of the time below zero; one-half of the river 2 ft. to 4 ft. deep, remainder about 30 ft.; casing had to be put down to bed rock to keep running sand out of hole, though in only one case did casing have to be sunk through any loose material; some running logs and ice took out casing and broke core barrel several times; first machine was too light for the work and another was expressed to the job; some delay waiting for repair parts and on account of distance from repair shop; boiler on shore piped steam to engine operating drill, all holes tested at several depths by hydraulic pressure from hand pump; rock not very hard but tilted up to 15 deg. to 30 deg. from vertical and much distorted so that holes frequently choked.

The drill runner was skilful and industrious, and the contractor did all that was humanly possible to further the work. Eleven borings 12 ft. to 45 ft. in rock, were made for 329 total lineal feet. The hole was 3 in. in diameter and was made with a shot machine in January, February and March, 1913. The erection of staging and making borings cost as follows:

Labor pay roll	$4,074
Freight, express, cartage	1,023
Materials, including carload of coal	2,004
Rental, engine and boiler	392
Plant 1968.43 minus salvage 1654.94	313
Railroad fares of crew	303
Traveling expenses of contractor	458
Provisions and camp outfit	1,159
Insurance, telegraph, and miscellaneous	394
	$10,120

Equals $30.76 per lin. ft.

The above is cited only as showing what the cost might run to under extremely adverse conditions. For fairly accessible location, holes on land or from inexpensive supports in easy water, moderate weather conditions and good management, the cost should not be over $6 or $8 per lin. ft. and might often be much less. The cores, it need hardly be said, should be preserved strictly in the order that they came from the hole. The best way to keep them is in shallow trays with longitudinal partitions, the depth of tray and width between the partitions being just sufficient to include the diameter of the core. On the top of the partitions should be tacked thin strips such that none but the smallest fragments can drop out or be removed, while still permitting examination of the cores. (See Fig. 10.) With each withdrawal of the core barrel the pieces should be laid in their order and the depth marked on the partition; obviously the distance between two depth marks on the partition may be less than the distance indicated by subtracting one depth from the other; the relation will depend upon the percentage of core recovered.

Interpretation of Results.—Even while the first hole at a site is in progress one rarely need be in doubt as to whether the core is from a boulder or from bed rock. Should the core be exactly the same kind of rock as adjacent exposed ledge, and (which is the best possible corroborative indication) should the stratification in both cases show the same angle with the perpendicular, the chances are practically infinite that the core is from bed rock. Even if cores at very moderate depths show perfectly satisfactory rock, it is well to put one or more holes to a great depth as insurance against any possible deep-seated condition that should be known.

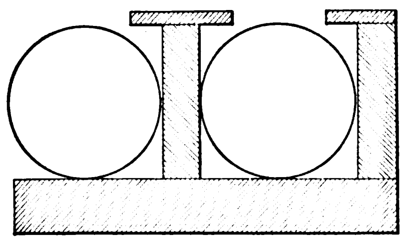


Fig. 10.—Section of tray to hold rock cores.

The vibration of the core barrel breaks the core at the slightest seam, and each piece recovered is in itself perfectly sound. The lengths of core show whether the rock is sound and massive, or seamy, although they are practically of no value as showing whether the seams are tight and harmless or open and objectionable. It will be observed (and in fact is to be expected) that the core will often wedge inside the barrel and itself grind upon the rock; fragments also will break from the core or drop in from the side of the hole and be reduced by grinding to such fineness that they will be washed away and occasionally account for large differences between feet penetrated and feet of core recovered.

It is of course better, though expensive, to draw the core often and avoid as much of this loss as possible.

The character, hardness, stratification, number of seams and depths between withdrawals should all be carefully considered in forming an opinion as to any discrepancy between penetration and length of core. Neither can any one interval or length be considered entirely apart from the adjacent ones or from the total, for the reason that at any withdrawal an appreciable or even considerable length of core may be left behind. Say the core may part or be broken 6 in. above the cutting edge, or drop that much before being gripped, the withdrawal under question will show 6 in. less than the truth and the next one may show the same amount in excess of the penetration. Obviously the total length of core recovered between the surface and any given depth cannot exceed that depth, although between any given depth and the bottom the core might show an excess.

Illustrative Examples.—For purposes of illustration the logs of five of the eleven holes whose cost was given on page 16 are given on page 19.

Elevation of water in river—89 to 90.

Considering the character of the rock and the conditions under which some of the borings were made, the percentages of core recovered may be considered as high. For boring No. 1 the machine was set directly on the rock while the others were made from a suspended bridge or staging.

A streak of somewhat softer rock, occurring no doubt below the bottom of Hole No. 1, is indicated in Hole No. 2, between elevations 45 and 52, at about the same elevation in Hole No. 3; at Hole No. 5, next in line, the soft streak occurs at 56.5 to 63; finally, in Hole No. 4, at 65 to 71. A careful examination of the stratification exposed in the canyon walls showed that such a change of elevation was entirely consistent.

In each case, it will be observed, the boring proceeded through the soft streak with a smoothness that required no frequent withdrawals of the core. In the case of Hole No. 3 the 8.17 ft. of penetration might have yielded more core if it had been withdrawn often, but it shows distinctly different rock from the 8.0 ft. penetration above which yielded 6.16 ft. of core. Similarly in Hole. No. 5 the 6.40 penetration yielding 2.20 ft. of core shows a different kind of rock than the 9.42 ft. of penetration yielding 8.50 ft. of core. The first three measurements in Hole No. 5 simply show slight inaccuracies of measurement, either of the penetration or of the assembled pieces of core. In Hole No. 4 (accepting the measurements as correct) the first ten measurements arrived at the soft streak with a loss of only 0.20 ft. of core; the last three measurements show that more than a foot of core must have been left in the hole when the 4.80 was withdrawn.

Sta. 0, Hole No. 1, Eleva. of top, 89		Sta. 0 + 48, Hole No. 2, Eleva. of top, 61.0		Sta. 1 + 02, Hole No. 3, Eleva. of top, 87	
Distance penetrated	Core recovered	Distance penetrated	Core recovered	Distance penetrated	Core recovered
7.8	6.0	6.6	6.33	9.3	7.75
3.34	3.33	7.1	4.25	5.5	3.25
4.0	3.60	5.0	2.8	8.0	6.16
5.39	5 10	1.5	2.1	5.53	4.7
3.27	3.20	3.0	3.0	5.0	2.58
3.17	3.17	4.45	4.6	8.17	1.5
5.08	5.08	2.7	2.1		
3.65	3.83			41.50	25.94
5.90	5.11	30.35	25.18		
2.40	2.25				
44.00	40.67				

Sta. 1 + 53, Hole No. 4, Eleva. of top, 88		Sta. 1 + 28, Hole No. 5, Eleva. of top, 86.6	
Distance penetrated	Core recovered	Distance penetrated	Core recovered
1.60	1.05	2.0	1.60
0.48	0.35	3.25	3.60
1.42	1.40	0.75	1.00
1.66	2.15	2.27	2.00
1.09	1.00	2.16	2.00
3.08	2.30	9.42	8.50
0.77	1.10	3.75	4.00
2.31	2.75	6.40	2.20
1.75	1.80		
1.64	1.70	30.10	24.9
8.20	4.80		
2.33	2.85		
1.17	2.10		
2.60	2.10		
30.10	27.45		

Testing of Holes by Air or Water Pressure.—Occasionally a seam is encountered of such width that the drill runner can perceive the drop of the cutting tool. The amount of information that an experienced drill runner can get from feeling the drill rod is little short of marvelous, and on important work he should be encouraged to keep very extended notes. After completing the boring it is highly desirable to test the hole at a number of depths by pumping in air, or preferably water, under pressure. This test is to show whether the seams encountered are tight or open, and if carefully conducted it may furnish as much and as valuable information as the core itself.

The process and required apparatus are very simple. Two pipes are used of such diameter that the smaller will go inside the larger, and the larger inside the hole to be tested, and in lengths sufficient to reach nearly to the bottom of the hole. (See Fig. 11.)

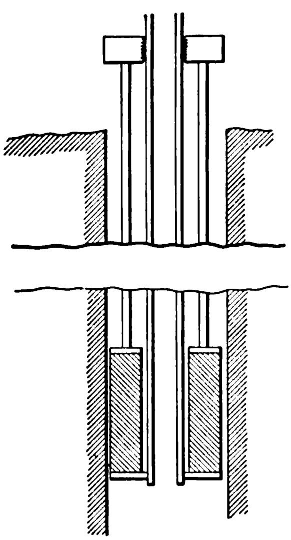


FIG. 11.—Apparatus for testing tightness of seams encountered by a boring in rock.

On the lower end of the smaller pipe is screwed a plate upon which is placed a thick rubber ring or gasket some 2 in. or 3 in. in height, both plate and rubber being just enough smaller than the hole to be readily introduced. On top of the rubber ring rest a washer and the larger pipe. At the upper end a screw thread on the smaller pipe with nut and washer are used to draw up on the smaller pipe, bringing the rubber ring in compression and expanding it against the sides of the hole. Water can then be pumped through the smaller pipe into the section of hole below the gasket, and the pressure recorded by gage. The amount pumped to maintain the pressure is of course a measure of the leakage into the rock. Care should, of course, be taken that the gasket is properly expanded against the sides of the hole so as to make a tight joint. The results may not always be conclusive, for while very slight or no leakage necessarily indicates tight rock, much leakage may or may not indicate open seams, as it is not always possible to get a tight joint or even to know whether it is tight or not. If the surface of the rock at the hole is under water there is no way to detect leakage past the gasket. If it is above water and accessible, however, leakage past the gasket may be detected and measured, although of course such quantity would be subject to loss in seams above the gasket. If the hole is in still water the test might be made with air, and any leakage would show up as bubbles.

In the case of a hole in very seamy or distorted rock, or rock tilted up to approach the perpendicular or where it may be known or suspected that the sides of the hole are full of cavities from the dropping of fragments into the hole while boring, the only way to obtain a tight closure may be to cement the pipe (one pipe in this case) into the hole and make the test after the cement has hardened sufficiently.

The test may be made at as many points in a hole as desirable, each one of course showing the total leakage below the gasket. Let each test be directed toward solving the maximum remaining uncertainty. Thus assume a hole 30 ft. deep test first at 15 ft. Then if large leakage is found move to 22.5 ft. Then if no leakage is observed move to 18.75. Whatever is the result the seam has been located within 3.75 ft. by three tests.

In boring across seams fragments are liable to break off and drop into the hole; this is more liable to occur if the stratification makes but a small angle with the perpendicular hole. In the holes under discussion this was very much in evidence, and many of the tests were inconclusive. Hole No. 1 was found to be absolutely tight as 75 lb. pressure below elevation 72. A leak of 1.64 cu. ft. per minute at practically no pressure was located between elevations 72 and 75, but as at that elevation it was only 7 ft. horizontally to the river, the leakage indicated nothing objectionable. That the soft streak was sound and entirely suitable for a foundation was indicated by a test in Hole No. 5 at elevation 70 where the hole was found absolutely tight at 75 lb. pressure.

The above-described procedure for testing holes is entirely suitable and sufficient for the purpose of a preliminary examination, *i.e.*, to determine feasibility, design and estimate of cost.

When, however, the foundation has been unwatered and the bed rock has been stripped of all loose overlying material, both opportunity and time are afforded for a more exhaustive and detailed examination. Such further examination, possibly involving additional holes, might be desirable in connection with the question of necessity for or extent of grouting the foundation.

An apparatus similar to the above, but employing two gaskets, may be used. One application of the pressure tests only that portion of the hole between the two gaskets, which may be set at any desired distance apart. The following description and illustration of such an apparatus is taken from an article appearing in *Engineering Record*, July 4, 1908, which described methods of testing the foundation of the Olive Bridge dam.

Testing Borings at Olive Bridge Dam.—The apparatus consists essentially of a double pipe long enough to reach to the bottom of the test hole and provided at the upper end with a clamp, a pressure connection and a pressure gage, and at the lower end with a set of expansion washers which serve to provide adjustable packing or bushing between the pipe and the bore hole. (See Fig. 12.)

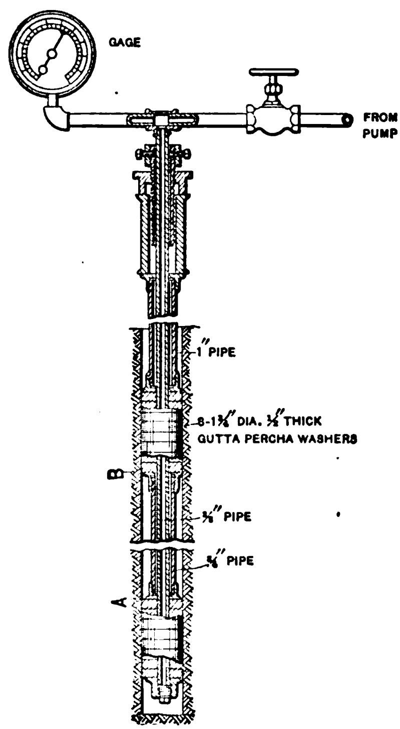
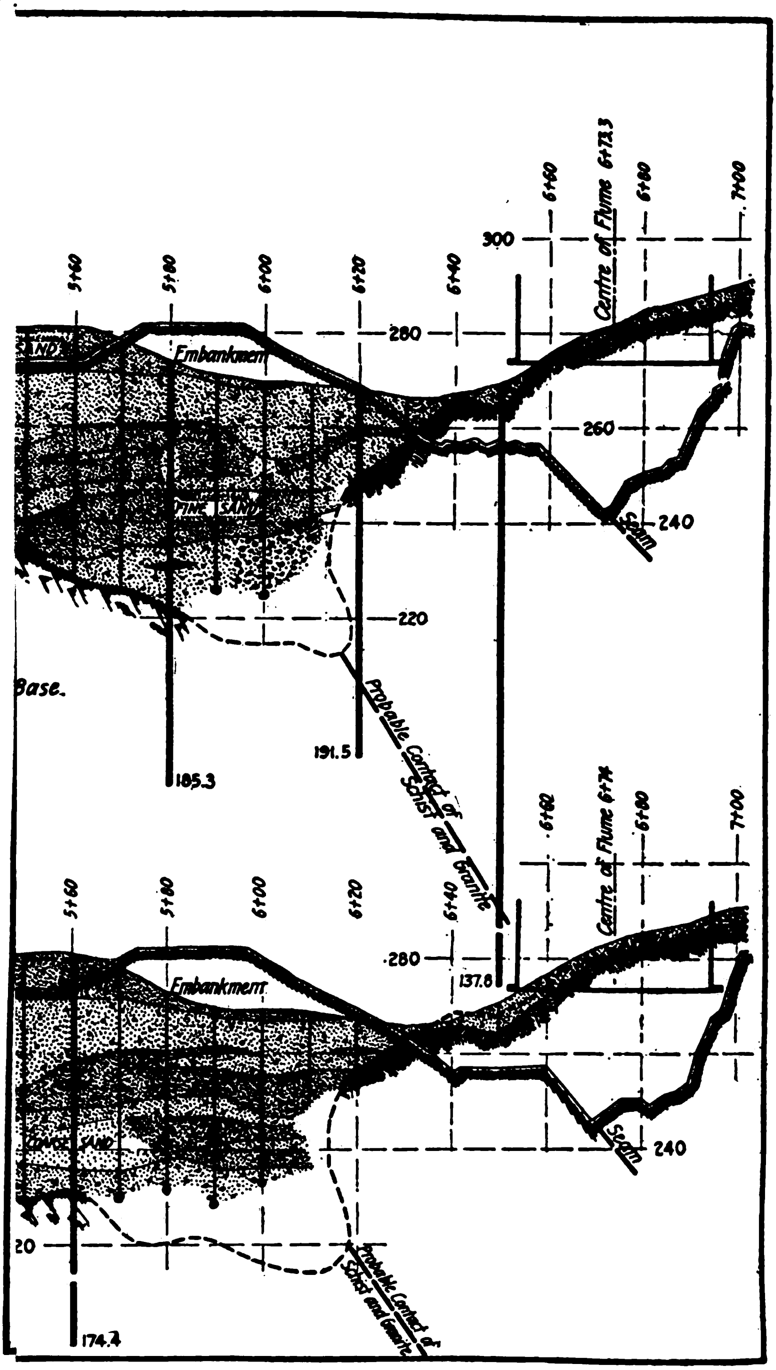


FIG. 12.—Apparatus for testing holes in foundation of Olive Bridge dam.

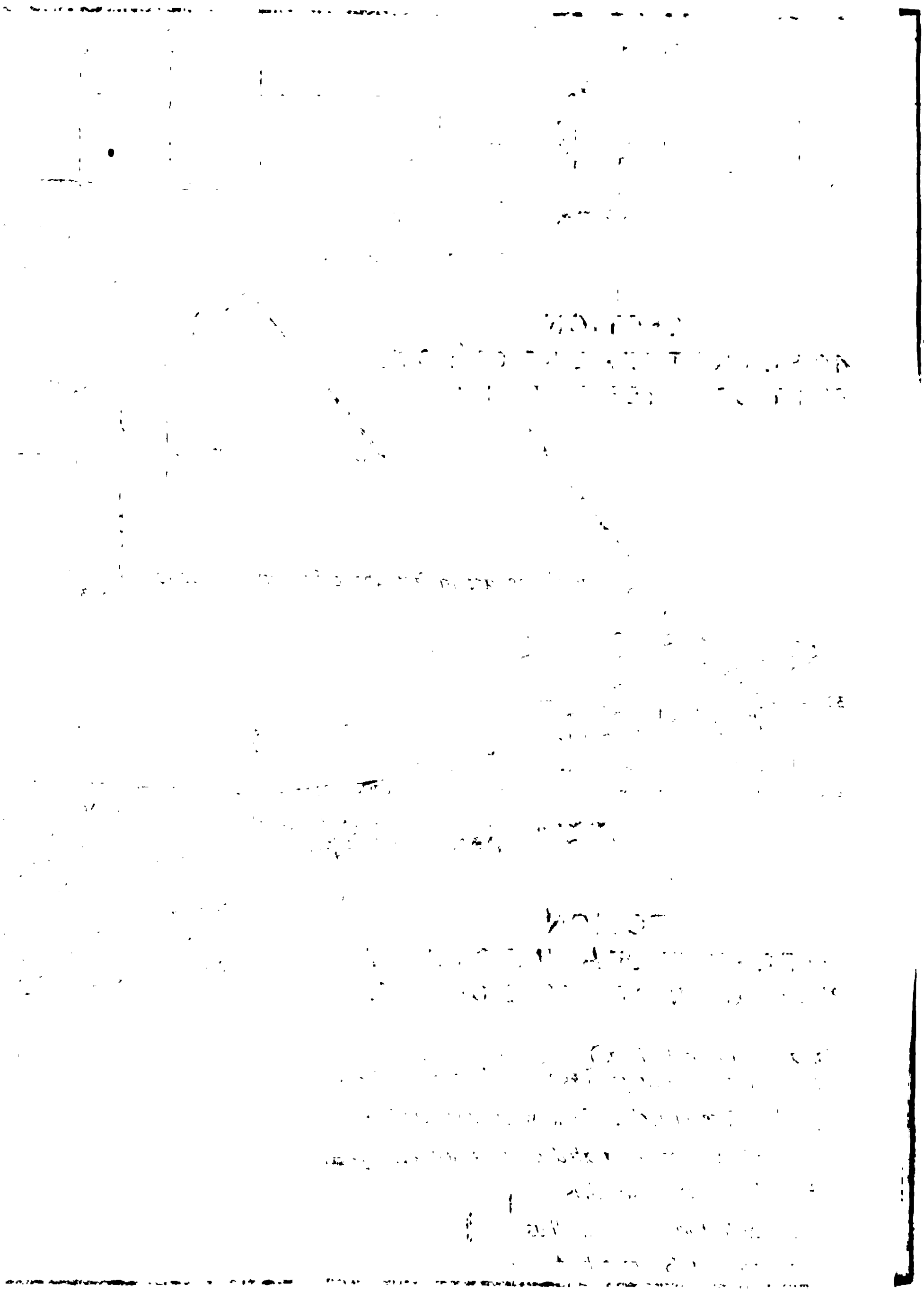
The inside pipe ⅜ in. in diameter terminates at the upper end with a tee having the pressure gage on one branch and the pump valve on the other branch. Just below the tee the pipe has a smooth fit in a short vertical sleeve threaded outside and screwed into an adjustable collar. Set screws through the sleeves provide for fixing the pipe relative to the sleeve, and the screwing of the fixed outside collar raises or lowers the sleeve and the attached inner pipe. The lower edge of the outside collar takes indirect bearing on top of the 1-in. pipe which encloses the ⅜-in. pressure pipe and extends to within about 20 in. of the bottom of the latter.

The lower end of the pressure pipe is closed with a plug and has a shoulder or flange engaging a set of ¼-in. pure-rubber washers that have a diameter just small enough to clear the diamond drill holes.

The washers enclose the pressure pipe and have on their upper surface a steel flange receiving the lower end of a perforated ¾-in. pipe 12 in. long, provided at the upper end with a flange and washer similar to those described for the lower end. The upper surface



on gained from borings. (*Following Fig.* 13.)



of the second set of washers engages a flange on the lower end of the 1-in. casing pipe.

The full length of the test hole below the bottom of the pipe can be tested at one operation by removing the plug in the end of the pressure pipe. When testing near the surface of rock containing nearly horizontal seams it may be well to accompany the test with careful observations of the elevation of the adjacent rock surface for it is entirely conceivable that the pressure might open a seam and raise the rock.

Wachusett Dam.—The Wachusett dam 1900–1906 is up to date the one whose site was most thoroughly explored and studied preliminary to construction. Nearly 900 wash drill borings and about thirty-five diamond drill borings were made, and the results published, with the specifications, for the information of bidders. A plan, showing the location of all the borings, and four out of the fourteen sections across the valley showing the nature of material and depths to rock, are given herewith. The interpretation of the borings was entirely substantiated later during the excavation. (See Figs. 13–14–15.)

CHAPTER II

TEMPORARY WORKS FOR STREAM DIVERSION

It is important that the temporary works for control of the stream during the early period of construction should be so designed and constructed as to afford a maximum of safety to the work and to those engaged on it, with a minimum of pumping and interruption. It must be borne in mind that the problem almost invariably involves more than one handling, or one diversion, or one channel for the water. In other words, when the masonry has progressed to a certain point, the manner or channel of diversion must be changed to permit further progress. In fact several changes either laterally, or in elevation, or both, may be required. It must here be stated most emphatically that a sound plan for stream diversion must clearly contemplate and amply provide not only for every step that must be taken but also for the order of procedure.

This does not mean that the works should necessarily be planned of a capacity to exceed the largest known flood, nor that chances as to capacity or time available should never be taken; but it does mean that the engineer should not allow himself to be drawn into any situation where he does not know what to do next. Even if a time comes when he can do nothing but sit on the bank and watch the flood roll by, he must have clearly in mind the thing to be done when the river has subsided to a working stage. To be caught with a plan that is in the slightest degree indefinite or incoherent is almost certain to be expensive as well as foolish. This does not mean that the plan may not be modified, or even changed entirely, if found necessary as the work progresses. The balancing of capacity of channel against probable stream flow, and the cost of temporary works against the cost which would result from a flooding of the work, is the gist of the entire problem. With increasing depth of pit it becomes increasingly desirable that it be not flooded, particularly if the loose material is such that flooding the pit is liable to result in its being filled by the loose material. With increasing flow it rapidly becomes expensive and then impossible to handle the flow without flooding the pit.

One alternative involves cost of temporary works up to a capacity which is so rarely exceeded that the chance may be taken. The other involves an estimate of the number of times the work may be flooded, the cost of emptying the pit of water and loose material, cost of repairs to temporary works and the value of the lost time. The net remaining working periods should be such that substantial progress may be made between floods.

In fact, it is the satisfactory combination and balancing of all the conditions which determine not only the design of the temporary works, but indeed whether the project itself is feasible or possible.

Every project of magnitude must differ more or less from every other project, and each must be considered as a problem by itself. Nevertheless, it may be profitable to discuss certain types of actual diversion works to see what general principles may be laid down.

Various Types of Diversion Works.—The condition determining what will here be called the *type* of diversion works, is the available room, *i.e.*, the width of the valley at or about water level. Three distinct types may be defined as follows:

Type 1.—A channel so wide that it may be encroached upon by the necessary working area, the stream meanwhile occupying the other portion.

Type 2.—A narrow valley where an artificial channel must be provided, usually upon one side.

Type 3.—A valley or canyon so narrow that reasonable access to the bottom requires that the stream be removed from it entirely by means of a tunnel.

Types 1 and 3 are fairly well defined, and different examples may vary only in the amount of water handled; Type 2 on the contrary covers a much wider range; a greater variety of conditions produces such variation in design that rarely may two examples be at all comparable. Conditions other than width of valley are the following: maximum amount of water to be handled, depth and nature of loose material between water level and foundation, character and amount of silt carried by stream as affecting the tightness and efficiency of an earth dam; cost of power for pumping purposes; whether or not derricks or cableways may be readily placed so as to handle any proposed cribs or flumes; manner in which materials for permanent dam are to be assembled and placed; location, size and character of permanent outlets through dam; whether maximum floods will be expected and allowed to pass over the work; whether running ice or logs may obstruct the channel; probable length of time required; availability or cost of materials; possible uses of water for permanent construction or by other enterprises or for navigation.

Some conditions will be favorable and some unfavorable. In fact certain unfavorable conditions may necessarily mean that certain others are favorable. Thus a very swift current would indicate a fall that might readily be made available for cheap power for construction purposes, and probably also that there is not much loose material to handle. A very wide stream might indicate that it was correspondingly shallow. Large and violent floods might indicate a long season of minimum flow, etc. Let us consider first temporary works of Type 1 of which McCall's Ferry and Keokuk are conspicuous examples.

The indicated procedure is to enclose approximately one-half of the site in a cofferdam; diverting the stream through the other half. In the half thus first enclosed and built are constructed the permanent outlets from the reservoir. If the volume of water to be handled is in excess of the capacity of the permanent outlets, additional temporary openings may be left through the dam after securing the foundation and building up to a certain elevation. (See Plate II, Fig. D.) These temporary openings must be so made that they can be filled with masonry later. After the first half has been thus built, the second half of the site is enclosed and built in a similar manner, while diverting the water through the openings in the first half; this, of course, necessitates the removal of a sufficient amount of the first cofferdam.

The second half may or may not contain additional temporary openings. The total capacity of the openings through the dam is to be determined from a study of the hydrograph of the stream, and a consideration of the length of uninterrupted working period which is necessary or desirable. The number of openings depend upon the scheme for finally closing them as well as upon the total desired capacity.

To digress for a moment from the consideration of temporary works purely as such, it may be shown here how the question of transportation of materials may be intimately connected with that of stream diversions, and also how certain works may serve the two purposes.

In the typical situation above outlined, the dam is usually a low and hence also a narrow one, the width of the stream is too great to be practicably spanned by cableways and the bottom of the pit is at no great depth. Economy in the matter of a cofferdam usually requires that it enclose the site in what might be called a hairpin shape, *i.e.*, paralleling the main dam at short distances up- and downstream from it. Under such conditions the materials may be brought to the works via tracks running on the top of one or both of the cofferdams.

The situation may present minor variations which do not, however, constitute a difference nor involve a departure in type of diversion works. Thus the stream may be divided by an island into two channels, such that the division of the two sections would naturally be upon the island, and possibly requiring no cofferdam around the ends. It might be that the length of the island in connection with the slope of the stream would eliminate the necessity for any cofferdam upon the downstream side of the work.

A variation which may involve no essential difference is where the first section of the dam may be built without resorting to cofferdams. Such a situation would arise where a sufficient width of the bottom of the valley was not occupied by the stream, but by material of such character and in such quantity that the leakage through it could be readily handled by pumps. Then the cofferdam for the protection of the second half would probably involve sheet piling for a short distance from the completed end of the first section to the river bank, and a continuation across the river with whatever type of cofferdam was best adapted to the situation.

The diversion of the stream through the first half of the dam might require the excavation of a channel through the bench of loose material, or else that the temporary dams be such as to raise the water, or some combination of the two.

Conditions indicating such a scheme of diversion may merge by any number of gradations into others which would properly call for what is here termed Type 2 of diversion works.

The chief difference would be that the width of the valley is considerably less; in fact, narrow to the point that an artificial channel must be constructed along one side. The matter of desirable working area on the foundation, and necessary width for the channel, may require considerable excavation from the side hill and also that the channel be crowded into the side hill as far as possible.

Cofferdams below the channel inlet and above the outlet, respectively, turn the stream into the channel and prevent it from backing into the pit from below.

To keep down the width of the channel it may be given a steeper grade than the river, requiring of course that the water above the upper cofferdam be raised above the normal. Where the channel crosses the dam the foundation may be previously prepared, and the permanent masonry put in up to the level of the bottom of the channel. The channel may at this point have permanent masonry sides, with dividing piers to facilitate final closure; may be arched so that masonry construction may proceed above it, and have, in fact, whatever best meets the scheme of subsequent procedure. The river side of such a channel may be formed of the material excavated from the channel; depending upon the character of the material it may be necessary to put in a core of sheet piling to reduce the leakage; also to protect the channel from erosion, as by means of rip rap. A minimum width of channel would involve sides, and probably bottom, of timber or masonry, which at once conduce to and withstand high velocities. This may be particularly necessary for that portion of the channel opposite the pit, where an embankment of loose material would encroach to an objectionable or prohibitive degree upon the working area of the pit. (See Plate IV, Figs. A and B, also Fig. 17.) We may thus pass by degrees to the extreme case of this type, *i.e.*, to a case in which the entire artificial channel is a flume. (See Plate IV, Fig. C, also Fig. 16.) Such a flume would probably be constructed of timber, with the possible exception of the section through the dam where the plan for subsequent diversion or final closure might indicate that it should be of masonry. At the Cross river dam, with a small flow to divert, the water was carried across the dam in pipes which were built into the masonry and subsequently filled with concrete. (See Plate VI, Fig. D.)

Other things being equal the narrow channel will result in deeper water, requiring a higher cofferdam. This fact should be considered not only in the design of the cofferdam but in its effect upon leakage and pumping. If the capacity of the channel or flume is to be probably exceeded it will be well to provide for the filling of the pit at some other point and in some other manner than over the upper cofferdam, such as through a gate in the channel or flume. If the pit is full when the upper cofferdam is topped, the danger of washing out the cofferdam and of filling the pit with loose material is probably minimized. If it is possible for a wooden flume to be submerged its flotation should be estimated, and it should be adequately anchored to its foundation.

Type 3, in the United States, is met with in the rocky canyons

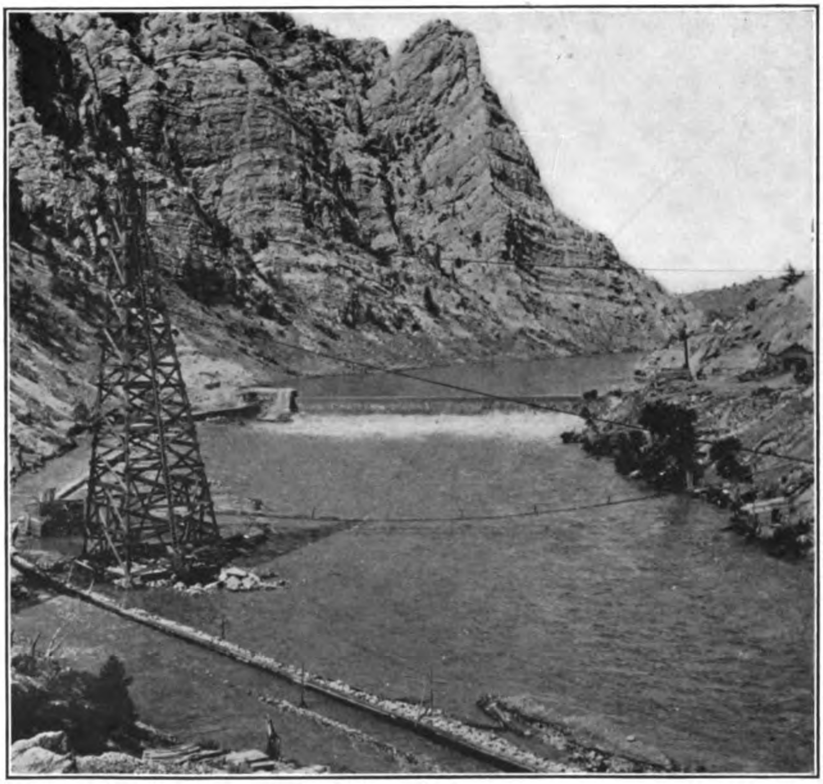


FIG. A.—Shoshone dam, showing one of the cableway towers, also temporary diversion dam and channel in time of high water.

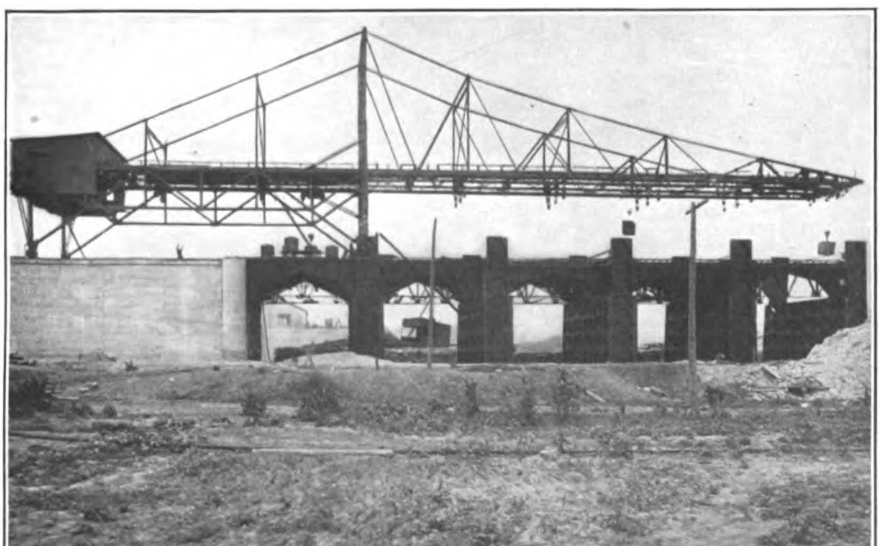


FIG. B.—Keokuk dam, showing traveler for placing steel forms and concrete.

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PLATE II

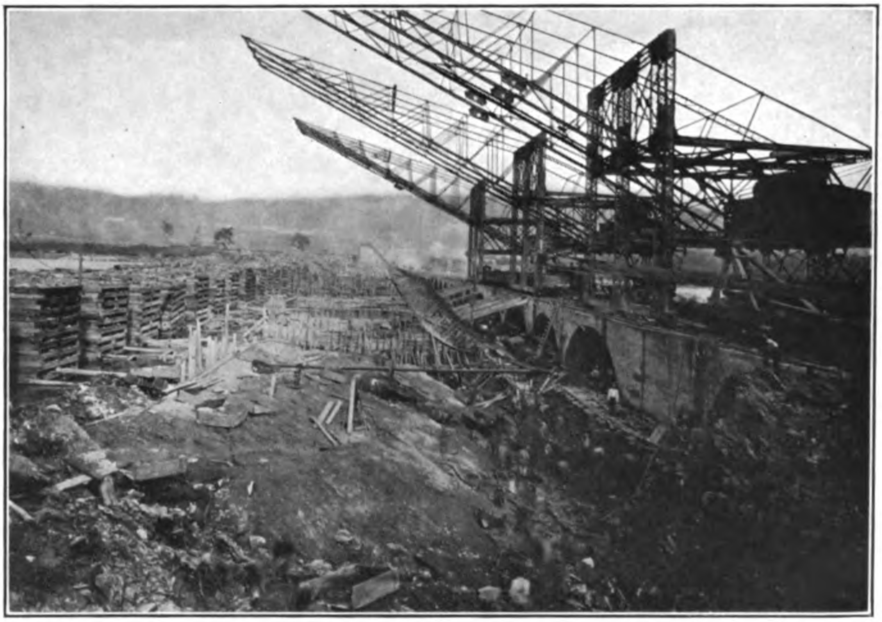


FIG. C.—McCalls Ferry dam, showing cantilever cranes for handling materials.

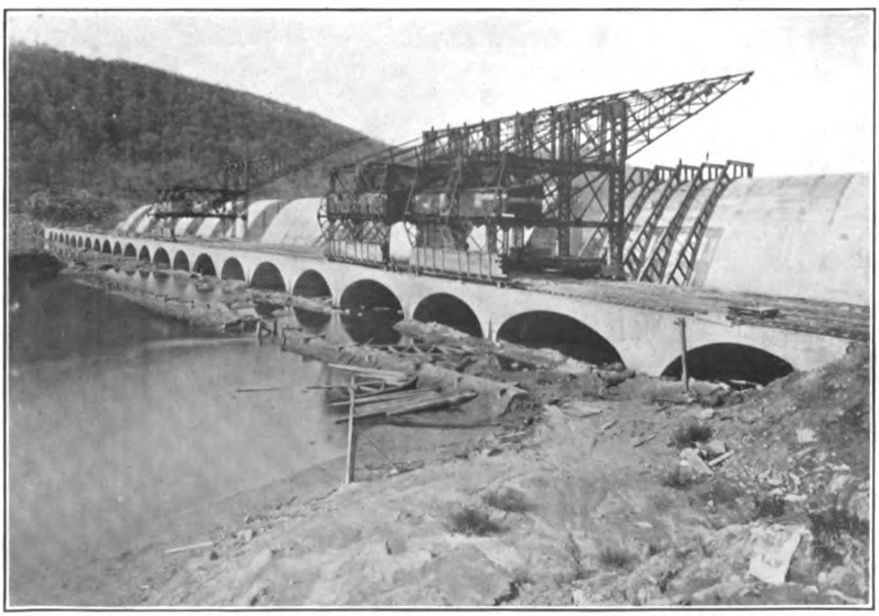


FIG. D.—McCalls Ferry dam, showing concrete construction bridge and cantilever cranes.

of the West, such as for the Roosevelt, Pathfinder, Shoshone, and Arrowrock dams of the U. S. Reclamation Service. (See Plate III, Figs. A and C.) Eastern dams, being in comparatively spacious valleys instead of abrupt canyons, have used Types 1 and 2 with a great number of variations in detail to suit particular conditions.

Wherever this scheme is employed it is probable that the tunnel is a feature of the completed works as a reservoir outlet, and should not be charged entirely to stream diversion. Considered purely as a reservoir outlet such a tunnel should be viewed very critically for two reasons.

First.—It seems to require or lead to, or at least so far to have been accompanied by, the installation of high-pressure gates. Such gates and their operating mechanism are complicated and costly and are not certain to give entire satisfaction in operation.

Second.—The leakage by or around such gates, although it may still be insignificant in amount, will be much greater than the leakage in connection with gates installed at or near the upstream end of an outlet through the body of the dam. The reason is that while the path of the leakage through masonry may be as long in one case as in the other, the areas over which the leakage is applicable or effective are vastly different.

In the case of gates on the upstream face of the dam, only a very limited area immediately around the gate can pass leakage by a short path through the masonry. The path rapidly becomes longer as area away from the gate is considered. In any event, the masonry, if not stone with thin joints and hence impervious, is laid in the open under conditions conducive to high-class work.

In the case of gates in a tunnel, leakage is applicable and effective over the entire inside area of the tunnel upstream from the gates. Passing through 1 ft. or 2 ft. of concrete lining the water reaches comparatively open channels, either through rock which is very likely to have been affected by blasting during the tunnel excavation, or else between the lining and the rock. Indeed, it would be a rare job of tunnel lining if no channels were there. Plenty of weep holes should be left in the tunnel below the gates in order that the leakage returning to the tunnel will exert no inward pressure on the lining.

Again, unless the rock is exceptionally favorable, or the tunnel deeply buried (*i.e.*, kept some distance away from the foundation of the dam) the process of excavating the tunnel may open seams in the rock between it and the dam that may later become a source of leakage.

Considered solely as a temporary diversion channel it must probably be said of such a tunnel that the capacity is limited, and that it is expensive. Upon most alternate schemes much work might be done previous to the installation of the plant necessary for the rapid and economical construction of the tunnel.

The tunnel must be driven, probably lined, and certainly provided with a screen at the entrance, before much if anything can be done toward diverting the stream. A screen is specified because a tunnel without one might possibly be blocked by drift in the tunnel, which would result in very serious if not fatal expense and delay. Though the screen be readily accessible it will be difficult to keep clear, in proportion to the amount of drift. In fact, should lumbering operations be going on along the river, it might be a practical impossibility to do so.

While conditions may indicate unmistakably that a tunnel is the proper solution, earnest consideration should be given to the alternative of an open channel crowded into the side slope or wall far enough to clear that area of foundation lying below the elevation of the bottom of the channel. This would involve the excavation of a notch in the rock along one side of the river at about water level, one side of the notch being the floor and the other the vertical uphill side of the open channel. Then erect a masonry wall for the other side.

The notch and wall may be amply long enough to connect with feasible temporary dams and still not be as long as a tunnel. There might be many more cu. yd. of excavation, but that part of it across the dam-site would practically all be necessary in any event in preparation of the foundation. Even if the net amount is much greater it would be open cut and much cheaper than tunnel cut. Any possible disturbance of the rock under the foundation of the dam would be entirely eliminated. Any temporary or permanent gate frames or guides could be more readily set and be more readily accessible. In the much less likely event of an obstruction, it would be a simple and easy matter to clear an open channel. Whenever desirable the channel could easily be closed, reduced in size to any dimension for permanent outlet, arched over so that work could proceed above it until later closure, or permanently closed and the stream diverted to a similar higher channel.

Actual Examples with Cost.—Type 1 as illustrated by the scheme of diversion employed at the McCalls Ferry dam across the Susquehanna River.

At this point the river is ½ mile wide, divided into two channels by a low island, about in the middle, and has 26,766 square miles of watershed. The flow of the river at times is as low as 3000 c.f.s., and the maximum flood is nearly 700,000 c.f.s. A study of the stream records indicated that it would be wise to provide for handling 50,000 c.f.s. without interruption to the work. The dam is about 50 ft. high and has an overflow section for practically its entire length, *i.e.*, 2350 ft.

Cofferdams diverted the entire flow into one of the channels, while in the one unwatered were built the powerhouse foundations and one-half of the dam. While the bottom was all prepared and the foundation masonry all put in up to a certain elevation, the dam was built in alternate sections of 40 ft., leaving an equal number of 40 ft. openings through which to divert the stream later. The work was handled by cranes traveling on a construction bridge just below the dam. In order that no delay might be occasioned by possible floods carrying away the bridge it was built of concrete in the most substantial manner. (See Plate II, Figs. C and D.)

This bridge has been criticised because it was so expensive (about $100,000) compared with the figure at which a steel bridge could have been erected. However, the size of the floods encountered during construction amply demonstrated that some kind of a bridge was the best solution of the problem.

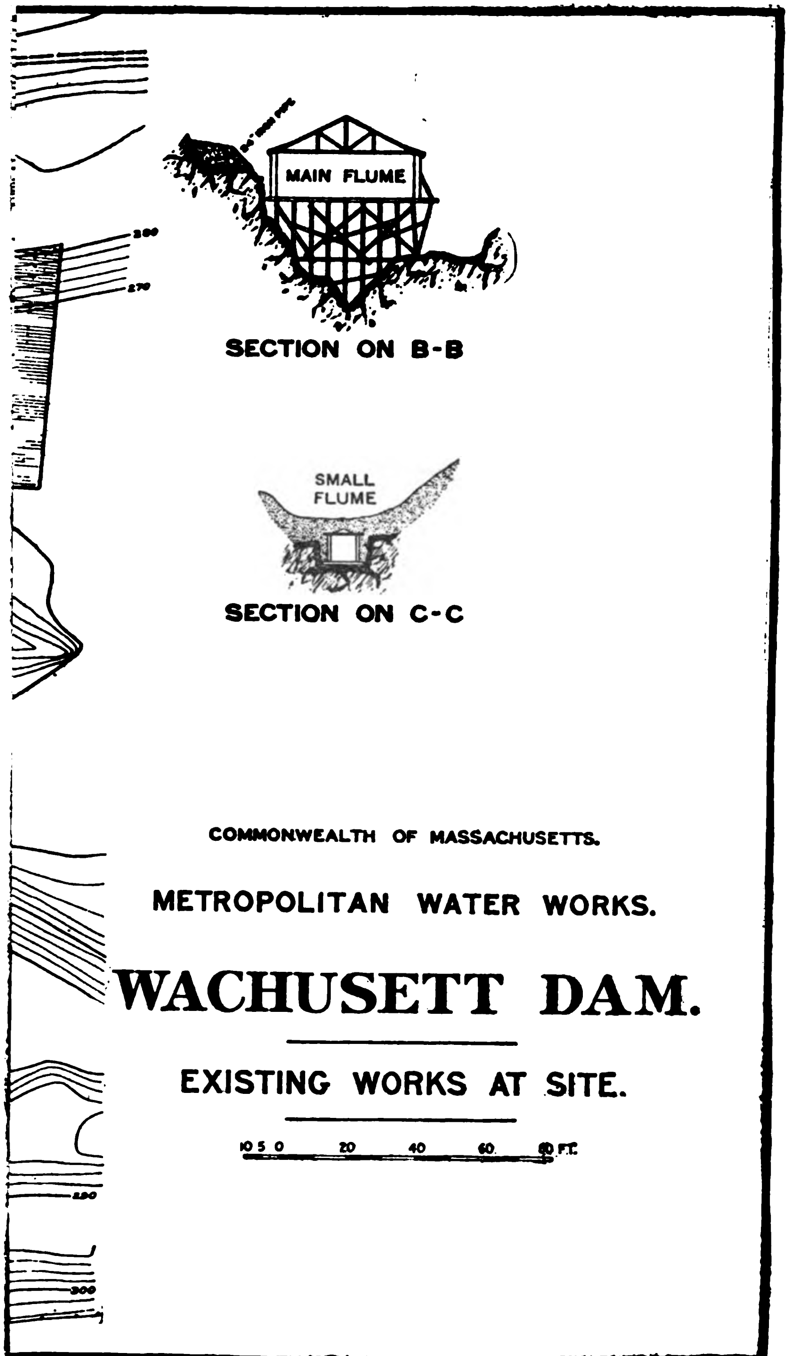
The cofferdam was composed of cribs 16 ft. wide by a length up- and downstream one and one-half times the depth of the water. The cribs were spaced 10 ft. apart in the clear and after being weighted the openings were closed by 12 in. square stop timbers. Along the upstream face was placed a double line of vertical 2-in. sheeting, and puddling material above that. The cofferdam thus constructed was remarkably tight, for 1000 ft. of it under 18 ft. head leaked less than 1 c.f.s.

When the first half had been built as above described, *i.e.*, alternately 40 ft. brought up to the top and 40 ft. left open above the foundation, a precisely similar process was gone through with on the second half, diverting the river through the openings left in the first half. When the entire dam was brought to the stage of alternate 40 ft. sections completed and open, the openings were brought up 5 ft. at a time. To accomplish this each opening was provided with a very ingenious shutter, the water was shut off, and 5 ft. of masonry was placed and allowed to harden until the water could be passed over it. Entire completion simply involved repetition of the process. Several considerable floods were passed during the construction period with but slight damage and only the unavoidable loss of time.

In the dam recently completed for the Mississippi River Power Company at Keokuk, Iowa, many of the conditions and methods were practically identical to the foregoing. The foundation was very favorable, being a solid blue limestone, which was found at about the same elevation all over the work and which required apparently very little preparatory excavation. It was necessary to pass large quantities of water 20,000 c.f.s. to 300,000 c.f.s. during construction.

The cofferdams were constructed of cribs exactly the same except for height, as the McCalls Ferry, and like the McCalls Ferry cofferdams, were remarkably tight. The dam itself was about four-fifths the height and twice as long. For the river section the construction bridge, in this case part of the permanent structure, was built first by a cantilever crane working from the structure itself and overhanging four bays in advance. (See Plate II, Fig. B.) Each bay consisted of a 6-ft. pier and a 30-ft. clear opening, the springing line of the arches being 11 ft. above the crest of the spillway sections between. As at McCall's Ferry the openings were left low to be later raised 5 ft. at a time by blocking off with a similar shutter handle by a derrick from the bridge.

A variation of Type 1 was the scheme employed at Spier Falls dam (1900–1903). No diversion seems to have been required for the construction of the first portion of the structure which was built upon a bench 20 ft. or so above the river level. Through this first portion were left four openings 7 ft. by 10 ft. with floor at elevation 30 designed to be closed later by wooden shutters and filled with masonry. There was also left a gap 90 ft. wide at elevation 40 in order to pass the larger floods. When it came to unwatering the main portion of the excavation immense crib cofferdams were required, the main upper one being 600 ft. long. Its average section was 150 ft. bottom width, 60 ft. high and 25 ft. top width, with a maximum section 250 ft. bottom width, 90 ft. high and 80 ft. top width. This was constructed cob-house fashion of logs drift-bolted together and filled with rock and gravel. Though portions of it were twice washed out by floods at critical times, the finally completed



(*Facing page 32.*)

cofferdam was entirely sufficient and quite tight. During the entire construction period the stream flow was never less than 1000 c.f.s., and floods of 40,000 c.f.s. to 50,000 c.f.s. occurred.

The total amount of excavation was some 270,000 cu. yd., the maximum depth being about 80 ft. About 4,000,000 gal. of water per twenty-four hours were pumped from the pit.

A question which naturally arises in connection with a study of the stream diversion at Spier Falls is why were the four 7 × 10 ft. openings put so high (with floor at elevation 30)? True the first section was built on a bench 30 ft. above the river, but it seems possible that they might have been placed near the river end of the section, and that an inlet and outlet channel of not excessive length might have been excavated to carry the water. If they could have been so placed, at or very near river level, that 30 ft. saved in elevation would doubtless have saved more than half of the material and labor involved in the crib work in the upper cofferdam. It would also have reduced somewhat the amount of leakage to be pumped and the danger of washouts by floods. Published accounts of the work indicate no reasons why this could not have been done, though quite possibly some existed.

Unfortunately costs are not available in connection with any of the foregoing examples.

The method employed at the Wachusett Dam, Type 2, was to carry at all times the entire flow of the river across the work through a flume at one side. It was about 250 ft. wide between rock at river level, and the loose material to be removed was gravel with a maximum depth of about 65 ft.; maximum amount of pumping was about 6,000,000 gal. per twenty-four hours at heads of 50 ft. to 70 ft.; one flume 700 ft. long, 40 ft. wide, by 15 ft. to 17 ft. high, supported across the foundations on piles and posts; grade 1 in 1000; upper temporary dam of earth with a core of sheet piling not, however, to rock; lower temporary dam of earth; another flume 7 ft. × 7 ft. × 500 ft. long carried water to the Wachusett aqueduct but was not required as far as handling the water was concerned; both flumes had head works to control the flow. (See Fig. 16, also Plate IV, Fig. C.) The cost of the temporary works was about $120,000.

The water above the upper temporary dam was held at an elevation about 20 ft. above the natural river level in order that water might be diverted during the construction period for two purposes; first through the small flume, above mentioned, for the use of the Metropolitan water district; second on the other side of the river through ¾-mile of 24-in. cast-iron pipe for the use of a cotton mill. Of course, holding the water up for these diversions added to the expense of the temporary works, complicated the operation, increased the leakage into the pit and consequently the pumping. But for these diversions no head works would have been necessary for the large flume, and the flow could have been entirely uncontrolled.

A flood, of 9700 c.f.s., the maximum known on the river, was carried by the flume a few weeks after completion. One attendant circumstance is worthy of mention. During the height of the flood, the water lost so much head in acquiring velocity after passing the head gates that its surface was some 5 ft. lower than the surface of the water outside the lower end of the flume. At this time the lower temporary dam had not been built and the water backed up to the upper temporary dam at an elevation sufficient actually to float a large length of the flume. The amount that it was lifted was afterward found to have been at least 18 in.

The large flume was in commission about a year and a half before the first stone was laid, and nearly three years altogether. It was abandoned at a low stage of the river and the flow turned through four 48 in. outlet pipes through the dam. A subsequent flood just topped the dam by a few inches at about 40 ft. higher elevation.

The scheme of handling the water was, of course, determined by the depth and amount of excavation, the expense and lost time if the pit had been filled, the length of time that the pit was required to be open and the fact that a flume within reason could be expected to carry the maximum flood.

The conditions at the new Croton dam were similar but much more pronounced. A very similar scheme was also adopted, namely, that of maintaining on one side of the valley a channel designed to carry the entire flow. The pit was larger and deeper and open for a much longer time. On the north side of the valley was excavated a channel, principally in rock, 1100 ft. long and 125 ft. wide, with its bottom 5 ft. above the original river bed. The natural rock formed the north side of the channel; the south side, next to the pit, was formed of a masonry wall extending 300 ft. each way from the center line of the dam. (See Plate IV, Figs. A and B.) also Fig. 17. This wall was about 35 ft. high, 13 ft. thick at the base and 3 ft. at the top; except where crossing the foundation trench it was backed by an earth embankment. The inlet and outlet was formed by curved, earth embankment wing walls extending from the ends of the masonry wall, and with tops 10 ft. wide and 5 ft. higher in elevation. Through the center of these embankments was a core of 3 in. tongued and grooved sheet piling, and the toe of their slopes on the channel side was protected from

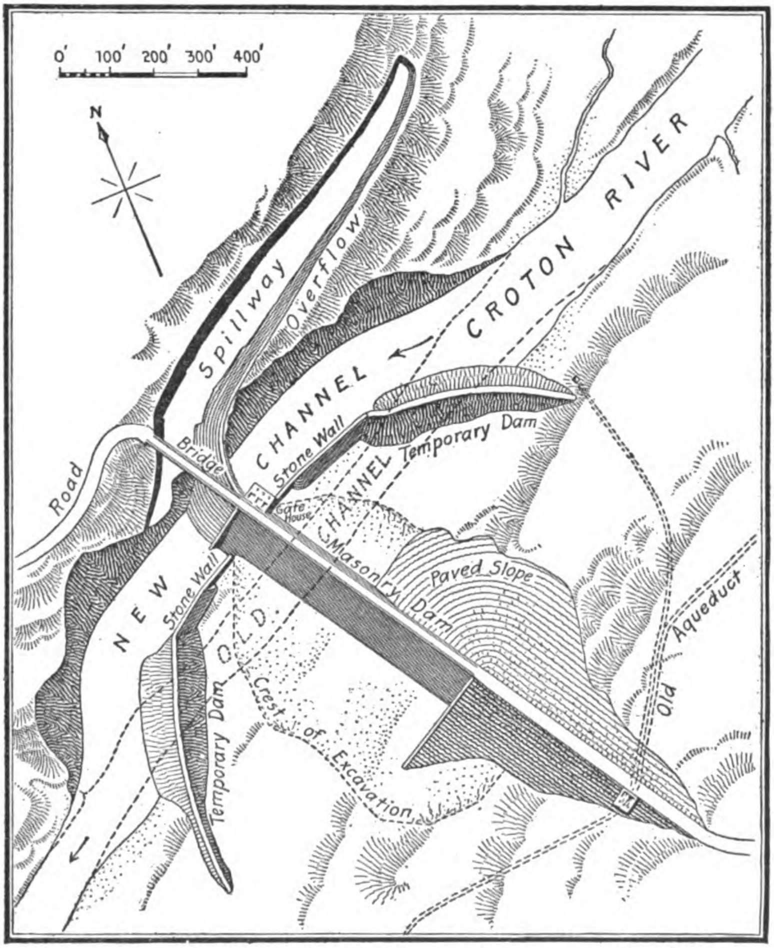


FIG. 17.—New Croton dam showing temporary stream diversion.

erosion by crib work faced with sheeting which extended down several feet below the bottom of the channel. The entire length of masonry and earth wall on the south side of the channel was 1600 ft.

Above the dam site are 360 sq. miles of watershed, but the effect of several storage reservoirs, and the daily draft of 300,000,000 gal. to 375,000,000 gal. to supply the City of New York, was such that for much of the time this channel carried but from 1 ft. to 3 ft. of water. During freshets it rose to a depth of 15 ft., to 19 ft. and upon two occasions to such a height that it poured over the masonry wall into the pit, though luckily at a time when the work was so far advanced that little damage or delay resulted.

The cost of the entire works for diversion and handling of the river during the construction of the dam was approximately as follows:

Excavating new channel	$120,000
Diverting the river	118,000
Pumping	80,000
Damage due to floods	74,000
Building and afterward closing relief openings through dam	30,000
	$422,000

Another similar situation exists at the Elephant Butte dam now under construction in New Mexico by the U. S. Reclamation Service. (See Plate III, Fig. D, and Plate IV, Fig. D, also Fig. 55.) At this point the Rio Grande is about 500 ft. wide, much of the bed rock is 70 ft. below river level and the material to be excavated is loose sand. The run-off of the river varies between 50,000 acre ft. and 2,000,000 acre ft. annually, with a mean of about 860,000 acre ft. High water may occur at almost any season of the year. The depth of the pit, the quantity of loose material to be excavated, 350,000 cu. yd., and the certainty that even a slight overflow of the work would result in filling the pit with the loose material, determined the scheme of diversion.

A flume 1050 ft. long; floor and land side of concrete, land side batter ¼ to 1, floor 46½ ft. wide; river side of timber with batter of ½ to 1, was built along one side of the river. The capacity of the flume is 20,000 second feet, or sufficient to carry all floods except such as might occur once in eighteen or twenty years. Where the flume crosses the dam, the foundation was prepared and the permanent masonry put in up to the floor of the flume, a maximum depth of 58 ft.

The Eleventh Annual Report of the U. S. Reclamation Service gives the following costs of the diversion works to June 30, 1912, at which time the final closures of the cofferdams had not been made; also there remained some work to be done on the flume at the crossing of the dam.

Preliminary for excavation and construction of cofferdams:	
Flume intake and outlet	$ 32,549.82
Excavation and flume, construction of flume, flume intake and outlet, and cofferdams	157,903.32
Construction of flume, woodwork	3,562.30
Concreting flume sections	21,721.57
Concreting dam sections of flume	4,745.08
Total	$220,482.09

For the construction of the Croton Falls dam the diversion channel started from a point 600 ft. above the dam, the total length was 1400 ft., and the capacity was 1000 million gallons per twenty-four hours or about 1550 c.f.s.

For a length of 800 ft., the channel was a timber flume 24 ft. wide × 8 ft. 2 in. deep, supported across the excavation. (See Plate VII, Figs. C and D.) The contract price for the diversion works was $90,000.

At the Cross River (Plate VI, Fig. D) and Kensico dams (Plate X, Figs. A and C) the amount of water to be handled must have been comparatively insignificant, as in the former case it was carried across the masonry in a couple of pipes, and in the latter case through a small flume. The Kensico dam is being built in order to store water from the Catskill Aqueduct. The watershed tributary to the reservoir is insignificant.

At the Cataract dam, N. S. W., built 1902 to 1907, the water seems to have been handled through 48-in. pipes, except that one rather exceptional flood topped the dam when it was up at elevation about 60. The diversion must have been simple and not large in quantity as the cost is stated as only $19,000.

The Medina dam near San Antonio, Texas, presented probably the extreme of favorable conditions for simple and inexpensive diversion. The dam is 166 ft. high across a canyon whose level limestone floor was some 500 ft. wide. The depressions in the limestone floor were but slight. On each side was an earth bench, say 5 ft. to 20 ft. deep at the foot of the cliff; this material could easily be excavated by scrapers while the masonry plant was being erected. The dam was erected to conserve storm waters, the ordinary flow being so small that it could be easily diverted by means of a sand-bag dam. A flood at any time during the progress of the work could not have done more than cause a few days' delay. There was no deep pit to be filled up, in fact, no large quantity of loose material that could be moved by a flood. The cost of handling the water, though it has not been publicly stated, must have been merely nominal.

In the case of the Roosevelt dam, Type 3, the river was about 250 ft. wide between nearly vertical canyon walls, average depth (through sand and gravel) to bed rock about 30 ft., maximum depth to lowest point of completed foundation 38 ft.; the stream carried considerable silt which would assist in making tight an earth dam. The floods though occurring in fairly well defined seasons were in winter enormous, and at all times very sudden. Hydro-electric power was generated by the Government and sold to the contractor at ½ cent per h.p.-hour. For pumping purposes hydraulic ejectors were used as plenty of water was available from the adjacent power canal at a head of 220 ft., thus the pumping was as simple and cheap as it could possibly be. A large part of the finer material in the excavation was handled by hydraulic excavators. A feature of the permanent project was a tunnel (of 110 sq. ft. cross-section) at river level through the canyon wall around one end of the dam; this, of course, was to be utilized to its capacity to carry the river during construction. As the work was started after several years of low or very moderate flow, the possible or even average flood conditions were not appreciated at anything like their true value, so it was contemplated to construct a flume across the pit in order to supplement the capacity of the tunnel. Soon after this was started a flood of 130,000 c.f.s. occurred, raising the river 30 ft. in fifteen hours. Before it was possible to resume work it had been amply demonstrated that it would be foolish to attempt to build and maintain a flume in such a stream, so it was decided to proceed with the tunnel alone. Temporary dams were constructed above and below the pit; the capacity of the tunnel was about 1300 c.f.s. and work was prosecuted whenever the flow of the river did not exceed this amount. The work was under water many times but each step made the succeeding one easier and finally the masonry reached such a height that troubles were over. The river was rather narrow and there was not nearly so much work below water as on the dams discussed above. The temporary dams could readily be rebuilt of spoil from the pit and from quarries on the hillsides. A permanent flume was a practical impossibility for any feasible flume would have required rebuilding several times, would have been expensive, objectionably in the way of masonry construction and would have added very little to the amount of working time on the masonry. The excavation of the tunnel cost about $21,000; if constructed for temporary diversion purposes alone, there would have been an additional, though not heavy, charge for permanently closing it.

PLATE III

FIG. A.—Roosevelt dam. Note at left the entrance to the diversion tunnel and the Screen tower.

FIG. B.—Elephant Butte dam site. (*Facing Page* 38.)

PLATE III

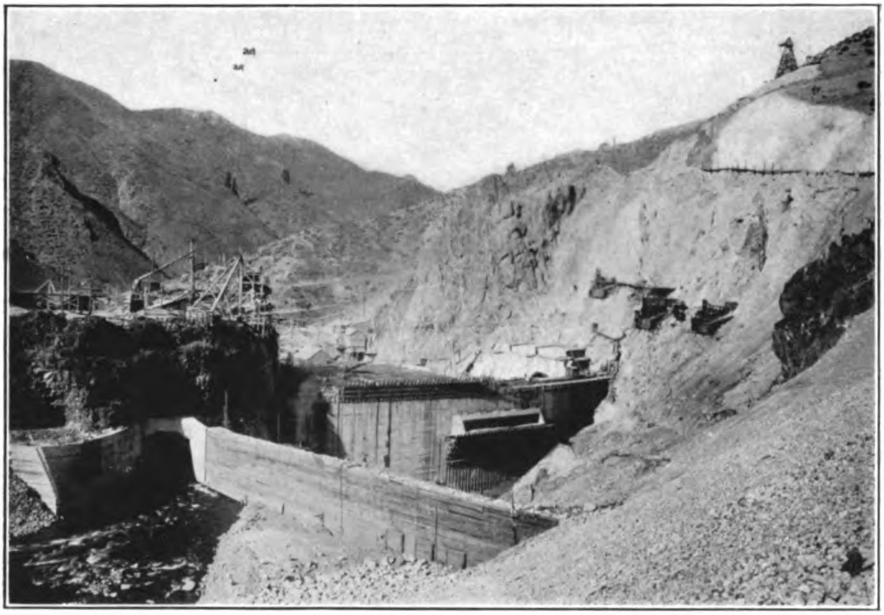


FIG. C.—Arrowrock dam, showing entrance of diversion tunnel.

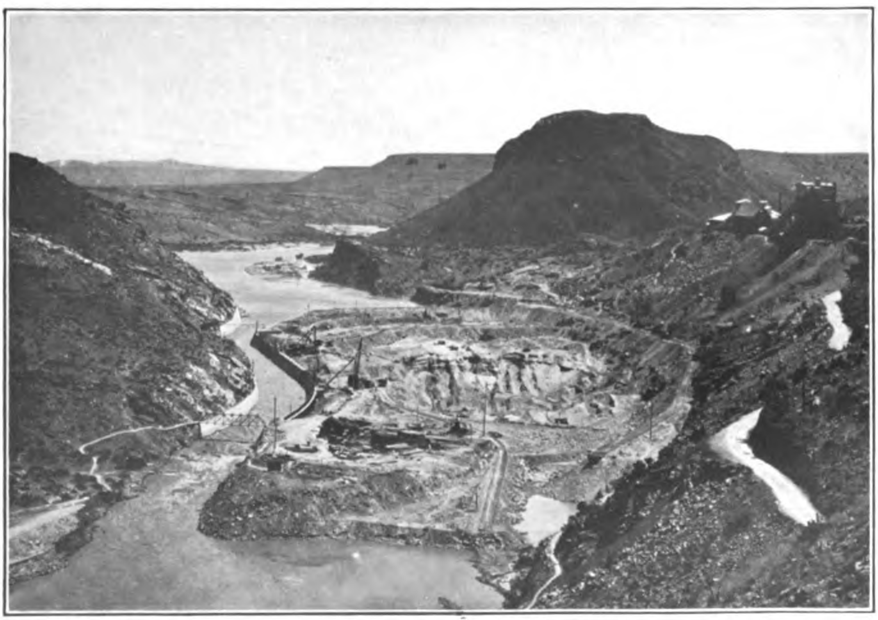


FIG. D.—Elephant Butte dam site, looking up stream during excavation for foundation.

The contractors' expenditures for starting flume, building and maintaining temporary dams, excavating washed-in material, damages except loss of time; in fact, all expense chargeable to floods and diversion of water, were about as follows:

Cofferdam, equipment (cost, erection and repairs)	$ 5,100
Cofferdam, labor	8,300
Cofferdam, materials and power	22,000
Cofferdam, repairs on account of floods	9,900
Re-excavating washed in material and other expense due to floods	13,700
Total	$59,000

At the Arrowrock dam (see Fig. 54) now being constructed in Idaho by the U.S. Reclamation Service the river is about 200 ft. wide between rock walls. As the dam is to be about 350 ft. high the distance from heel to toe of masonry will be somewhat more than 200 ft. Borings indicate that the bottom of the foundation will be generally 65 ft. to 70 ft. below river level with a maximum of 90 ft. The average maximum floods are 15,000 c.f.s. to 18,000 c.f.s. and, under the circumstances, it was the obvious course to provide a channel with sufficient capacity to carry them. Thus the tunnel, which is 487 ft. long, was made 30 ft. in width × 25 ft. in height, to top of arch. The bottom and sides are lined with concrete so that a maximum velocity of 30 ft. per second is expected, corresponding to a discharge of about 20,000 c.f.s. With such a depth of foundation, and 225,000 cu. yd. of excavation below river level, a procedure such as at Roosevelt, where only the low flow was diverted, would have been absolutely futile. The working season between floods would have been too short to make the necessary excavation and lay any masonry, to say nothing of the expense of repeating the process.

As to cost of diversion works, the eleventh annual report of the U. S. Reclamation Service gives the following figures for work up to June 30, 1912: Tunnel excavation and lining; concrete bell mouths at inlet and outlet; and some work on the timber crib cofferdams above and below the pit, though they are not complete; cost $227,490.

At the Shoshone (Wyoming) dam it was 75 ft. between canyon walls, and about 85 ft. below river level to the lowest point of foundations. Here a small tunnel 10 ft. square in cross-section and 500 ft. long was driven; 900 ft. of 9 × 13 flume carried the water to the tunnel and 400 ft. of similar flume returned it to the river below. The temporary diverting dam was a crib structure 430 ft. long and 17 ft. maximum height,[1] filled with earth and rock and covered with planking. The diverting works were planned to carry 2000 c.f.s. The flood season occurs from May to August when discharges of 12,000 to 17,000 c.f.s. may be expected.

The working periods, and the causes for the various delays when work was suspended, were as follows: Contract let September 23, 1905, winter of 1905–1906 built flume, flood destroyed flume June 13, 1906, after which other contractors took over the work and rebuilt flume. Flood July 5, 1907, took out 100 ft. of dam, repairs made so that excavation was started December 2, 1907, and completed April 1, 1908. First concrete laid March 30 to April 12, 1908. Flood till April 26, 1908. Concrete April 29 to May 2, 1908. Flood conditions till August 28, 1908. Concrete September 25 to November 30, 1908. Severe winter weather till March 16, 1909. Concrete March 16 to 29, 1909. Flood March 29 to September 1, 1909. Concrete September 1 to January 16, 1910, completed. Thus during something over four years elapsed time, one winter season of four months saw the excavation all taken out once, and a total of 202 working days divided into five or six periods was required to build all of the masonry. It is evident that between the winter seasons on one hand and the flood seasons on the other, complicated further by the necessity for repairs after the floods, the actual working periods were short and precarious, and might easily have been rendered ineffective by a slight adverse departure from actual conditions. Depending on the value of time in completing the dam, it might have been economical to spend much more time and money on diverting works that would have handled the floods, and then built the dam in one year.

At the Pathfinder dam the river diversion was accomplished through the permanent outlet tunnel. (See Fig. 52.) This tunnel was 480 ft. long, 13 ft. wide, 9 ft. side walls, 10 ft. to arch, lined with concrete and cost about $33,000. The width of the canyon at river level was about 90 ft., and the bottom of the foundation was only 20 ft. below. The water was diverted into the tunnel by means of a rock fill cofferdam. As this was constructed during the winter when much ice was running in the river, it was found to leak considerably with the advent of warm weather. The leak-

[1] Originally planned 200 ft. long × 12 ft. high, contract price with flume was $37,000. See page 249.

age was intercepted by sand-bag dams and conducted across the site in a small flume.

At the Hales Bar dam recently completed for the Chattanooga & Tennessee River Power Company, rather peculiar and very adverse conditions were encountered. The dam is some 1200 ft. long and 37.5 ft. in height above low water. There was about 5 ft. depth of water, and below that about 10 ft. of loose material overlaying a bottom of limestone which contained many seams and fissures. For portions of the length of the dam, extending from each end toward the center, the foundation was satisfactorily unwatered by surrounding them with ordinary crib cofferdams as described under Type 1 of diversion works. (See Plate V.)

The bottom of the river was entirely destitute of any fine, banketing material which would have in large measure sealed the fissures. Further, for somewhat over half of the total length of the dam the fissures carried so much water that it was impossible to prosecute the work by ordinary methods. The method finally adopted was unique in its application to such a situation, was entirely successful and reflects credit upon those who conceived and executed it. Briefly the method was to sink reinforced concrete caissons which were later incorporated with the permanent structure of the dam. The caissons were usually 70 ft. up- and downstream by 54 parallel to the dam; though for a portion of the work two rows of smaller caissons were used. The cutting edge is beveled inside at an angle of 45 deg., so that the roof of the working chamber which is 4½ ft. above the edge has dimensions 9 ft. less each way. The caissons were sunk through the loose material and into the rock to a maximum depth of 36 ft. below low water to the cutting edge; though after preparing the bottom, and in treating the crevices, concrete was put in for some 10 ft. or 12 ft. below that elevation. Concrete was added on top as the caisson was sunk, and on reaching and preparing an acceptable bottom the entire working chamber was filled with concrete. The caissons were started upon loose material above the water level, a crib structure surrounding the site first having been built and filled with sand and gravel. The crib served to hold the loose material from being washed away by the current.

CHAPTER III

PREPARING THE FOUNDATION

Masonry is or should be much sounder and tighter than rock, so the question of a foundation is how much indifferent rock should be replaced by good masonry. The depth to which it will be necessary to excavate the rock has probably been tentatively estimated in advance from information gained from borings. While this estimate may be modified greatly as the excavation proceeds, it is well to start excavation with some idea as to probable depth in order to lay out the work and govern the methods. If a horizontal seam is known to exist, which may be tentatively assumed as the limit of the excavation, care should be taken that drill holes do not extend below it, or if they are drilled below it refill them with sand up to the seam so that the explosive will come above it.

Before building any masonry or finally passing upon any portion of the foundation, a large area should be exposed for inspection in order that one may be sure that excavation of an adjacent section may not demonstrate that the section under consideration should have been carried lower. One may at times, when struggling with water or loose material, be tempted to start masonry upon a small area, but in the bottom of an important foundation this should be carefully guarded against. When a stage has been reached such that the bottom of the dam is built, and the masonry is being extended both ways up the hillsides, this caution need not apply with as much force. In other words, by the time the entire bottom has been rigidly inspected, accepted, built upon and left behind, considerable experience has been acquired regarding the particular foundation. Hence there remains less chance of encountering a surprise, and smaller areas may be accepted at a time. Say 100 ft. square has been exposed in the bottom of the foundation of a big dam, there are uncertainties upon three sides; while for the same area half way up one side there is a less uncertainty on but one side, *i.e.*, the uphill side.

Final Stage of Excavation.—As the rock excavation proceeds toward a possible satisfactory foundation, increasing care should be used that the rock is not unnecessarily shattered. The amount of explosive should be limited, and, if possible, black powder should be used instead of dynamite. A common specification is that the final 2 ft. of rock excavation shall be made by picking, barring and wedging, and without using any explosive whatever. The desired result is obviously that rock shall not be shattered or tight seams opened below what might otherwise prove to be an acceptable foundation. While this result might be accomplished by 6 in. of barring and wedging in some kinds of rock, and require several feet of barring and wedging in some other kinds, the final foundation should show no traces of the effect of explosive. It will assist in the proper appreciation of a foundation if one conceives of it, not as a rock foundation, but as a foundation composed of an aggregation of rocks more or less intimately associated. With this conception in mind, aim so to prepare the foundation that it will consist of the smallest number of rocks within reason. Seams varying from tight to wide open, occur all through it; they may be empty, may carry water or may be so filled with clay or other loose material as to be classed as mud seams and be tight in the sense of not permitting leakage.

N.B.—The term "tight seam" may be used in two senses:

(a).—A seam so thin that some force or shock must be applied to separate the rock on the two sides of it.

(b).—A seam of any width or thickness so filled with clay as to be tight against leakage.

Generally the seams, whether with or across the stratification, become fewer and tighter as the rock is penetrated, in other words, the rock becomes more massive. From the foundation should be removed all rock that has been moved from original position, that sounds hollow on being struck with a pick or bar; or small pieces that would serve simply to keep the masonry from contact with more massive rock below. Reduce as far as practicable the number of lin. ft. of seam that will come in contact with the masonry, thus reducing the likelihood of an uplift pressure.

Soft rock does not by any means indicate rock unsuitable for a foundation; it may apparently be quite soft and still be entirely able to bear the pressure of the proposed structure. It is also likely to contain less objectionable seams than a harder rock. In building upon rock whose stratification is horizontal or nearly so, it may be well to go into it to such a depth that there will be a large mass of rock in front of the toe of the dam, in order that there may be a considerable resistance to any tendency toward sliding, either at the bottom of the masonry or at a still lower plane. In the case of high masonry dams it is usual to excavate in the foundation a so-called cut-off trench, parallel to and under the dam near its upstream side; later filling it with masonry as part of the dam. The purpose of this trench, aside from a possible small value as a bond with the rock, is to cut off possible leakage under the dam. Obviously a trench say 20 ft. deep below the general foundation will get into tighter rock, and cut off any horizontal seams as effectively as if the whole foundation had been carried down. In order to avoid shattering the sides of the cut-off trench while excavating it, the sides should be channeled or treated to obtain the same effect. At the Wachusett and several other dams, a line of 3-in. diameter holes 3 in. apart in the clear, was drilled on each side of the trench, forming planes of cleavage, to which the rock readily broke when shooting out the mass between the lines. The side lines of holes were not loaded.

Cleaning.—The rock in the foundation has long existed at a relatively even temperature, protected from disintegrating influences, and many kinds may crumble, scale or otherwise deteriorate rapidly on exposure to sun, air, or a different and fluctuating temperature. Hence it may be found necessary to go over the bottom for a final preparation just in advance of the masonry, even if the foundation had appeared perfectly satisfactory but a short time before. All seams that are wide enough should be raked and washed out so that the mortar or concrete may be forced into them for some depth.

For the final cleaning stiff wire brooms should be employed; also jets of water under considerable pressure. Indeed, if proper jets are used they may, in some kinds of rock, accomplish a final and highly desirable step in the excavation. A good jet has a most admirable way of searching out and removing superfluous or weakly bedded fragments The final foundation should be absolutely clean from everything but microbes.

Starting the Masonry.—We arrive now at that feature which, of all others in dam construction, is an art rather than a science. Practically all dam foundations are more or less wet, and to handle the water properly while starting the masonry is a process to tax one's experience, ingenuity and patience. Every difficult foundation is a masterpiece. The desired result may indeed be simply and positively stated, namely, keep the water out of and away from the masonry until the masonry has set; but a correspondence course might as successfully attempt to qualify one for painting a Mona Lisa as for building a difficult foundation.

However, some of the obvious steps and processes will be briefly described in the hope that it will assist in an appreciation of the art. Most of the water entering the pit enters along the up- and down-stream sides at or near the foot of the slope of the loose material, *i.e.*, above the rock surface. It may be necessary to, in effect, pave portions of the slope with rock from the excavation in order to hold it from flattening out indefinitely and coming into the pit. If the loose material itself contains enough stone of suitable size, a practically equivalent paving will naturally result in a short time from the action of the water coming through the slope. The finer material will be washed in, while the bank ravels and recedes until it becomes stable.

The main body of water should be intercepted at the edge of the rock excavation and led to a sump situated preferably outside of the masonry lines. This can probably be accomplished best by a small masonry wall. Having selected the area, preferably in the lowest part of the pit, upon which to start the masonry, examine the adjoining or surrounding rock bottom in order to determine the source and quantity of water entering the area which it is possible to divert or keep out. This diversion may, like the first, be accomplished by small masonry walls leading to the same or other sumps, or a small masonry dam may divert the water through suitably sized pipes. If the water cannot readily be made to flow to a lower sump, it may be necessary to pump or bail from a small reservoir thus formed.

After all practicable diversion of water from entering the area has been accomplished, some that must be overcome in some other way will remain. Water may still be entering through seams all over the area, the previous operations being in the line of reducing as much as possible the quantity to be finally overcome. In this last process appears the art of the artist or the helplessness of the tyro. The essence of the ultimate element of the process is the actual smothering of the water by the masonry, and the consequent forcing of the water to take a course via the rock seams to remote and higher outlets. Determining considerations are the quantity of masonry to be laid, the height to which it must be brought, and the time required to do it; also the fact that for a short time a given depth of concrete will balance the head of a greater depth of water. No stream of water is so small that its possible effect may be ignored.

The aim should be to handle it so that its effect, if any, will be not under the masonry, but on top or at the side where it can be readily observed and remedied later. Not only that but the builder must know after the operation has been completed that it has been successful.

The effect of moving water in contact with fresh masonry is of course to remove the cement, and if given time enough it will simply leave a mass of sand or gravel or stone without any cement. If forced to find an outlet through a mass of concrete, the water will merely render it nothing but a mass of sand and gravel, while if the water runs over or at the side of the concrete it will simply remove only enough cement so that its channel will be paved with the sand and gravel. The first condition may be more or less complete, and may or may not be recognized, while the second condition will be readily recognized and as readily repaired at some future time. It may often be found desirable to introduce pipes in the masonry to serve as vents or channels for the water. These should be carefully placed so that there will be a free connection between their end and any water-bearing seam; also that they may serve their purpose with the least possible head of water against the masonry. They should be extended carefully by adding short lengths at a time till the masonry around their inlet end has acquired some strength. The stream of water itself may be trusted to keep free its own passage into the pipe, and (if the pipe is properly set) with an insignificant surrounding area of bottom affected by the moving water. The various water-bearing seams and their obvious or obscure connections should be carefully observed in order to determine how the water may be successfully backed through them. Depending upon the size and configuration of the area first to be built upon, it may be desirable that all of the higher part be built upon first, thus chasing the water to the lowest part, which may be easily done. In the area thus first covered introduce the necessary pipe vents, then, when the masonry has hardened sufficiently, concentrate upon the relatively small lower area, bringing it up in a short time, to such a height that the course of the water will be reversed so as to outlet through the vents previously prepared. In an operation like the above, the element of time is an important consideration in connection with the rate of flow. Obviously, some appreciable period of time is required for the water to produce a harmful effect; therefore reverse the stream before it gets that time.

Forty cu. yd. per hour of masonry construction may accomplish successfully in one hour what 5 cu. yd. per hour for a month might not do. Under such circumstances it may be well to mix the mortar or concrete considerably drier and richer in cement than would be otherwise advisable or necessary in order that it may absorb some of the entering water and even lose some of the cement without harm. It has been attempted above to outline only some of the more common and obvious methods of procedure. In actual practice other expedients, as well as many combinations and variations of the above, may be resorted to.

With the gradual spreading and raising of the masonry until the water-bearing bottom is covered, the various vent pipes should also be brought up. No attempt should be made to discontinue them till there is abundant weight of masonry to hold whatever pressure might result from closing them. While this elevation may be a matter of conjecture, it should be at least well above the original surface of the rock. While these pipes are being brought up a most excellent opportunity is afforded for studying the water-bearing courses under the dam, in order to ascertain the connections between the various seams and how best to discontinue the pipes by grouting or otherwise. The streams from some of the pipes may very readily respond to any raising or lowering or shutting off of the outlets of adjacent pipes, and a free connection of seams shown most clearly. Others may respond more slowly or even not at all.

Grouting.—The amount of grout to be pumped into the foundation may in many cases be so small that a hand force pump such as is made especially to handle grout will be ample for the purpose. (See Plate VI, Fig. C.) Where the anticipated amount of grout is large it will be better to arrange for forcing it in by using compressed air.

A very satisfactory apparatus for the purpose is the Caniff grouting machine. (See Fig. 18.) Cement and water are introduced through the inward swinging door on top, agitated and mixed by blowing in compressed air at the bottom, and then, after shutting the door, blown into the hole by air entering the tank near the top. The necessary manipulations of the valves and their sequence should be obvious from the illustration. The machine is double in order that one tank may be filled as the other is being emptied. In addition to doubling the capacity of the grouting gang, continuity of the operation is often highly desirable or necessary to obtain the best results.

It is assumed that a pipe has been connected with the hole to be grouted, and the connection secured by cementing the pipe into the hole; or in case of a pipe leading from a seam, the connection is secured by having built a sufficient depth of masonry around and above it. To this pipe, which should be 2 in. in diameter, connect the outlet pipe from the tank, put the grout into the tank, close the grout inlet and apply the air pressure through the top of the tank. While using a grout pump or tank, it should be occasionally washed out to prevent clogging up. Of course, it should also be cleaned most thoroughly on discontinuing work.

The hole may first be tested by forcing in clear water; the rate at which water can be forced in being some index of the amount

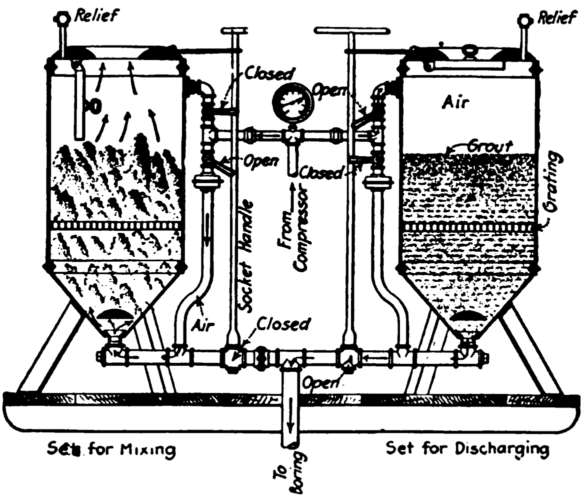


FIG. 18.—The Caniff grouting machine.

of cement required for a suitable grout. A free open hole may first be treated with a heavy grout, finishing up with a thinner mixture, such as would be properly used in a relatively tight hole. It will usually not be necessary to force into every pipe, as when forcing into one the grout will make its appearance at any adjacent pipe or pipes which connect with the same system of seams. Any such pipe or pipes from which grout is issuing in such volume and of such consistency as to make it apparent that the intermediate seams are well filled, may be shut off at once, as the purpose is accomplished, thus forcing the grout on its way to the next outlet.

Where the connections (via seams) between pipes are reasonably direct and free several pipes may be satisfactorily filled by forcing into one. Should the connection be indirect or not so free, the fact will be apparent from the quantity and character of whatever grout issues from the pipe. In that case the first set of pipes may be closed and the pump or tank moved to another pipe on what may be called another system. It is always well to go over the holes a second time after the grout has had time to set, say the next day, and test the completeness of the first work; often some additional grout (even if thin grout) can be forced in. Each set or system of holes should be tested, preferably by pumping into some hole from which the grout issued the first time.

The entire operation is important enough to justify some study in order to secure the best results, the data being the behavior of the water in the various pipes as they are brought up, supplemented if desirable by such tests as forcing into some of them water so colored that its outlet may be readily and positively discerned.

If holes are tested by forcing in water or air at the time the foundation is being prepared, and again (as a preliminary to grouting operations) after much masonry has been laid, there may be observed an effect due to the weight of masonry tending to close the seams in the rock. This is said to have been quite distinctly the case at the Kensico dam where some masonry 100 ft. in depth was laid in the deepest part of the gorge, before certain holes in the rock were grouted. Whether the holes are grouted before or after such a settlement would seem to be immaterial, for the only essential point is that the weight of rock or masonry above the seam being grouted should be so ample that the opening of the seam under the applied pressure will be absolutely prevented.

While some foundations have been thus grouted over their entire area, and while in the future there may be others where such procedure may be proper, still the thing which is logical and in consonance with recent practice in draining foundations would be to grout a zone along the upstream face, leaving the remainder open so as to pass any water which may penetrate that zone. If similar treatment is given to the entire area, it may result in making the downstream portion the tightest portion, conducting to an uplift pressure. The matter is further discussed under Drainage of Foundations, pages 105–106.

Data on Two Grouted Foundations.—Information regarding two rather extensive grouting operations is here abstracted from published accounts.

The first work was at the Estacada dam near Portland, Oregon, and the second at Lahontan dam. Undoubtedly, experience on the first suggested or influenced somewhat the procedure on the second. While the formation at the two places is quite different and doubtless accounts very largely if not entirely for the difference in results, it is interesting to observe that on the latter work grouting was done at 100 lb. pressure, whereas 200 lb. was used on the first.

The Estacada hydro-electric power development of the Portland (Oregon) Railway, Light & Power Company includes an Ambursen dam 90 ft. high which was completed early in 1912. The foundation was extensively treated by drilling and grouting. The following notes regarding the work are abstracted from an admirable paper entitled "Grouted Cut-off for the Estacada Dam," as contributed to the January, 1914, *Proceedings* of the American Society of Civil Engineers by Harold A. Rands.

The entire region is covered by a lava flow, some 200,000 square miles in extent, to an average depth of 2000 ft. This volcanic débris is found in every degree of solidity and hardness, varying from the hardest basalt rock to material as loose and friable as garden soil. At the immediate site of the dam the material is a lava conglomerate or breccia, composed of irregular hard fragments embedded in a softer matrix which varies from friable sandy material to a fairly compact clay. The material was such that a Sullivan diamond drill with a single tube core barrel obtained slightly less than 5 per cent. of core; a double tube core barrel used on part of the work produced nearly 80 per cent. of core.

Holes were put down in two rows 6 ft. apart, with 6 ft. spacing of holes in the row. These were known as primary holes, and after being grouted the treatment was tested by one (or in cases) two or three proving holes between the rows. The total number of holes was 555, aggregating 34,038 lin. ft. At first both steam-operated Sullivan diamond drills, and electrically operated Davis-Calyx shot drills were used. After using the diamond drills for 1119 ft. of hole they were moved to other work and the remainder of the drilling was done by the shot drills, using the G-O class machine of the Ingersoll-Rand Company. The drills were each driven by a 5-h.p. motor at 187 r.p.m. The bit had an outside diameter of 2⅜ in. and the core of 1½ in. The average progress was 13 ft. per ten-hour shift. The grouting was done by a Caniff machine, using air at 250 lb. per sq. in., usually starting the process at 25 lb. pressure, or whatever was required to start the discharge, and ending up at 200 lb. as the hole tightened to refusal. Though thicker and thinner

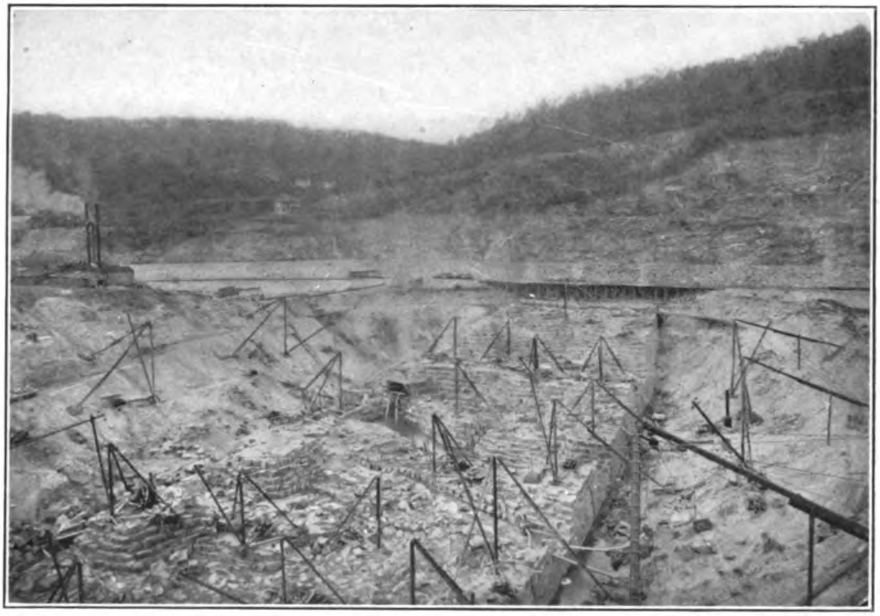


FIG. A.—New Croton dam, showing arrangement of derricks. Note at far end the masonry wall between the temporary diversion channel and the pit.

FIG. B.—New Croton dam in early stages, temporary channel in foreground.

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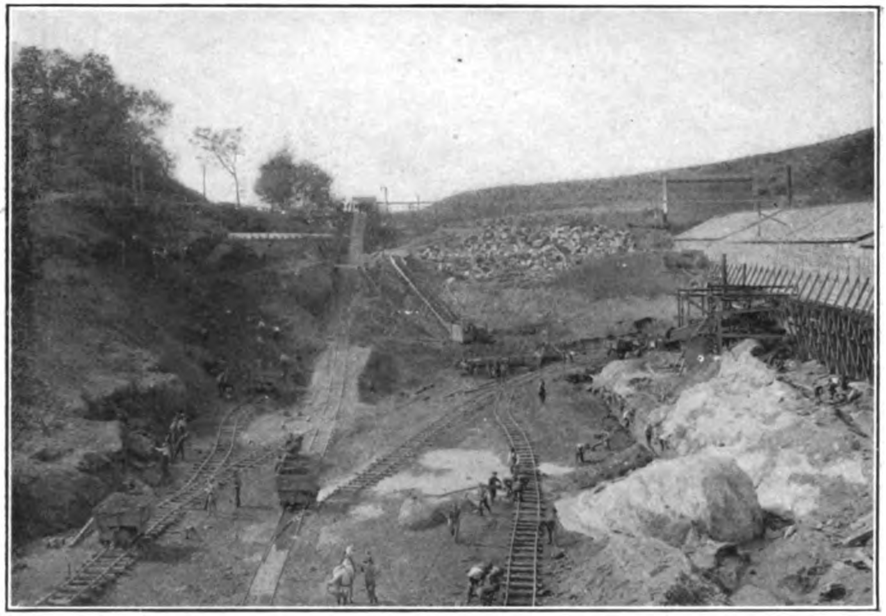

FIG. C.—Wachusett dam site, looking upstream, showing excavation previous to installation of cableways. Large flume at right, small flume at left of incline.

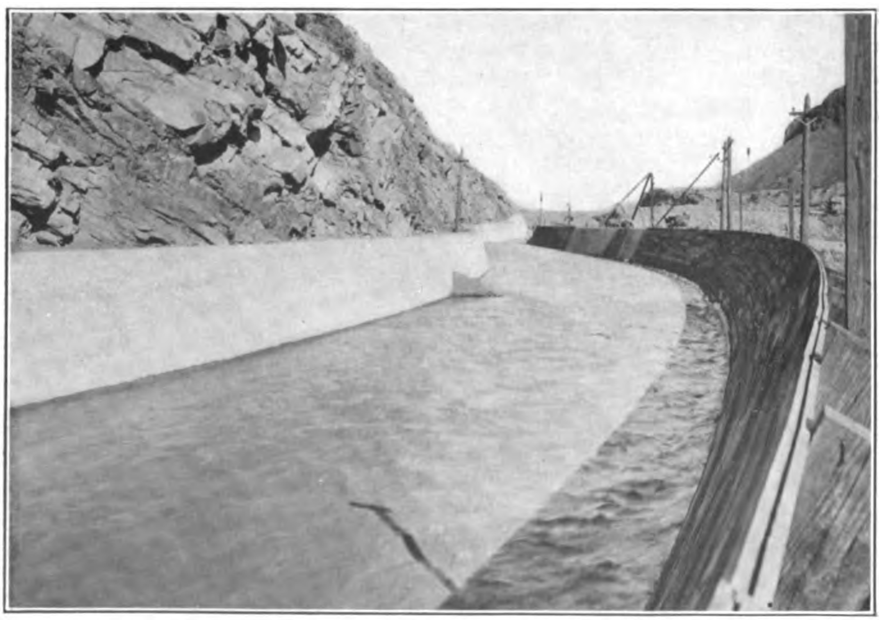

FIG. D.—Temporary diversion channel at Elephant Butte dam, looking upstream. See also Plate III. Fig. D.

mixtures were used at times, the best results were obtained with one part of cement and three to five parts of water, by volume.

Twenty-five primary holes in the left section, grouted during September 1910 showed results as follows: Average pressure 19.4 lb.; average leakage 85.7 gal. per min.; total cement 167.25 barrels; average per hole 6.69 barrels; most cement in any one hole 40 barrels. The water for the foregoing leakage tests was supplied from a pump and the pressure was observed from a pressure gage. In the spring of 1911 holes in the right channel were tested and grouted as recorded in the following table:

	Leakage gal. per min.			Barrels of cement used in grouting		
	max.	min.	ave.	max.	min.	ave.
39 First primary holes, set in river bed, tested from tank 182 ft. above bottom of hole.	100.0	1.0	53.7	9.25	1.0	4.0
18 Second primary holes, casings set through a capping of several feet of concrete, tested from tank 146 ft. above bottom of hole.	63.0	1.0	13.0	11.25	1.0	2.72
12 Proving holes, conditions same as for the 18 above.	44.5	1.2	7.95	4.5	0.75	1.73

In order to show how much tightening between the foregoing primary and proving holes may have been due to the concrete capping, and how much to the grouting, compare with the following holes in an adjoining but somewhat tighter section in which all drilling and testing followed the placing of the concrete.

	Leakage gal. per min.			Barrels of cement used in grouting		
	max.	min.	ave.	max.	min.	ave.
18 Primary holes, 146 ft. head	17.0	0.6	4.4	10.5	0.75	2.49
4 Proving holes	2.8	0.8	2.2	1.75	1.0	1.19
Island section, May and June, 1911						
14 First primary (Casings through concrete, 146 ft. head.)	8.5	1.4	4.1	50.25	9.5	6.85
9 Second primary (Casings through concrete, 146 ft. head.)	6.6	0.7	2.4	1.5	1.0	1.27
6 Proving (Casings through concrete, 146 ft. head.)	2.3	0.9	1.2	1.25	1.0	1.05

After the dam was completed and with water standing at the crest, two holes were drilled underneath the dam for the purpose of observing leakage through or under the grouted zone. The first was 35 ft. downstream from the cut-off, was drilled 2.5 ft. lower than the grouted cut-off, and with the top of the casing 59 ft. below the

Return to left channel, September and October, 1911 conditions as in island section above	Average leakage of holes in gal. per min., at depths of					Cement used barrels, average
	10 ft.	20 ft.	30 ft.	40 ft.	50 ft.	
14 Downstream primary	10.3	15.0	29.9	33.2	47.3	8.38
13 Upstream primary	3.05	4.3	14.7	47.2	56.9	6.09
11 First proving	1.4	3.9	15.5	50.7	63.6	1.05
12 Second proving	1.6	5.1	13.7	22.1	31.7	1.84
17 Final proving	2.0	3.8	6.5	15.0	35.6	1.55

water level in the reservoir the leakage was 7.5 gal. per min. The casing was extended upward and the water came to rest at elevation 24.2 ft. below reservoir level. The second hole was 30 ft. downstream from the most pervious part of the grouted cut-off and was drilled 1 ft. below it; with the top of the casing 72 ft. below reservoir level the flow was 53.8 gal. per min., with the casing carried up to 13 ft. below the reservoir level the flow ceased.

COST DATA

Quantities

Total drilled 555 holes			34,038 lin. ft.
Average per drill per ten-hour shift			13.2 lin. ft.
Average shot per drill per shift			1.1 lb.
Primary or outside holes grouted	375 taking	1,526.5	bbl. cement
Proving or middle holes grouted	160 taking	275.25	bbl. cement
Tight holes filled	12 taking	15.00	bbl. cement
Holes lost	8		
Setting casings, etc		125.25	bbl. cement
Total	555 taking	1,942.00	bbl. cement
Most cement in any one hole		50.0	bbl.
Average for 535 holes taking grout		3.37	bbl.

Cost

Labor drilling	$19,842.60	$0.59 per lin. ft.
Labor grouting	6,285.32	0.18 per lin. ft.
Cement at $2.20 per bbl. at cut-off	4,272.40	0.12 per lin. ft.
Repairs, oil, waste, shot, etc	5,908.35	0.17 per lin. ft.
Depreciation on grouting plant, 50 per cent	5,116.70	0.15 per lin. ft.
Total direct	$41,425.37	1.21 per lin. ft.

Other charges were prorated as follows:

General plant, camp, etc	$15,304.63	0.45 per lin. ft.
Cofferdams and pumping	5,121.63	0.15 per lin. ft.
Engineering and superintendence	6,414.91	0.19 per lin. ft.
Total cost	$68,266.54	2.00 per lin. ft.

The following conclusions were drawn from the experience at Estacada. The first four are undoubtedly sound; the last one though probably justified in that particular formation might not apply to a different one.

First.—Do all drilling, testing and grouting through casings set in the concrete cut-off.

Second.—Do all testing from elevated tanks, and not by pump.

Third.—Test and grout each hole as soon as drilled, and for a few days thereafter keep the drills away from the probable zone of diffusion.

Fourth.—In grouting, especially at high pressures, it is best to close the valve before the tank is entirely empty, because the air which follows the grout into the hole is apt to make trouble.

Fifth.—Begin with a comparatively thin grouting mixture and, if taken freely, thicken until each succeeding batch requires either an increased time for discharging or an increased pressure. To force charge after charge of thin grout into a hole probably means in a great measure the wasting of cement.

For further interesting information, and for results expressed in much greater detail reference should be made to the paper.

In *Engineering News* of April 3, 1913, is a very good description of the grouting of a zone below the cut-off trench of the Lahontan dam. The operation appears to have been very skilfully conducted and successful in accomplishing the desired result. The following is an abstract of the descriptive article:

The material penetrated was red sandstone varying all the way from very hard and close-grained like marble to a tough red clay. It was originally proposed to penetrate 50 ft. into the material with a diaphragm wall built in a cut-off trench, but the scheme was changed to a 30-ft. diaphragm wall extended by 30 ft. of grouted zone underneath. In the cut-off trench were placed 5-in. diameter pipes, of No. 24 gage galvanized iron with stove pipe joints; 26 ft. of such pipe, supplemented at the top by 4 ft. of 4-in. wrought-iron pipe, completed the casings to the depth of the trench. The pipes were placed 3 ft. apart in two rows and staggered so that the effective spacing was 18 in. The boring and grouting were done through these pipes after the trench had been filled with concrete. Two Ingersoll-Rand class G-O Davis Calyx type drills were employed for doing the boring, using the Calyx bit in the clay and the chilled shot equipment in hard rock.

The scheme of stream diversion required the work to be prosecuted in two sections; on the first section alternate holes 6 ft. apart were drilled, tested and grouted, then drills came along again on intermediate holes; on the second section holes 12 ft. apart were treated first, followed up by an equal number half way between, and then by double the number to reduce the spacing to 3 ft. The material was very seamy and much trouble was encountered from pieces which dropped in from the side, thus choking and binding the drilling tools. Grouting had a marked effect in stiffening up adjacent ground so that subsequent drilling was easier.

Drilling was prosecuted continuously in three shifts with two men on each drill. Drillers received 40 cents an hour and helpers 30 cents. The outside diameter of the bits was $2\frac{3}{8}$ in. and of the core $1\frac{13}{16}$ in. The maximum eight-hour progress on drilling was 19 ft. and the average was 6 ft., but omitting some of the earlier experimental work would make the average 8 ft. After the holes were drilled they were tested for leakage, a nearby canal being a convenient source for water at 127 ft. head.

The grouting was done with the Caniff machine described, a small motor-driven compressor furnishing air at 100 lb. pressure.

	Leakage of water under 127-ft. head, gal. per min. per boring	Sacks of cement used in grouting, average per boring
Primary holes, 18 first section	33.1	10.3
Secondary holes, 17 first section	5.5	12.7
Primary holes, 10 second section	71.5	42.3
Secondary holes, 10 second section	28.3	17.6
Tertiary holes, 11 second section	11.2	4.5
Second tertiary holes, 11 second section	6.4	3.7

Lin. ft. of cut-off treated	220
Number of holes drilled and grouted	83
Average depth holes drilled and grouted, ft	32
Total depth holes drilled and grouted, ft	2593
Total sacks cement used	1174
Total cost per ft. of hole	$3.57
Total cost per ft. of cut-off wall	42.12
Total cost of boring and grouting	9267.17
Cost of equivalent length of cut-off by trenching and concreting, based on figures for 30-ft. trench, probably minimum of	12,590.00
Saving by boring and grouting process	$3,322.83

FEATURE COSTS

	Total	Ft. of hole
Foreman	$753.77	$0.29
Making forms, placing pipe	474.94	0.18
Drilling	2397.81	0.93
Grouting	742.68	0.29
Corral expense	98.00	0.04
Supplies, fuel and oil	19.60	0.01
Supplies, lumber	75.03	0.03
Supplies, miscellaneous	192.32	0.07
Material, cement	814.45	0.31
Material, pipe	1121.38	0.43
Repairs	186.74	0.07
Power	86.83	0.03
Repair plant	138.46	0.05
Equipment depreciation	911.87	0.35
Total field cost	$8013.88	$3.08
Engineering	487.03	0.19
Superintendence	195.43	0.08
Clerical	165.45	0.06
Camp maintenance	306.54	0.12
General office expense	98.84	0.04
Aggregate cost	$9267.17	$3.57

CHAPTER IV

MASONRY CONSTRUCTION

It is not the purpose to enter here upon a history of the evolution of the art of masonry construction, however interesting such a history might be; nor to describe ancient masonry structures however creditable and instructive; but to discuss present practice, the changes of recent times and those changes which are likely to be made in the near future.

It is true that in recent years dams have been built under very primitive conditions. Isolated locations remote from transportation facilities, with consequent prohibitive expense of cement and modern plant, have not prevented the building of some important dams. Labor if abundant and cheap can be used in place of machinery, and adequate if not high-class cement can be made by improvised methods from materials quite generally occurring in nature. Admitting the justification and necessity for such methods, and recognizing not only the satisfactory result but the high character and skill of their engineers, still the development of the cement and transportation industries renders it less liable that future masonry dams will be constructed on any radical departure from standard practice. The time up to and including the construction of the New Croton and Wachusett dams, may be called the age of mortar, and the time since then the age of concrete. Each will be described.

Sand.—Sand should be free from organic matter with sound particles, graded in size and with not more than 15 per cent. of material fine enough to be classed as clay. When the cost is about equal, the matter of quality would probably more often favor natural pit sand than sand crushed from rock. To obtain pit sand free from organic matter it is usually necessary only to strip the surface of the pit to a sufficient depth. The soundness of the particles need not be argued, for a material whose particles can be readily divided, crushed or rubbed down is not sand, and is out of the question. The size of the particles should be graded from coarse to fine for the same reason that gravel or crushed stone should be graded for concrete, *i.e.*, in order to get the most dense mixture possible with the same quantity of cement.

In 1900 and 1901, when the desirability and results of proper grading of sand and gravel had not been as completely demonstrated and were not as commonly appreciated as at present, quite extensive experiments in that direction were carried out in connection with the preparations for the Wachusett dam. These experiments were designed to show the strength and permeability of mortar composed of various percentages of coarse, medium and fine sand, the divisions between those arbitrary classifications being the 30- and 100-mesh sieves. Coarse sand was that which stayed on the 30-mesh sieve; medium sand that which passed the 30 and stayed on the 100 and fine sand that which passed the 100 sieve.

The numerous experiments showed most conclusively that mortar made with a graded sand was stronger and more impermeable. A large portion of the pit that furnished sand for the Wachusett dam contained fine material in excess of the proportion which the experiments indicated as being ideal; however, by daily testing of samples and by scraping from different parts of the pit, the standard of 50 per cent. coarse was followed fairly closely. It seemed to be practically immaterial how the remaining 50 per cent. was divided between the medium and the fine, that is, within the possible range of that pit.

As to the effect of material in the sand fine enough to pass the 200-mesh sieve, it seems possible that the effect would be the same as if this material were mixed with the cement as is done in the manufacture of sand cement. The U. S. Reclamation Service is experimenting extensively in this direction. (See *Engineering News* for June 19, 1913.)

The method of measuring the sand should be specified because it makes some practical difference whether the sand is shoveled into a measuring box or drawn into it through a chute from a bin. With sand falling into the box from a height of 2 ft. or 3 ft., the sand spread, leveled and struck off with a straight edge, the box is apparently filled by a less quantity of sand than would be required if it were to be thrown into the box a shovelful at a time. At the Wachusett dam this was quite carefully determined to be 3.61 per cent., *i.e.*, a box measuring 103.61 per cent. when filled from a chute will hold 100 per cent. of sand shoveled in.

Manufactured Sand.—If acceptable pit sand is not obtainable or obtainable only at too great an expense, resort must or may be made to sand crushed from rock. Roughly speaking, it might be said that when the cost of pit sand amounted to 75 cents per cu. yd. it would be well to figure on the cost of manufacturing sand. Of course, special circumstances as hereinafter discussed might be such as to change this figure materially. The cost of pit sand may usually be assumed as being nearly all transportation cost, which depends on length of haul and whether teams or cars are used. The plant charge, including apparatus for screening, elevating and loading, is usually a small item.

The cost of a plant for manufacturing sand also depends on several circumstances. If on account of the character of stone or for any other reason it is necessary to open a quarry and install a complete crushing plant isolated from all other or similar operations, and chargeable entirely to the sand, the cost will be one thing. However, if the stone used in the dam for other purposes (*i.e.*, large stone or for concrete) will crush into suitable sand, the cost for both plant and operation may be very much reduced by manufacturing the sand in connection with other quarrying operations. The quarry for large stone and concrete material naturally is operated on a large scale. Usually there is so much waste stone in connection with quarrying that the stone for sand would be practically a by-product which would require handling only to the crushing machinery, beside which some of the crushing machinery would not have to be duplicated. The attendance might also be very little more if sand was manufactured than if it was not.

An isolated plant to produce say 10 cu. yd. an hour might consist of the following:

One gyratory crusher		$1,500
One small Blake crusher		700
Two sets 36-in. crushing rolls		3,700
Elevators and screen	1,000 to	2,000
Motor		1,000
Compressor and air drills		3,000
Belt, shafting, building bins, etc	3,000 to	5,000
Cars, track and miscellaneous		2,000
Freight, haul, erection	3,000 to	4,500
Total	$19,000 to	23,400

Call the plant cost $20,000 to $25,000 while the labor cost of quarrying the rock and crushing to sand, including renewals, repairs and power would be 75 cents to $1 per cu. yd. Thus 100,000 cu. yd. of sand would cost between 95 cents and $1.25 per cu. yd., assuming no salvage on the plant. Any reasonable salvage allowance might reduce these figures by 5 cents to 7 cents per cu. yd. However, if the operation can be carried on in connection with other quarrying and crushing operations, using largely stone that would otherwise have to be wasted, the extra expense chargeable to manufacture of sand might easily be reduced to a plant charge of $12,000 and a labor charge of 40 cents to 50 cents per cu. yd., thus making the cost for say 100,000 cu. yd., 50 cents to 60 cents per cu. yd.

A saving of about $2000 in plant cost might be effected by using a pulverator instead of the two sets of crushing rolls, provided the rock is suitable for such a machine. Claims are made in behalf of the machine unqualified by any stipulation as to the hardness of the rock. However, unless improvements have been made quite recently, it seems to be a fact that on the harder rocks the lost time and expense for renewing the wearing parts is too great an item to render the process advisable.

The question of whether the rock will make suitable sand should receive consideration not so much on account of the character of the rock but whether or not the proposed crushing process will reduce that particular rock to sand with grains of suitable size and shape. This can be determined by crushing, by means of the process it is proposed to use, a large enough sample of rock to test fairly the process as well as the product. Thus at the Roosevelt dam where all the sand was reduced from broken stone by crushing rolls, the sand was at first unsatisfactory. The rock used was a hard dolomite limestone and the rolls with the best manipulation possible produced a very flaky sand which did not contain enough fine particles. (It may be mentioned here that the fine rolls cannot be set as close as the size of sand required; capacity of rolls in cu. yd. per hour requires that some thickness of stream be fed and that much of the crushing be done by stone in contact with stone.) This rather coarse flaky sand produced a mortar which would not hold water; it was without consistency or any working qualities. Luckily the remedy was immediately at hand in the shape of a rather soft friable sandstone which was used with the dolomite in about equal parts. The sandstone crushed into better shaped particles and so fine that the result of the mixture was almost ideal.

Mortar.—Cement is usually purchased for delivery in sacks under a specification which defines a sack as containing 0.9 cu. ft. of cement and weighing 95 lb. While occasionally a sack should be weighed as a check, it is customary to use a sack as 0.9 cu. ft. and measure the quantity of sand to correspond with a certain number of sacks of cement. The measuring devices should be arranged so that the sand can be rapidly and accurately measured and delivered (with the cement) to the mixer with the least labor. The mixer should be a batch mixer of which many satisfactory types are on the market. The water should also be measured in order to secure uniformity in the consistency of output. It is useless to specify the exact amount of water as it will naturally vary with weather, climate, condition of sand and somewhat with the use to which the mortar is put. Two or three trials will determine the amount of water for a proper consistency. A gage should be set in the measuring tank in such a way that it may be readily adjusted for varying conditions. The mortar should be thinner than in the laboratory practice of making briquettes but should be stiffer than is used in ordinary brick work, as it must properly bed heavy stone. It should be stiff enough for the heaviest work on the dam, and for use in filling in with spalls it may be readily thinned on the dam by the addition of water.

Until within the last few years it was universally considered that mortar should be used before it had begun to set, and on any work where there seemed to be a chance that mortar might remain unused for any length of time it was customary to specify that mortar remaining unused for twenty to thirty minutes after being mixed should be thrown away. Included with other experiments carried on in connection with the Wachusett dam, was a set designed to show whether or not such a specification was necessary. Quick- and slow-setting mortar of both Portland and natural cements, with various periods of from thirty minutes to two hours elapsed time between mixing and using were made into briquettes and tested at various periods after making. Some of the tests were made on mortar worked (*i.e.*, tempered) continuously during the time between mixing and using, and some were simply allowed to stand and were tempered just before using. All the experiments, for the Portland and slow-setting natural cement showed no loss and often a gain in strength. Only the quick-setting natural cement showed some loss.

These results were so conclusive that during the construction of the dam 2-yd. batches of mortar were mixed and used under such conditions that two hours often elapsed before the last of the batch was used. If it had been found that thirty minutes was the advisable limit of time in which to allow a batch to be used, a radical difference in the method of mixing and using the mortar would have been involved, and it would probably have been necessary to resort to the method of sending the ingredients on dry and adding the water on the dam.

Concrete.—Better concrete can be made from gravel than from crushed stone, but it is almost invariably the case on a large masonry dam that it is much cheaper to use crushed stone. The run of the crusher may be used without screening, and if it contains enough very fine stone some slight allowance may be made in the amount of accompanying sand. However, it will be found that it is better to have rather fine aggregate and that 2-in. maximum size is better than 3 in. The reasons in favor of fine aggregate and plenty of sand are that the resulting concrete is more fluid, and that it can run and enter the places it must better than a harsher mixture; also when once in place many more spalls can be put into the concrete. Into a 1–2½–4 concrete of proper consistency and with 2-in. maximum aggregate, many more spalls can be put than are necessary to equalize (for cement per cu. yd. of masonry) a 1–2½–5 concrete with 3-in. maximum aggregate and containing all the spalls practicable for that mixture. The concrete should be used soon after being mixed, not to avoid initial set but because it must be mixed quite wet; so wet that if allowed to stand it will unmix to a certain extent, *i.e.*, the aggregate will tend to settle and the water to come to the surface. This should be avoided as far as possible.

Stone.—For the same reason that concrete aggregate should be graded the stone may, in fact should, be of all sizes from spalls up to the largest that can be economically handled by the plant, although actually some sizes are skipped between the spalls and the smallest "derrick stone." The stones as they come from the quarry should be roughly rectangular in shape, clean and sound, free from seams and thin edges. In practice, however, much work must usually be done upon the stones after they come onto the dam in order to meet these requirements. The quarry forces are apt to be much more concerned with their yardage of output than with the quality; then also the inspection, and acceptance or rejection, are done upon the dam. Adjacent and within reach of each derrick is the stock pile of stone, or space in which to store temporarily the stone coming from the quarry. A great number of stones on hand, and variety of size and shape, are conducive not only to more yardage of masonry built per day but also of a better quality of work. Right here may be mentioned the supreme test of the experience and natural ability of a foreman mason: the expert picks the stone and the non-expert picks the place. In other words, the expert knows where he wants to set a stone and casts his eye over his stock pile, picking a suitable size and shape with an unhesitating accuracy often marvelous; the non-expert goes to the stock pile and puts his hooks onto any readily accessible stone and then sets about to find a place in the masonry where it will fit reasonably well. The stone on coming from the quarry may or may not be dogged so as to hang properly in the hooks for setting, so that much of that work must be done on the dam. Here again we see that the quarry foreman is more intent upon getting the stone out of the quarry. He can load this stone to send out by means of a hold too casual and hasty to be used in setting the stone in the masonry; neither is he interested in whether the stone hangs level and right side up. It is probably not practicable to do all of this work, cleaning, inspection, etc., in the quarry, but as much as is practicable should be done there. It must be remembered that the masonry progress depends on the number and efficiency of the force that can be employed on the comparatively restricted area of the dam, and if any work can be done off of the dam do it. Further, if cleaning is done on the dam that portion of the dam must be cleaned later, and also remember that a stone which is rejected on the dam has been loaded and transported to the dam in vain.

As the stone hangs in the hooks the inspector must see that it meets all requirements not only as a stone but also in connection with its proposed place in the masonry. If there is a seam through the stone open it and permit the pieces to be used if possible. Thin edges should be trimmed off with a sledge hammer. If dirty, swing the stone away from fresh work and clean it by means of a jet of water, and, if necessary, with wire brooms. See that the stone is hanging so that it will come down squarely into its bed without tipping; it may be necessary to set down the stone and move the hooks for this purpose. See that the stone is to set in the masonry on the same bed that it occupied in the quarry, *i.e.*, irrespective of actual dimensions see that the stone is not turned up on its side or end. If the stone is too high compared with the size of its bed, split it and use the two pieces.

Roughly it may be called poor practice to use a stone that is much higher than the least horizontal dimension of its bed. The stone should be of such shape that when set the top shall not overhang the bottom to any large extent. In other words, if the sides depart from the vertical in a marked degree, the departure shall be in such direction that they can be seen from the top. The reason for this requirement will be explained later. (See Settlement of Masonry, page 96.) See whether the shape of the bottom of the stone calls for any special treatment of the mortar bed to receive it. Lastly, see that the stone is wet before lowering it to its bed. The preparation of the rock foundation has been described; equal care should be observed in connection with a masonry surface about to be built upon. It should be thoroughly cleaned by water jet and wire brooms, any laitance, inert or damaged cement removed; also any spall or stone of whatever size whose bed is broken (*i.e.*, has been hit or jarred so that its bond with the masonry has been broken) must be taken up.

The inspector on the dam must assume for the time being that the cement is acceptable, but he should watch the mortar carefully to check as far as may be the character of the inspection at the mixer. He should notice any serious departure from proper proportions of ingredients and the character of the sand. He should also see that the mortar is well mixed and of the proper consistency, calling for more or less water as occasion requires. Any measured quantity of water per batch may require modification, according to whether the sand is wet or dry or according to weather conditions. If the batch of mortar or any portion of it stands for some time before it is used, he must see that it is kept properly tempered. Continuous tempering is not required but occasionally it should be worked over with hoes and, if necessary, water should be added to bring it to the proper consistency for use. If proper attention is paid to this feature there is very small chance, barring a freak cement, that the mortar will become unfit for use within one and a half or two hours from the time of mixing.

To prepare a bed for the stone use plenty of mortar, heaping it somewhat in the center and with 2 in. or 3 in. in depth at the edges. If the bed of the stone is concave it may be necessary to build a portion of the mortar bed to some height; this may be done by using spalls in connection with the mortar. In any case the aim is that the center or concave part of the stone shall first make contact with the mortar, thus insuring that all air shall be expelled as the stone settles and the mortar spreads. The operation may require mortar of different consistency for different stones. Obviously a stone heavy in proportion to the area of its bed should have a stiff mortar, while for a lighter stone it will be sufficient to turn on the hose as the mortar is being spread. When the stone is down in place and the hooks are off, a bar should be introduced under one edge or corner and the stone floated back and forth a few times. This floating tests the adequacy of the bed, brings the stone to a better and more intimate contact with it and flushes out the superfluous mortar. A slight application of force to the bar will be sufficient to effect this if the stone is properly bedded. No amount of force will float the stone if it has sunk too far in its bed so as to rest at any point upon the masonry or any projecting point underneath, and the fact will at once be apparent. Even the location of the "hard spot" is shown, *i.e.*, the point about which the stone may rotate without otherwise moving. The stone should be picked up at once and the hard spot be removed or a thicker bed be prepared. When the stone is finally set the superfluous mortar which has been forced out around the edges may be shoveled up for use elsewhere, but strictly subject to the following precautions:

Conditions must be such that the remaining mortar cannot fall away from any part of the bottom of the stone just set, these

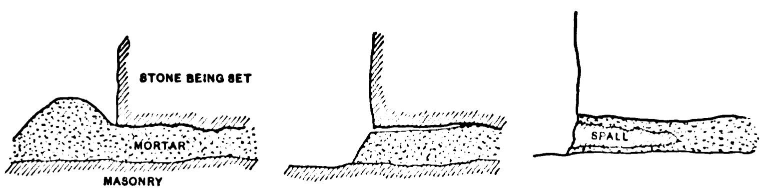

FIG. 19.—Showing the bedding of a stone in mortar.

conditions being consistency of mortar and thickness of bed joint. The mortar which has been squeezed out around the edge of a large stone, piles up and creates what may be called a head (in the hydraulic sense) acting against the escape of more mortar and holding up that mortar to contact with the bottom of the stone. If any of this mortar is removed, the mortar remaining under the stone may not have enough support to prevent its falling away from the stone for some distance in from the edge. The remedy may be: first, for a thin bed joint, leave enough mortar; second, for a thicker joint, produce the effect of two or more thin joints by driving in thin spalls as underpinning. Clearly if the bed joint is quite thin or is thus made quite thin, very little mortar relatively is required to hold the bed up in place. Fig. 19 illustrates the several conditions.

If a proper bed joint is not preserved after it is made the result is no better than if it had never been made in the first place. This whole matter is one that requires most watchful attention. Carelessness in this respect is the cause of more imperfect work than any other detail of masonry construction.

In the past it has been common to specify that many if not all of the stones after having been bedded once shall be picked up in order to demonstrate that the bed had been properly formed and the stone properly bedded. Such a specification is absolutely useless except for the purpose of occasionally enlightening and satisfying some person who has not had enough experience to have acquired an inspector's eye. An inspector who knows his business and attends to it need never be in doubt as to whether a stone is properly bedded. If the stone is picked up it is almost invariably necessary to do some work on the bed before lowering it again, even if the first bedding was perfect. Such work is to shovel to the center the mortar that has been flushed out around the edges. If this is not done the stone on being lowered the second time is often liable to disarrange, sink deeper into or find a hard spot in the bed; if this work is done it restores the conditions and adds little or nothing to the knowledge attending the first bedding of the stone.

Much attention is paid to the bond, that is to say, the arrangement of the large stones, the aim being not only to introduce as many stones as possible but to avoid continuity of joints in any direction. This result, as applied to joints in any vertical plane, is attained with very little effort. It requires, however, some watchfulness to see that stones properly break joint across the horizontal planes or natural layers in which the construction progresses. As soon as some area has been covered with large stone the vertical joints are filled with spalls and mortar, hand laid up by masons with trowels. As regards soundness and cleanliness the spalls, of course, should be subject to the same careful inspection as the large stone. They should be of various sizes, say from the size of a brick up to what one man can readily handle; smaller ones are not economical as they require about as many motions and as much time to handle, and it is cheaper to use mortar. The mortar used with spalls should be thinner than for large stone, and this is accomplished sufficiently by occasionally turning the hose over that portion of the work, the mixing or tempering being done with the trowel incidental to and coincident with the spreading and handling of the mortar. The process of filling the vertical joints with spalls and mortar necessitates the introduction of frequent vertical breaks or joints, limiting the area filled at one time or by a particular mason.

For economy of labor these vertical stops should be built at places of narrowest vertical joint between large stone. Some care should be used that the exposed faces should not be left rough but be smoothed and pointed up so as to be comparable with the adjacent rock face. Obviously these vertical stop-offs should be arranged and constructed on the same idea that the large stone are set, *i.e.*, to avoid any continuity of joint. As the filling approaches the height of the large stones it is well to stop off below their elevation, say at the level of the lowest of them and below the highest. This method economizes time and mortar and furthers the desired breaks in horizontal planes.

On stopping work, as for the night, all exposed mortar joints or surfaces should be smoothed off with the trowel, in effect pointed. This compacts the surface mortar so that when set it is more dense. The smooth surface not only affords less or no lodging place for dirt but is much more readily cleaned if dirty; also it is a better surface to build on or against as air is less liable to be trapped. Loose, detached or thin splotches or excrescences of mortar often dry out in a short time. Consequently they never acquire a set and amount to nothing but so much sand which is difficult to remove satisfactorily on a rough surface. All this, to say nothing of appearance, is another test of the quality of a mason. The work of the good mason will present a neat, cleaned up and pointed up appearance.

Cyclopean Masonry.—The last of the important dams of this country to be built by the method which has been described, *i.e.*, by bedding the large stones in mortar and the filling of vertical joints with spalls and mortar laid up by masons, were the New Croton, 1892 to 1906, and the Wachusett, 1900 to 1905. Beginning with the Boonton dam, 1900 to 1906, a new method was employed which has since been used on the Cross River, Croton Falls, Olive Bridge, Shoshone and other dams. This method seems certain to entirely supersede the former practice. Briefly it is to bed all large stone in concrete, also to do all filling of vertical joints with concrete into which spalls may be rammed.

This radical change is the natural result of the development of concrete practice during the last twenty years; which development has been due to the gradual recognition of the fact that a wet mixture of concrete results in better work than the once universal dry mix. This modified concrete practice, in addition to improving the quality of the concrete, effected a material saving in the labor of ramming the concrete, and also effected a saving in cement on ac-

FIG. A.—Hales Bar dam showing forms for a concrete caisson.

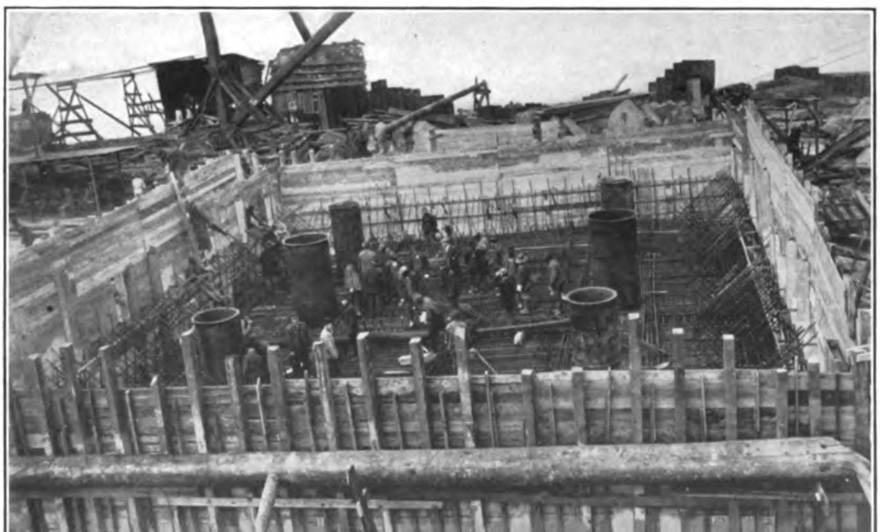

FIG. B.—Hales Bar dam, placing reinforcement and locks for concrete caisson.

(Facing Page 66.)

PLATE V

FIG. C.—Hales Bar dam, placing concrete in caisson.

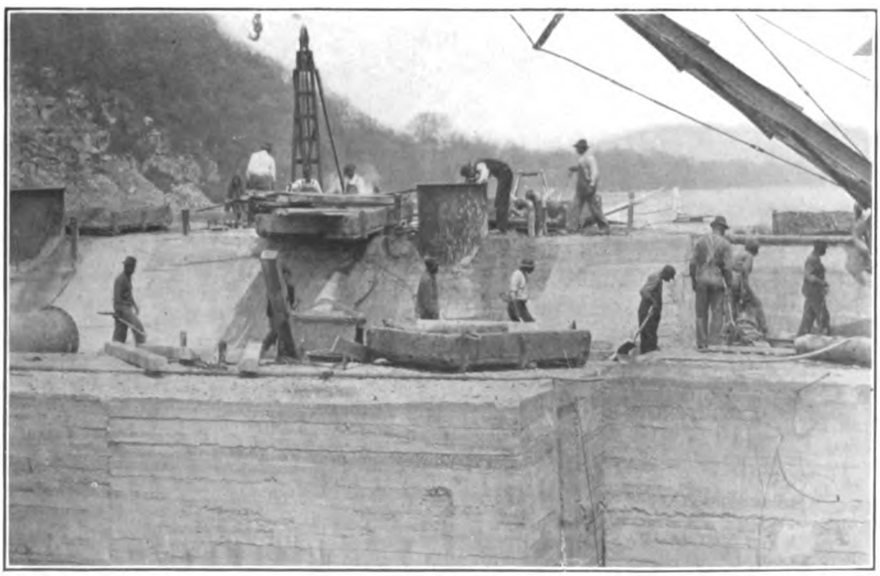

FIG. D.—Hales Bar dam. The crevices between adjacent caissons were raked out and filled with concrete.

count of the spalls that could be rammed into the concrete after it had been placed. Another important cause of the change to a wet mixture was the development of reinforced concrete work and the practical impossibility of satisfactorily ramming dry concrete in connection with reinforcement.

This new method of construction of masonry dams simplifies and cheapens the work immensely besides resulting, if properly done, in a more impervious structure than the old method. The large stone and spalls must of course be subjected to the same careful inspection and cleaning. Fairly wet concrete is dumped down on the wall and the large stone simply set down in the mass, no elaborate process of making the bed and bedding the stone being required. One precaution should be carefully and may be easily observed, namely, have an ample depth of concrete under the large stone which is being placed. One may otherwise be apt to extend the setting of large stone out too near the edge of the batch of concrete where it has run down too thin.

If the concrete has spread out so thin that the bed of the large stone is partly above the concrete, any subsequent batch of concrete dumped near by is practically certain to trap air under the large stone and result in leaving an open space. Except for this simple precaution the setting of the large stone is a comparatively casual operation which may be as well done by reasonably intelligent laborers as by expert masons. It is not necessary or even desirable that the large stone should hang in the hooks with level bed, the same purpose (of expelling all the air as it is lowered) may be accomplished even better by lowering the stone in such a position that a corner or an edge will enter the concrete first and then tilting the stone to its final position as it is lowered.

The large stone may be set so that the joints between them will be anywhere from 6 in. to 1 ft. or more in width, depending on circumstances. Stone say 3 ft. high should be set with the wider joint, and stone 1 ft. high may be set with a much narrower joint. An approximate rule might be that a joint should have a width equal to one-half of the height of the stone, the point being to leave joints that can be readily and satisfactorily filled later. As each large stone is set, spalls may be rammed into the concrete in the joint between it and adjacent stone without waiting for the joint to be filled to the top; as beyond a certain depth spalls cannot be rammed or pushed at one operation and otherwise the bottom foot or so of the joint would contain no spalls. After a considerable area has been covered by large stone set in this manner, the filling of the vertical joints may be completed by dumping in concrete and ramming spalls into the concrete.

If any of the large stone have been set for such a length of time that their concrete bed has begun to harden, it will be necessary to exercise care that the process of dumping the concrete does not break the bed. Usually, however, the area will be covered with large stone and the joints will be filled before enough set has taken place to make it a matter of concern whether the large stone are hit and slightly disturbed or not.

When the concrete is dumped and as it is flowing into and along the joints, men with shovels or bars should assist the process seeing that the joint is completely filled and that no nesting of the coarser aggregate occurs—in short, that the concrete does not unmix. It may often be needful to shovel some concrete into a high, narrow or tortuous joint to be certain that it is properly filled. In order that the maximum of spalls may be introduced into the concrete it may be necessary to make two operations of filling the joints, *i.e.*, not dump at one time or place such a depth of concrete that spalls cannot readily be pushed to the bottom. There should always be a depth of concrete sufficient to bury practically the entire bulk of the spalls for if the spalls protrude to any appreciable extent they interfere with the proper deposition of subsequent concrete. Under no circumstances is it allowable to introduce spalls first and then pour concrete over and around them. It is not economical to try to introduce spalls beyond a certain percentage, as then the labor cost of doing it properly more than offsets the saving of cement. On the Roosevelt dam it was found that of the space thus filled by concrete and spalls, the economical limit was about 22 per cent. spalls to 78 per cent. concrete.

Both of the above described methods were employed for about three years, say 1903 to 1906, lapping over each other as it were during that period while several dams were being finished according to the old method under which they had been begun; since then the new method has prevailed. During this three-year period at least two important dams were begun, embodying in their specifications a method best described as being a combination of the two. In the Roosevelt and Pathfinder dams it was specified that the large stone should be bedded in mortar, as by the old method, and the vertical joints filled with concrete and spalls as by the new. This transitional variation, though it looks at first sight like a compromise between the two methods, is in reality practically the same as the new one. The rate of progress, cost and quality of resulting masonry are practically the same. By bedding in mortar and securing thinner bed joints a somewhat greater percentage of stone is secured in the dam, but as the mortar requires more cement the result is no difference in the amount of cement per cu. yd. of total masonry.

Although so far the results from the transitional and new methods do not warrant the statement that one is any better than the other, still, as will be shown in the chapter on Probable Future Methods, it is probable that the new method or straight cyclopean concrete will be the standard practice, but modified by a tendency toward the use of less large stone.

Comparison of Rubble and Cyclopean Methods.—To compare figures regarding each method, take several prominent examples of dam construction. The Wachusett and New Croton dams were built under the old classic specification, namely, large stone bedded in mortar, vertical joints hand laid up with spalls and mortar; the Pathfinder and Roosevelt dams were built under the combination method, large stone bedded in mortar, vertical joints filled with concrete and spalls while the Olive Bridge dam embodies the cyclopean rubble method, large stone bedded in concrete, vertical joints filled with concrete and spalls.

	New Croton	Wachusett	Roosevelt	Pathfinder	Olive Bridge
Large stone, per cent	50	54	39.6	48.5	25.3
Spalls, per cent	26	17	10.4		
Mortar, per cent	24	29	13.8	12.5	....
Concrete, per cent	....	....	36.2	39.0	74.7
Barrels cement per cu. yd.	....	1.00	0.74	0.90	1.01
Rate of construction in cu. yd. per hour per derrick.	....	3.8 to 6.0	9.0 to 18.0	......	20 to 27
Average rate when conditions were favorable for maximum progress.	About 5.0	Not over 5.5	About 16.5	......	21.0
Total cu. yd.	855,000	280,000	344,000	62,000	488,300 including 56,200 cu. yd. of face blocks.

Progress in Masonry Construction.—The number of cu. yd. built per derrick per hour is the only really logical and enlightening unit by which to compare intelligently progress under one method with progress under another in the analysis of conditions as they apply to particular dams, foremen and derrick locations. The construction plant for any large dam should be so designed that the materials will be assembled, prepared and brought onto the dam as rapidly as the masonry can be built by working all the derricks that can economically be brought to work upon it. In short, the limit to masonry progress should be and usually is the number of derricks that can readily work without mutual interference, and the cu. yd. per hour due to the particular specification under which the work is being done. Obviously some such unit is the only logical basis of comparison as to efficiency of plant and personnel between two dams where the same specification is being followed. A monthly estimate of so many thousand cu. yd. is meaningless unless accompanied by a statement of the number of derricks engaged and the total number of derrick hours.

It is to be hoped that all engineers in charge of such work will keep and publish records of output on the basis of cu. yd. per derrick hour. Monthly estimates should show accurately the total cu. yd. per month. This monthly total may be easily divided into totals for each derrick in proportion to the amount of stone concrete and mortar used by each derrick, which figures can be easily kept by the inspectors, and these totals per derrick should be divided by number of hours. Such analysis and comparison are most entertaining and instructive. Of most interest to the engineer is the monthly estimate divided by derrick hours as this illustrates the different efficiencies possible under various specifications and also at various stages of the construction of the dam. Of more interest to the superintendent is the number of cu. yd. per hour for each derrick as this shows the efficiencies of the different foremen and discloses whether certain derricks are working under some temporary handicap of position, or whether some conditions require remedy.

On one large dam it was the practice for the superintendent to keep similar figures. These figures were published each week and were accompanied by a bonus for the crew standing at the head of the list. Out of about nine foremen the bonus almost invariably went to one of four, which of the four would depend upon what derrick happened to be working in the most favorable position during the week. Thus this record showed both the efficiencies of different foremen and the effect of favorable location.

In addition to the number of cu. yd. per derrick hour, figures showing the relative amounts of large and small stone, mortar and concrete can easily be kept for each derrick. They are enlightening as to character of work being done and should be frequently brought to the attention of the foreman and inspector of each derrick. As an illustration, the following is an imaginary record of two derricks for six ten-hour days.

		A	B
Used	Stone, cu. yd.	408	394
	Spalls, cu. yd.	102	105
	Concrete, cu. yd.	367	336
	Mortar, cu. yd.	143	125
Cu. yd. masonry built		1020	960
Cu. yd. per derrick hour		17	16
Cement used, barrels		871	783

According to the foregoing figures Derrick A has built 60 cu. yd. more than Derrick B, or 1 cu. yd. per hour, and without further scrutiny it would seem that the foreman of Derrick A was the more efficient. On examination of the figures, however, it will be seen that Foreman A used more mortar for bedding his large stone than did Foreman B; also that in the space (vertical joints) filled with concrete and spalls, Foreman A put slightly less than 22 per cent. spalls, while Foreman B put slightly less than 24 per cent. spalls. It is not argued that a comparison like the above should be taken to indicate that the faster rate and the extravagance in the use of cement necessarily accompany each other. There is a presumption, however, that to a certain extent they may be cause and effect; only numerous comparisons between records of different foremen and different records of the same foreman could determine this.

Now let us assume reasonable values for the materials and labor and see how the masonry costs compare.

		A		B	
Stone		408.0 at 0.80	$326.40	394.0	$315.20
Spalls		102.0 at 0.50	51.00	105.0	52.50
Concrete	sand	190.8 at 0.75	143.10	174.7	131.03
	broken stone	304.6 at 0.50	152.30	278.9	139.45
Mortar sand		135.9 at 0.75	101.92	118.8	89.10
Cement, barrels		871.0 at 1.75	1,524.25	783.0	1,370.25
Labor			216.00		216.00
			$2,514.97		$2,313.53

A built 1020 cu. yd. at a cost of $2.46 per cu. yd. B built 960 cu. yd. at a cost of $2.41 per cu. yd. Then it becomes a question of amount of overhead and interest charges plus any other value such as bonus or penalty attached to time, whether the performance of Derrick A or Derrick B should be considered the better. Thus, if five derricks are working and have 300,000 cu. yd. to build, then performance B requires twenty-two more working days to complete, and makes 300,000 cu. yd. at 5 cents or $15,000 less.

Plant has become standardized so that a derrick on one dam means practically the same thing as a derrick on another. They can for similar positions cover the same area under any masonry specification. A derrick crew consists of about the same number of men, viz.: foreman, engineer, and eight or nine men. A less number of men would mean that the derrick was not being worked to its capacity and more men would be in each other's way.

Conditions Governing Progress.—It is believed that the difference between the progress to be expected by the two methods employed on the Wachusett and Roosevelt dams, however startling it may appear at first sight, is correctly represented by the figures in the foregoing table. Under whatever method the masonry is built there are a variety of changing conditions between the start and finish of a masonry dam that affect very materially the number of cu. yd. it is possible for a derrick to build per hour.

First.—In the bottom of the foundation, when starting the masonry, there is some slight delay on account of some final refinements in cleaning and preparing the rock which must be performed coincident with actual masonry construction.

Second.—The delay due to the handling of the water and proper insertion of grout pipes as described in the chapter on starting the masonry.

Third.—The plant is new and untried, some details of the method have yet to be worked out, and the men in all the various capacities even if experienced and able cannot at once take the stride that should develop after a period of team work.

Fourth.—Adjacent blasting or other operations connected with the excavation and preparation of the remainder of the foundation may have an effect on the efficiency of the derricks on masonry. Most certainly will this be the case if removal of material is allowed to encroach unduly on the time of the plant engaged in supplying the masonry derricks.

Fifth.—Exigencies in the way of limited space available may make it impossible so to place the derricks that they will work to best advantage and without interfering with each other.

The maximum of efficiency is reached when the bottom is entirely covered and brought up to such a height as to be above water or any delay from handling of water; when the work of preparing the foundations at each end has been reduced to a minimum or so prosecuted as not to be any interference; when an intelligent plan can be developed for placing the derricks properly, and an intelligent system can be adopted for properly coordinating their necessary changes of position; and lastly when the force has had its training in team work, and any defects in plant operation have been corrected. This maximum efficiency will be maintained without noticeable diminution up to such an elevation that the dam is about 60 ft. in width; at this point two causes (which indeed had begun to operate before) now begin noticeably to reduce yardage and to do so more seriously with every step to the top of the dam.

First.—The face work is a larger percentage of the total work, and as it requires more time per cu. yd. than the interior work, it has a retarding effect on the total yardage of the derrick. The retardation increases up to the top of the dam, and especially on arriving at any special work designed for architectural effect such as pilasters, cornice and coping courses, parapet walls, etc.

Second.—As the area on which the derrick has to work has been cut down, the yardage is affected. A derrick at say 120 ft. width of wall can work in practically three-quarters of a circle of radius equal to the length of the boom. This may be cut down greatly without seriously affecting the efficiency but at about 50 ft. width it becomes necessary to turn the derrick so that one side of the A-frame coincides with the upstream face of the masonry, and the area available is about one-quarter of a circle. Still higher, the end of the other side of the A-frame must be supported outside of the masonry lines, and the reduced area under the boom must still afford room for temporary storage of materials until the wall gets so thin that landing stages have to be provided outside on the downstream face. (See Plate VIII, Fig. B.) Handling to and from such stages is a process conducive to very circumspect movements. The men are either in each other's way, or else the force is reduced somewhat, and in such circumstances all operations are conducted rather gingerly.

In addition to the above general conditions, affecting the efficiency of all the derricks at various stages of the construction of the dam, are the following particular conditions which affect somewhat the efficiency of a derrick almost from day to day: Whenever in the progress of the work a derrick must be moved, it is almost invariably the case that it is moved from a depression to a pinnacle, namely, from a place where it has been building above its level to a place where it must begin building below its level. Assume that a derrick has just been moved to a new setting, it must begin building upon a level 12 ft. to 15 ft. below the level upon which it sets. The masonry in the bottom must be thoroughly cleaned as must also the masonry on the racks as the depression is filled. When the landing and storage space are upon a level with the derrick, it is necessary to climb up and down the rack to handle the loads. The foreman may have to change position in order that his signals may be visible to the engineer, and they may even have to be relayed. The engineer often cannot see the load being handled or the stone being set. Again, because of the longer fall line the load does not follow the boom quite as readily and none of the motions are as precise. While these defects are slight individually, their combination has a bad effect on the output of the derrick. As a partial offset there is the fact that in filling a depression less face work is necessary. This means that while the work on the up- and downstream faces of the dam may be the same, the remaining two sides are against masonry already in, which is easier than building a rack.

The maximum output of the derrick is at the time when it is working between say three courses below and one course above the level upon which it is setting. Above this level certain causes operate to reduce efficiency; *i.e.*, reduction of working area, climbing up and down the rack and possible interference with signals. Of course the fall line is shorter, and the engineer can nearly always see the load. There is a delay from the fact that there is more face work, *i.e.*, referring to the transverse faces or racks which in this operation must be built, while in the previous one they were built against. While the rack is far from being face work in the sense of the up- or downstream faces of the dam, it still involves some selection of the rubble stone and considerable attention to their placing so that the rack may present a proper bond to the subsequent masonry. All the vertical joints between the rack stones must also be carefully filled and smoothed up on the face as it is to be left for some time and must be left with a workman-like appearance.

The foregoing are the unavoidable causes which affect output. The avoidable causes are: capacity of remainder of plant, which should be large enough to keep the derricks on the masonry fully supplied; amount of work trimming and cleaning the stone, which should be done in the quarry as far as possible; conduct of adjacent work such as excavation for foundations, etc.

Arrangement and Moving of Derricks.—The efficiency of the masonry derricks depends greatly upon an intelligent plan for placing them and coordinating their subsequent movements. On starting masonry construction a plan should be made showing the limits of the masonry at the bottom. This plan may be extended and corrected from time to time as additional foundation becomes available at the ends or as the thickness of the dam is reduced. To lay out and preserve a system for placing derricks, it will be found convenient to use cardboard figures cut to show the space occupied by a derrick and covered by the sweep of its boom. The plan should indicate any areas, outside of the masonry lines that may at some stage be occupied by a derrick. If there is considerable depth of refill against the masonry on either or both sides of the dam, and if this refill can come up with the masonry, it may be desirable to place the derricks outside of the dam while such masonry is being built. (See Plate VII, Figs. A and B.) If the derricks are to be served by fixed cableways, their lines should be shown on the plan to see how they can reach each derrick in order to set it, move it or supply it with material. If the cables are traversing it is sufficient simply to know that they cover the entire situation. As much of the masonry must be built by derricks setting upon the masonry itself, it is even more important that these should be properly spaced and their movements systematized.

Obviously with the booms working in segments of a circle there must be some overlapping in order to cover the entire area. For certain thicknesses of wall down near the bottom, a layout might be a row of derricks just outside of each face with another row in the center (Plate VIII, Fig. A), or conditions might indicate two or more rows of derricks on the masonry. (See Plate IV, Fig. A.) A certain less thickness would require the two rows to be staggered as a transition to the one row that must be arrived at with a thickness of 100 ft. or less. (See Plate VIII, Fig. A; Plate X, Fig. D; Plate XI, Fig. A), The foregoing is not meant as an argument that the position and movement of a derrick must be studied and determined as minutely or as far in advance as if it were engaged in erecting the steelwork of a skyscraper. It is an argument, however for the necessity of a fairly definite plan at any stage and an appreciation of how one scheme of setting must merge into another. It is neither necessary nor possible to plan each position in advance to the end of the work. The superintendent, nevertheless, should have very definitely in mind what the situation will be a week hence and have a close approximation to what it will be a month in advance.

After covering what may be called the bottom, the dam as it comes up grows longer and narrower so that the area is not materially affected for some time. The number of derricks that can be advantageously worked does not vary very much from one stage to another. In addition to the normal number of derricks it is well to provide one or two as substitutes in the case of repairs or other contingencies. It is of no advantage, otherwise, to have idle derricks on the ground or to erect them far in advance of the time when they can be used. It is much cheaper to move a derrick than to pay for one. Adequate cableways, particularly if they are traversing, make it a very short and simple matter to move a derrick. It was no uncommon occurrence at the Wachusett dam to pick a derrick out of the line at one end of the dam, set it in at the other and lose less than an hour actual working time of the derrick. Of course, this operation is not always so short and simple. If the loss in efficiency when building in the bottom of a depression or on the top of a pinnacle were properly appreciated, the derricks would be moved oftener than they are. If instead of starting say 15 ft. below its level and building to 15 ft. above, these heights should be halved, the gain in efficiency would more than pay for the larger number of moves.

The accompanying illustrations show the mounting of derricks and their position with respect to the work for quite a number of dams and variety of stages. A peculiar mounting of derricks was resorted to during the latter part of the work on the New Croton dam, where cyclopean masonry was substituted for a portion of the core wall and embankment section, as elsewhere described. Within the working area two square steel towers were erected, a derrick was mounted upon each corner of their tops and the towers were built into the masonry as the work progressed. (See Plate VIII, Figs. C and D.) This method does not appear to possess any startling advantages under ordinary circumstances. The specifications for the Cross River and the Croton Falls dams, which were built soon after, each contained an item calling for a bid price per ton for steel towers built thus into the masonry. It is interesting to note that in neither case did the contractor avail himself of that method of mounting his derricks.

At the Kensico dam, most of the derricks are mounted in pairs on travelers moving upon I-beam stringers, which are in turn supported by concrete piers. The illustrations in Plate X, Figs. A, B and C, show the method perfectly. This system, as well as the manner of transporting materials to the derricks, seems to indicate a penchant on the part of the contractor for seeing things move on wheels, and an inappreciation of the celerity with which cableways can accomplish the same ends. (See Fig. 32.)

Here two cableways are available and are used for nothing but to handle plant, but instead of using them to the limit for that purpose cumbersome alternatives are resorted to with no apparent accompanying advantage. In one detail the Kensico derricks are to be highly commended, namely, the stiff legs are weighted with concrete blocks instead of boxes of spalls.

CHAPTER V

QUARRYING

The term "Quarrying" as applied to the process of producing stone for use in the construction of a masonry dam should properly be explained and qualified somewhat. In many cases this process would be more aptly described as rock excavation and crushing. Methods employed in large permanent quarries for the commercial production of stone for general building purposes are very seldom applicable to the kind of quarries under consideration, because stone within practicable distance of a dam seldom occurs in such deposits or is of such quality as to warrant the opening and working of a first-class quarry.

Assuming that there are a certain limited number of dam sites and also of stone deposits admirably adapted for quarrying, the mere matter of coexistence in space would preclude the likelihood of their occurring together frequently. A possible further reason why they should not is that the location of many streams was fixed by distortions, ruptures, or faults in the earth's surface, and stone is not apt to be found in regular unbroken masses, adjacent to such distortions. However that may be, stone of some kind and quality is necessarily to be found at any possible site for a masonry dam. To obtain stone suitable for the bulk of the masonry is merely a matter of stripping off the top soil and disintegrated rock. It is often impracticable, even where possible, to obtain the necessary percentage of stone suitable for cutting into face stone, coping, etc. (See Plate VI, Figs. A and B.) A failure to appreciate the true inwardness of this fact has occasionally resulted in a much higher cost for the output of the quarry. A brief study will show that it is often advisable to import the cut and dimension stone at a cost apparently high per cu. yd. rather than to employ in the rubble stone quarry the methods which are necessary to save the cut and dimension stone from it. On account of the yardage rather than the quality involved, the same principle operates also in the matter of desired percentage of large rubble stone as distinguished from spalls and concrete material. In a word the entire difference lays in the methods that may be employed for quarrying and handling. For instance, when 60 per cent. to 70 per cent. of the useful output of the quarry must be large stone and a portion of them be suitable for cutting, considerable care must be exercised in the amount of explosive used. The shooting must be hard enough so that the stone may be subsequently barred out of the face or lifted by derricks, but still it must not shatter the rock or open seams unnecessarily.

The methods must at all times be suitable for the production of large stone. They cannot be relaxed upon any apparent deterioration of the quality or formation of the stone because any overshooting may shatter the stone in the next lift or that to be moved by the next line of holes. In this event, much small stone will be produced at the same expense as the massive stone. If loosened in large masses the stone must subsequently be plug drilled and split to a working size. Consequently more men per derrick are required, and the yardage output of the derrick is limited. The recovery and utilization of the large stone interfere with or preclude the use of the most economical methods of loading and removal of crusher material and waste. The reduced output per derrick requires that a larger number of derricks be provided, consequently a more extensive working face and probably more stripping to develop the quarry. On the other hand, different quarrying methods may be adopted if we eliminate the necessity for saving stone to cut for face and dimension work. If, in addition, we reduce by one-half the amount of large rubble stone by substituting concrete, the difference in methods and cost may be revolutionary.

The quarry may then be shot much harder. This will throw the rock down and clear from the face to where it may be more easily handled; and will also break it up so that little or no subsequent work is necessary to reduce it to a size that may be handled. If the methods are determined by the condition that the largest possible percentage of large and shapely stone must be obtained, a certain cost and output result; whereas if the production of large stone is purely incidental to securing an immense output of concrete material, a much lower cost and larger output follow. To anticipate somewhat a proposition which will be developed in the chapter on Probable Future Methods, it may be said that many of the dams of the future will probably be of concrete with no added large stone. In such a case the production of the concrete material will be reduced to the pure and simple proposition of excavating and crushing. The consequent simplification of plant and methods will produce marked reductions in the extent of working face of the quarry, and in plant and operating cost far below anything yet attained. Records of actual differences in cost per cu. yd. between quarrying for large stone and excavating for concrete material might be misleading except in the probably rare case of the two being based on experience under identical conditions in the same quarry. Rock excavation might be closely comparable to average records, but we have seen that the usual quarry contiguous to a masonry dam is apt to be an indifferent or a poor one, and that the quarrying costs consequently are apt to be higher than average quarrying costs. The following quoted quarrying costs in connection with the construction of the New Croton dam perfectly illustrate the principle above set forth.

"Cost of quarrying ($0.841 per cu. yd.) in above table applied to a period when only rubble masonry was laid; when face stones were obtained from same quarry the cost of quarrying was increased from $0.841 per cu. yd. to $1.80 per cu. yd. for entire output of quarry. This included splitting the face stone to within about 3 in. of required dimension."

The foregoing absolute costs are not particularly significant, as conditions obviously are so variable that they would rarely be applicable to another situation. The relative cost, however, is most interesting and pertinent. In another quarry, this relative cost might be more or less. It is not to be doubted also that there would be a similar if not as wide difference between the cost of quarrying for rubble and excavating for concrete material. This difference likewise would vary in amount according to the nature of the quarry.

The construction of a masonry dam usually requires a certain amount of stone before reaching the elevation where the face stone begins. This stone is naturally the first encountered after stripping, while a working face is being developed, and, as it is used for concrete material and rubble, the methods for getting it out may be more of the nature of rock excavation than of quarrying. The character of the stone and of the quarry while thus being developed is naturally a matter of great interest. Prompted by his hopes that the quarry will be able to furnish face stone, one is apt to assign too much weight to two reasons why it may be better the further it is penetrated. These two reasons are: first, that stone is likely to improve in quality with distance penetrated from the surface; second, that less violent shooting than may have been used in development will result in a much more massive product. It is not to be doubted that there may usually be some basis for such reasoning and that there are occasional cases where quarrying for face stone is unmistakably indicated as the proper procedure, but it will more often be found economical to bring face stone from a quarry which does business in that class of stone, even if this means a big transportation cost.

A seemingly promising procedure for simplifying the methods and reducing the cost of production of the two classes of stone is to devote one portion of the quarry strictly to the production of one class and another portion to the other. In reality, this means the opening of two quarries. While this separation might accomplish something, it still implies that a large part of the quarry is of a character to warrant the quarrying for large well-shaped stone, and of sufficient promise of unchanging quality to warrant the risk of delaying the entire work on account of lack of face stone. The fact still remains that more quarry must be stripped, opened and faced up, that more plant must be installed and that much small stone must be wasted or used at greater cost for handling. In short, the entire matter is reduced to the previously stated proposition that the production of face stone should be unmistakably indicated, by an ample margin, before incurring the expense and chance of delay.

Instances of Cost.—The following figures on cost of stone production in connection with the building of the Roosevelt dam illustrate a case where it was most desirable to introduce a large percentage of stone into the masonry, as the cement (manufactured at dam site in a mill erected for the purpose) cost $3.14 per barrel, and the quarry was such that large stone could be produced cheaply. The quarries were at each end of the dam at spillway level. It was necessary, in fact, to excavate the stone to make the waste channels. The stone was a very distinctly stratified, hard, fine-grained sandstone; on a dip of 29 deg. which favored the operation of quarrying. (See Plate VI, Fig. B.)

In portions of the quarry the percentage of waste was not more than 15 per cent.; in other portions it was 75 per cent. or 80 per cent. The division between the cost of producing the stone which was used in the masonry, and of handling the stone that was wasted, is necessarily somewhat arbitrary, because the operations were not sharply differentiated in space or time. Of the 180,700 cu. yd. of waste whose cost is given, about 130,000 cu. yd. came from the quarries, and the remainder from elsewhere. Of the 209,800 cu. yd. of stone used in the masonry, 118,650 cu. yd. were derrick stone (which quantity includes 10,952 cu. yd. which was subsequently cut for the upstream face), 31,150 cu. yd. were spalls, and about 60,000 cu. yd. (solid measurement) were crushed for concrete material. The foregoing quantities represent the first 90 per cent. of the entire job. The wagon haul for the plant and supplies was partly from Globe 40 miles, and partly from Mesa 60 miles.

COST OF PRODUCTION OF STONE FOR ROOSEVELT DAM

	209,800 cu. yd. of stone used in masonry			180,700 cu. yd. of waste, of which 130,000 cu. yd. was quarry waste		
		Total	Per cu. yd.		Total	Per cu. yd.
Plant, assumed as 75 per cent. of first cost delivered on work.	....	$31,860	$0.152	....	$27,225	$0.151
Plant erection, hours labor..	17,800	6,400	0.031	3,957	1,230	0.007
Plant repairs, hours labor...	10,200	4,380	0.021	1,352	475	0.003
Labor, hours..............	553,752	186,884	0.891	221,490	69,118	0.382
Power at 1/2 cent per h.p.-hr.	....	9,975	0.047	....	6,985	0.039
Blacksmiths' coal.........	....	1,480	0.007	....	730	0.004
Explosives................	....	10,285	0.049	....	8,110	0.045
Repair parts and supplies...	....	23,630	0.113	....	7,310	0.040
General expense..........	....	10,560	0.050	....	10,667	0.059
Proportion of office and camp buildings.	......	3,500	0.017	....	3,940	0.022
Total................	....	$288,954	1.377	....	$135,800	$0.725

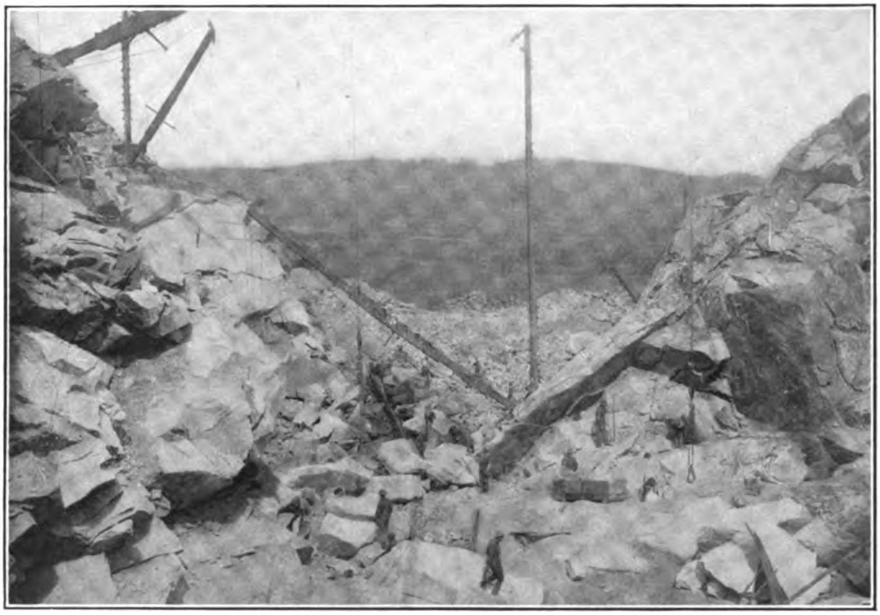

FIG. A.—Granite quarry for rubble Wachusett dam. An attempt was made to get face stone from this quarry but it was abandoned after a few months.

FIG. B.—Sandstone quarry at north end of Roosevelt dam.

(Facing Page 82.)

PLATE VI

FIG. C.—Cross River dam, showing hand force pump for grouting.

FIG. D.—Cross River dam, of cyclopean masonry and concrete block faces. Note temporary pipes for carrying the stream across the site.

CHAPTER VI

FACE WORK

In addition to questions of pure utility, the architectural appearance should be given careful thought in connection with any important structure. This is as true of dams as of buildings. The amount of justifiable additional expenditure is as variable in one as in the other and depends on location, surroundings, client and other circumstances. The matter will not be discussed here except to say that entirely adequate and pleasing effects can be produced at small expense.

Clearly, an overflow dam affords comparatively very little opportunity for esthetic treatment, and this almost entirely on terminal or connected structures. Until recently the exposed faces have been of stone with more or less work bestowed on them to produce the desired courses, joints and faces. Some recent important dams have faces of concrete blocks which can be readily molded of uniform or any required dimensions and appearance. As concrete blocks are artificial stone, the only difference to the builder is the detail of whether blocks are molded or stone cut. Faces formed of concrete deposited against forms are considerably cheaper, but at best they present an appearance somewhat incongruous in a structure with pretensions to beauty. On smaller, and particularly on overflow dams, they may with care be so used as to present a not unpleasing appearance.

Joints in the face work have often been specified to be ½ in. in thickness. Apparently this specification is based on two assumptions: first, that a thin joint presents a better appearance; second, that it is better that the largest possible percentage of face area be stone rather than joint. In general masonry work, and for the sake of appearance, the thickness of the joint may possibly vary somewhat with the size of stone, *i.e.*, rise of courses; but for such massive work where the courses are usually from 18-in. up to 3-ft. rise, and particularly where the stone are left with pitched faces, it will be found that 2-in. joints present a much better effect than thinner ones. Joints are a proper as well as necessary feature of masonry faces, and require no apology. Large stone with bold faces require for congruity that the joint be not obscure.

It is admitted at once that the upstream face should be as impermeable as possible and that stone is more impermeable than mortar; but in the actual every-day exigencies of construction it will be found that a thin joint is very apt to defeat its own purpose. A joint 1 in. or 2 in. thick is more certain to be completely filled than a ½-in. joint, besides being easier both to cut and to build. Another argument will be discussed later. (See Settlement of Masonry.) The preparation of stone and bed and the process of setting are practically the same as has been described for the interior stone, except that one face of the stone must be placed to coincide with the face of the dam. The settling of the stone into the mortar is usually accomplished by pounding with a heavy wooden maul instead of floating it with bars.

One detail of masonry practice, probably all right in ordinary building construction, but which should be looked upon with grave suspicion when applied to the face of a dam is the use of wedges which are inserted in the bed joint at the face to hold the stone to proper line and grade while being set. True the mortar is piled up higher than the wedges and the stone is settled in the mortar until it rests upon the wedges; but the point is that it does rest upon the wedges (else they are useless) and may or may not rest upon the mortar throughout the whole bed. In fact, subsequent slight adjustments of the position of the stone, or other adjacent operations, often operate to break the intimate contact which should exist between stone and mortar. Again, the wedges must be removed. As they may enter several inches in from the face, and as the loosening is usually effected by rapping the outer ends with a hammer, thus rotating the wedge upon the bearing point, the mortar thereby displaced may or may not be adequately replaced by the subsequent pointing. Care should be taken that the mortar is so stiff that it will not fall away from the stone for a greater depth than will certainly be reached by the pointing.

Most of the stone showing on either face of a dam is generally of the class of work known as pitch-faced work, in which the face of the stone has had no work bestowed upon it and shows no tool marks. The line bounding the face is at the intersection of two planes, one of which has been cut down to the uniformity demanded by the specified joint; and the other (the face) shows a rough rock face, *i.e.*, a quarry face modified only by certain spalling off which was accomplished by the pitching tool working at the boundary line. Thus the cutting is done to form the four planes bounding the face, and the area of those planes is usually (assuming average length of stones and average ratio of rise to depth) about four times the area of the face.

Obviously the thinner the joint the more work is required to cut the surface to a plane of commensurate uniformity. For whatever thickness of joint required, it is customary to specify a corresponding accuracy of the planes for a certain distance back from the face. This distance should be one and one-half to two times the depth specified for the pointing. A thicker and less uniform joint is allowable at a greater depth from the face. Of course, the stone should not project beyond the plane enough to interfere with the setting of the stone or the adjacent one.

Among the main requirements of usual practice are these: that the depth of stretchers shall be from one to one and one-half times the rise of the course; that the length shall be not less than two times the rise; that for headers the least dimension shall be the rise, and the length 4 ft. to 5 ft. Sometimes headers are specified in each course, sometimes only in alternate courses. Whether headers occur in each or in alternate courses the spacing should be three or four (depending on length) stretchers between headers. The bond, namely, the distance along a horizontal joint between vertical joints on opposite sides of it, is usually specified to be not less than a certain amount, often 12 in., although this may properly vary with rise of course.

If the rise of the courses is not uniform throughout, appearance of the face as a whole requires that no course should be laid upon or over a course of less rise; also that the decrease in rise should be gradual and uniform. There may be other minor requirements some of which come within the usual understanding of masons and inspectors as to what constitutes "good practice" or "workmanlike manner." There are also many ways of wording clauses of specifications in order to secure work of the character outlined by the foregoing main requirements. For instance, we may specify that a certain percentage of the face area shall be headers. While each of these clauses may be admirable in itself, a combination of them should be scrutinized carefully to see that they do not conflict, or that they are not unnecessarily restrictive. In other words, in any specifications whatsoever, specify the desired standard with as few clauses as possible, and see that they do not conflict nor specify two things.

A case in point was the specification for the upstream face work of a recent important dam. Apparently all the clauses that occurred to the writer of the specification, each of them unobjectionable and often to be met with, had been so combined that if all had been literally and exactly observed, the builder would have been limited to two sizes of stone (a stretcher and a header) in the entire face. In reality the quality of work which the writer had in mind would have permitted quite a departure from such a standard. Such a combination of clauses leaves a prospective bidder in doubt as to how many and which clauses are to be enforced, and can but have a marked adverse effect upon the price bid. This is especially true when it is proposed to obtain the face stone from a still unopened quarry in connection with the dam.

Inclination of Beds.—Some necessary differences in treatment, both in cutting and setting the stone, between the upstream and

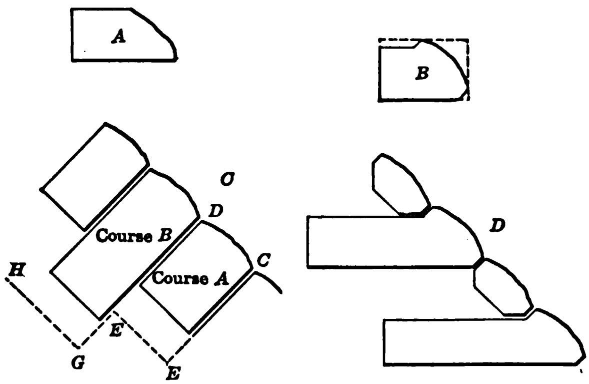

FIG. 20.—Showing different shapes and methods of setting downstream face stone or blocks.

downstream faces of the dam deserve mention. The upstream face of a masonry dam is usually plumb for the greater part of its height. Where it is battered below a certain elevation the latter is usually about one in twenty, or too slight to affect the methods of cutting and setting from those applicable to a plumb wall. Thus for a course of 30-in. rise with the beds kept horizontal, the top pitched line is 1½ in. back from that at the bottom. This distance is not enough to have any practical effect upon the cutting or the size of block required to furnish a given stone, and to have no effect whatever upon the setting.

However, when we turn to the downstream face, with a slope varying between say 45 deg. and practically or quite vertical, we are faced by a situation that calls for a choice between several difficulties. At once the question arises whether the bed joints shall be horizontal or normal to the face of the dam. Let us see what each scheme involves.

If the beds are to be kept horizontal it does not mean that the stones can be cut as in Fig. 20, A, because such an edge is not allowable in stone work; they must be cut as shown in Fig. 20, B, with the bed joints normal to the face for say 4 in. before turning to the horizontal. For a stone of say 12-in. horizontal top bed and 24-in. rise on face of 45-deg. slope, the original block of stone must be of the dimensions shown on two sides by the dotted line, or say about 1 ft. 8 in. × 2 ft. 8 in. In this case about 30 per cent. of the volume of the stone must be wasted, and the expense of cutting is very much increased. For a header of 3 ft. horizontal top bed 23 per cent. of the original block must be wasted. The template for cutting the stones must of course be changed as the work progresses in order to follow the slope of the face and the varying angle between it and the horizontal bed.

If the entire bed is kept normal to the face as was done on the New Croton dam, as illustrated in Fig. 20, C, the result is a regular shaped stone with minimum waste and labor of cutting, but troubles are immediately encountered when it comes to setting the stone. As the backing must be left low enough and far enough back so that the stone may certainly be set without having to cut away anything from either the stone or the backing, one must leave room in excess of the actual requirement of the face stone. The Lewis hole may of course be such that the stone hangs as it is to be set, but as it is being set the rubble backing must be built to such an extent and in such manner as to hold the face stone securely in place against its tendency to slide on its bed.

This process, however conducted, is slow and expensive, and can result only in considerable delay to the work on the adjacent backing for it cannot proceed until the course under construction has hardened sufficiently to hold the face stone up to line. Fig. 20, C, shows how the surface of the masonry must be at certain stages. Thus when starting to set course A the surface of the masonry must exist as from C at the face, to E, thence to F and G; when starting to set a header course B the surface of the masonry must be D–F, G–H.

This necessary procedure in setting such face work merges, without very much awkwardness, with the procedure of building a backing of rubble masonry in mortar, as was the case on the New Croton dam. Both processes are slow and the necessary steps and recesses in the rubble backing are perfectly feasible. When, however, we come to contemplate such face work in connection with a backing of cyclopean concrete, and analyze the various steps in the process, we see that expensive expedients and much delay inevitably result. Soft concrete would not hold a face stone from sliding; even after hardening in the horizontal strata natural to its deposition, some rubble backing would have to be built in connection with setting the face stone. Such rubble backing would not be as effective when laid on the horizontal concrete as it would if laid between the face stone and line E–F, Fig. 20, C. The only feasible method would be first to build the concrete by means of forms to the lines E–F, G–H, and then set the face stone as in the case of rubble backing. This procedure would result in a much less effective bond between face work and backing, and if this effect were to be minimized by keeping the two along at the same elevation, serious delay would result to the progress on cyclopean concrete.

A compromise between the two foregoing methods of radial and horizontal joints, which avoided many of their difficulties, was employed on the downstream face work of the Wachusett dam, as illustrated in Fig. 20, D. Necessarily some stone was thrown away in cutting the headers, but there was only a small waste in the stretchers, which were light stone cut with their natural beds parallel to the face of the dam. Although some rubble backing had to be built in connection with setting the stone, it was only enough to hold the stretchers from rotating, as there was no tendency to slide. No awkward steps or recesses were necessary; and the backing could at all times be kept level at about 3 in. below the face work. The change in pattern for varying face slope applies only to the headers, the stretchers being practically uniform throughout.

The downstream face of the Roosevelt dam (except the pilasters near the top) was formed entirely of selected stone as they came from the quarry, set with horizontal beds. (See Plate VIII, Fig. B.) No work was done upon them except a little with a sledge hammer to square them roughly, or occasionally (after setting) to trim roughly the outer top edge to line. The steps protruded beyond the face, *i.e.*, the re-entrant angles were kept at the neat line. Naturally the joints were thicker, probably up to 5 in., and lacked the regularity of the other face. The general effect, however, is entirely satisfactory.

Concrete blocks are made very nearly as shown in Fig. 20, A except that they have a smooth face. Of course, the considerations of labor and wasted material to produce such shapes in concrete do not apply as in the case of cut stone. The stability while setting does not require any attendant backing up, hence the backing when built may be concrete as well as rubble.

Cost of Face Work.—For a description of the quarries at the Roosevelt dam see page 81. The stone for the upstream face was cut to make a 2-in. joint, no depth from the face specified, and the cost of cutting was as follows:

Stone cutters	27,244 hours	$13,713.45
Stone cutters helpers	15,502 hours	5,097.45
Blacksmith	874 hours	514.00
Blacksmith helpers	874 hours	260.50
Foremen	5,927 hours	3,611.00
		$23,196.40

This force cut the stone for 98,575 sq. ft. of face area, at a cost, including supervision and tool sharpening of $0.235 per sq. ft.; or assuming that with headers it averaged 3 ft. thick, 10,952 cu. yd. at a cost of $2.118 per cu. yd. No proportion of cost of derrick service, plant, power, or overhead charges was distributed to cutting, but is all included in cost of quarrying. (See page 82.)

At the New Croton dam, the granite facing stone was quarried at a cost (previously mentioned) of $1.80 per cu. yd. This figure included the splitting of the stones to within about 3 in. of the required dimensions. The stones were cut for a ½-in. joint for a depth of 4 in. from the face, and beyond this depth the joint might not exceed 2 in. in thickness. Each course was to be composed of two stretchers and one header alternately, the stretchers not less than 3 ft. nor more than 7 ft. long, nor less than 28 in. deep. The headers were to be not less than 4 ft. long, and the face work throughout was to be estimated and paid for as 30 in. thick. The cost of cutting these stones, for a rock face as described above, was $6.48 per cu. yd. The prices paid for the chief classes of labor in the quarry was as follows, for an eight-hour day: Superintendent of quarry $175 per month, superintendent of stone cutting $150 per month, foreman of stone cutters $5 per day, quarrymen $1.50 to $1.75 per day, hoisting engineers $2 per day.

The cost of quarrying and cutting granite for the Pathfinder dam, under practically the same specification as applied to the Roosevelt dam, has been stated as follows:

Labor	$0.91 per sq. ft.
Supplies, powder, etc	0.16 per sq. ft.
Total	$1.07 per sq. ft.

Or assuming it 3 ft. thick equals $9.63 per cu. yd. This figure does not include steel, oil, blacksmith's coal nor plant charge.

The cost of quarrying and cutting stone depends very largely upon the kind and quality of the stone; upon how it occurs in the particular quarry; whether hard or soft; whether or not it splits easily and regularly and whether it can be quarried in large well-shaped blocks with little waste. The cost of cutting further depends upon the thickness of the proposed joint. The cost at the Roosevelt dam was low because every condition was favorable. Thus the regular sheets of very distinctly stratified rock parted easily on bedding planes, the dip was one that facilitated removal from the quarry face and the stone while hard for sandstone could still be cut readily. The costs at the New Croton or Pathfinder dams would apply much more nearly to average conditions.

In the class of stone work above discussed the stones have random lengths, *i.e.*, the length can be varied within liberal limits in order that they may be cut readily and with a minimum of waste from the stone as quarried. Specifying the exact length of the stones immediately places the work in the different and much more expensive class of dimension stone. Since the stones are quarried in a large variety of lengths, the short ones must be wasted or used for something else, while many of the remainder will be so long that considerable must be split or laboriously cut away and wasted. Again, with whatever care the limit is approached, it occasionally happens, in working a stone down to a specified dimension, that the limit is passed and the stone lost through unforeseen splitting from a single effort or step in the process of cutting.

Cost of Concrete Blocks.—The very uniformity in size that makes cut stone expensive makes concrete blocks cheap, *i.e.*, standardizes the forms and reduces their number. Absolute uniformity in size is not necessary to secure a satisfactory effect in pitch-faced stone work; it seems, however, to be more appropriate than random lengths in a face of smooth concrete blocks. In any event, uniformity in size facilitates the process of laying, especially when closing a gap by laying from both ends. When such a closure is made with stones of random length, it is almost invariably necessary to cut part of the last stone to make it fit. The cost of concrete blocks is naturally not affected by the thickness of the proposed joint. The cost per cu. yd. of the blocks depends upon the following factors:

First.—The cost of producing at the block yard, the sand and crushed stone for 1 cu. yd. of concrete, *i.e.*, the cost of say 0.9 cu. yd. of crushed stone (depending on the nature of the quarry, the quarrying and crushing plant and the methods). Assume for illustration that this cost is 75 cents, and that the cost of say 0.5 cu. yd. of sand, either pit or crushed from stone, is 25 cents.

Second.—The cost of the cement, say for a 1–2½–4½ mixture, 1.4 barrels at $1.50 per barrel, equals $2.10.

Third.—The cost of forms, yard for manufacturing and storing and whatever labor-saving equipment may be desirable.

Fourth.—Labor and power.

The third item depends very largely upon the number of cu. yd. of blocks it is proposed to manufacture, and the fourth item depends upon the equipment.

At the Kensico dam with about 60,000 cu. yd. of blocks to manufacture, the yard is 1100 ft. long by 150 ft. wide with several railroad tracks the entire length. Concrete materials are delivered to three mixers mounted upon a traveler which spans three rows of forms at a height just sufficient to clear them. One or two traveling cranes stack the blocks when they are hard enough to be handled, and the cranes also load onto cars the blocks to be sent to the dam. The entire labor cost in the yard of mixing and placing concrete, caring for, handling and storing the blocks has been stated as almost exactly $1 per cu. yd. The total cost per cu. yd. may then be about as follows:

Stone	$0.75
Sand	0.25
Cement	2.10
Labor	1.00
Plant	0.40
Power say	0.10
	$4.60

The above assumptions as to cost of concrete blocks, though subject to change on account of local conditions, are probably fair enough to indicate that in most cases the total cost of concrete blocks would be less than the cost of cutting stone, to say nothing of the cost of quarrying the stone and the effect upon quarry methods, output and cost.

The thinner the joint desired the greater will be the margin of cost in favor of concrete blocks.

Pointing.—Soon after the mortar acquires its initial set and before it acquires its final set, the joints should be raked out to a depth from the face of two to three times the thickness of the joint. The refilling and pointing of the joints should be deferred as long as possible, for it is well known that a mass of masonry continues to settle for some time after being built, and the pointing is more effective if there is little or no subsequent settlement of the masonry. (See Settlement of Masonry.) Unless it be desired to refill loose material against the masonry or to raise the water level on the upstream side, it would be better to let all of the pointing wait until the dam has otherwise been completed for some time. In advance of the pointing, the joints should be thoroughly cleaned of any dirt or loosely adhering cement, and washed out with a hose. The joints should be filled with a mortar somewhat richer in cement than is necessary for the body of the work. The mortar should be used quite dry, say of the consistency of laboratory practice in making briquettes, and it should be thoroughly rammed and caulked into the joint. When the joint is completely filled the mortar should be rubbed down with a convex tool nearly the width of the joint in order to produce a practically flush but slightly concave joint. This last process slightly consolidates the face mortar and gives the joint a neat, accented, finished appearance. If possible the work should be done in cool, cloudy weather; or if exposed to the sun, or to conditions such that the mortar is liable to dry out before properly setting, the pointing should be protected by wet gunny sacks. Water in such quantities as would wash out any of the cement must, of course, be kept from the area which is being pointed.

Owing to the fact that pointing is usually done at odd times, as the weather conditions are favorable or as men may be available from other parts of the work, the cost of the work is difficult to ascertain, and it has rarely been observed with any accuracy. At the New Croton dam all the pointing was left until the end of the work. The joints were ½ in. in thickness, were raked out to a depth of 2 in. and were pointed with 1 to 1 Portland cement mortar. The average cost of pointing 326,500 linear ft. of joints was 5 cents per lin. ft. Wages not stated but probably most of it was $3 per eight hours. One bag of cement was required for every 100 ft. of joint.

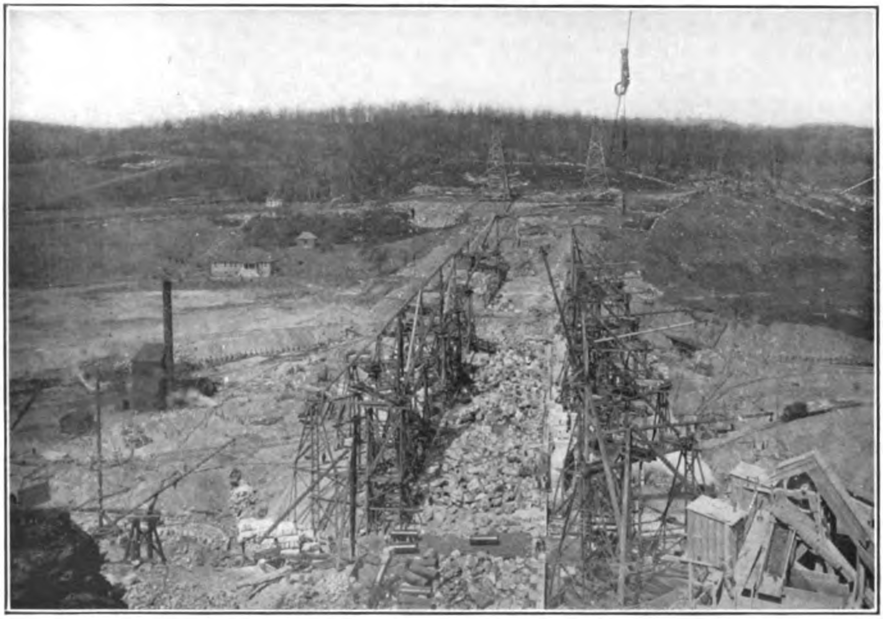

FIG. A.—Croton Falls dam, showing arrangement of derricks.

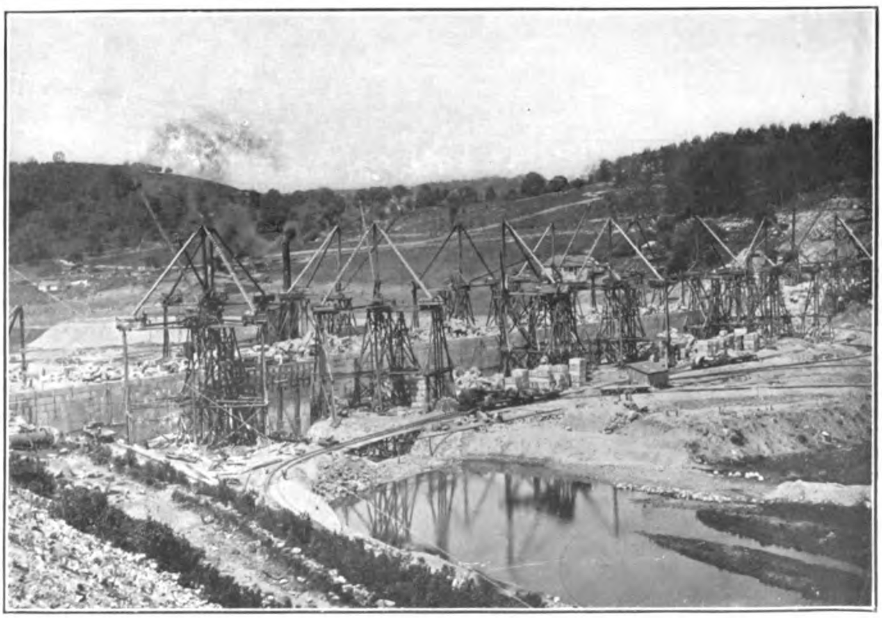

FIG. B.—Croton Falls dam, showing arrangement of derricks.

(Facing Page 92)

FIG. C.—Croton Falls dam, showing preparation of foundation and flume for carrying stream across the pit.

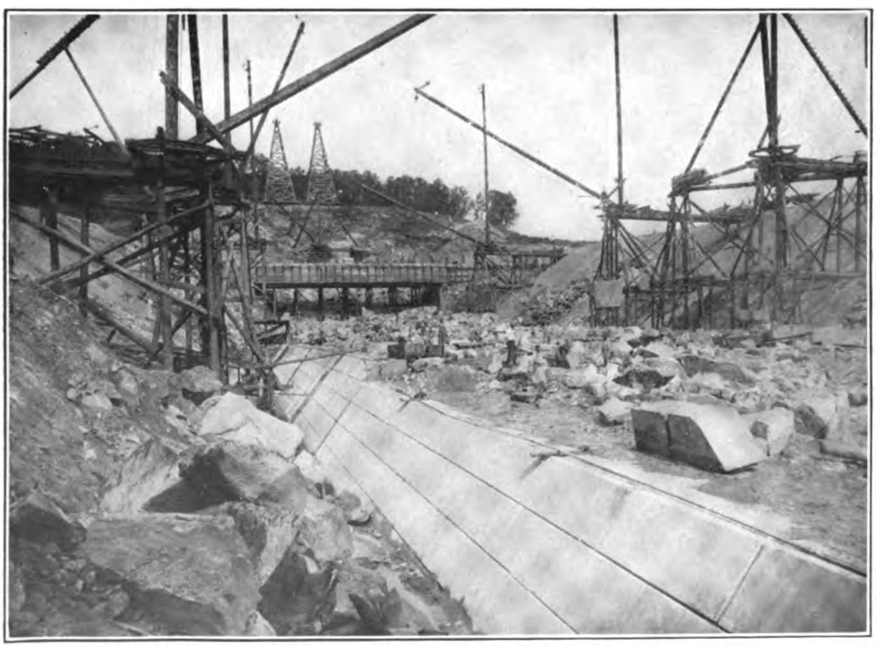

FIG. D.—Croton Falls dam. Cyclopean masonry with concrete block faces.

Care should be taken to rake out the joints to the full depth at the proper time, as the cost is very materially increased if much mortar is left to be chiseled out when the pointing is done. The cost of pointing a thicker and deeper joint would not be in proportion to the size except in the matter of materials, and that is a small part of the total. Probably 10 per cent. to 15 per cent. of the cost is in rigging and moving the necessary stagings. The cost of cleaning out the joint would not be more than proportional to the depth (within range of actual practice) of the joint. Even if some of the raking out had been overlooked and the joint had been left full of mortar, the cost of cutting it out would not be in proportion to the amount actually removed. Then for the cost of pointing a joint 2 in. thick × 3 in. deep, as at the Roosevelt dam, multiply materials by six, labor by two to two and a half, and, for intermittent work add possibly 10 per cent. to the labor.

CHAPTER VII

MISCELLANEOUS FEATURES

Freezing Weather.—When it is desired to lay masonry in freezing weather certain precautions must be observed in order that the masonry shall not be damaged. To the water used in mixing the mortar or concrete, salt should be added in quantities varying with the temperature. (See Fig. 21.) The water and sand should be heated. Steam may be used to heat the water in the measuring tank, and coils of steam pipes may be arranged in the sand bin. On discontinuing work for the night, salt should be sprinkled freely over the fresh masonry. In severe weather the masonry should be covered with tarpaulin and live steam blown in under the tarpaulin.

During warm weather water is freely and necessarily used on the wall for cleaning purposes to temper the mortar and to replace water lost by evaporation; during cold weather this must be practically discontinued and steam hose used in place of the water hose. The only necessary purpose in winter is to clean the stone, and this is best done by a steam jet.

One condition and effect worthy of attention, although insusceptible of practical measurement, is the temperature of the large stone. As the large stone comprise 50 per cent. or more of the mass of the masonry, it is obvious that it makes a difference whether their temperature is zero or freezing. It should take some time after any drop in atmospheric temperature before the stone get as cold as the air. If the stone are used immediately after quarrying they may be much warmer than the air, on the same principle as an earth cut is worked in winter. When the cut has some depth and the work is continuous, the frost is negligible on the working face except on the exposed top, *i.e.*, at the original surface.

Use of Salt.—In connection with the accompanying formulæ for amount of salt in mortar (see Fig. 21), it should be said that at both the New Croton and Wachusett dams masonry work was not started unless the temperature was as high as 20 deg. Fahr., and either rising or expected to rise. Hence the practice on those dams should not be taken as a precedent for work at a lower temperature.

Settlement of Masonry.—Two causes operate to produce a settlement in masonry:

First.—The contraction of volume due to the setting of the mortar or concrete.

Second.—The weight of the masonry results in actual compression. The first cause operates while the cement is setting, the second operates for years.

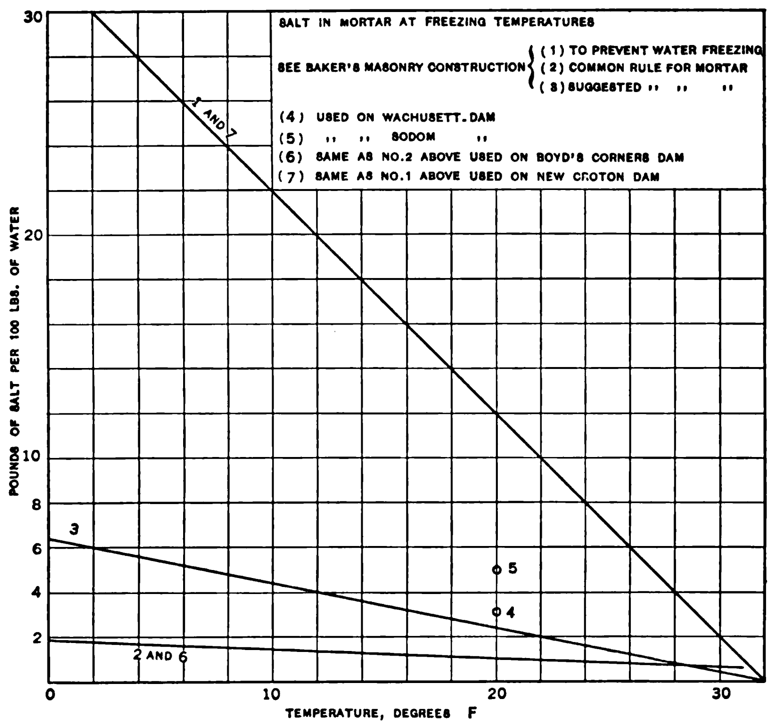

FIG. 21.—Use of salt in mortar during freezing weather.

N. B.—A very fine and noticeable example of the settlement of masonry is seen in the Washington Monument, where the settlement is still going on. It is not necessary to know the original thickness of the mortar joints. One will notice around the base minute fragments which have been spalled off from the corners of the stones. The process may be analyzed as follows: Universally the joint on such stone work is thinner at and near the face than it is generally throughout the interior of the joint. The relatively thick mortar in the center permits a greater settlement than does the relatively thin joint at the face; consequently there is an unequal distribution of the load, more being carried at and near the edge, which tends to spall off the unsupported corners. It is probable that this spalling off is somewhat furthered by a slight creeping of the mortar toward the face. This process will continue until the corners of the stone have spalled off to what might be termed the angle of repose.

Except at or near the surface of the masonry the effect of the first cause is distorted or concealed by the accompanying second cause. In other words, while the mortar or concrete is setting and while it is still in a condition to be easily compressible, a slight superimposed weight will squeeze out the mortar in opposition to any horizontal contraction. The net effect is more settlement than that due to either cause alone.

The objection (see page 63) to overhanging large stone, and subsequently building up with spalls and mortar under the overhang, is that both contraction and settlement operate to pull the mortar away from the stone and to produce a joint less liable to become tight from subsequent operations. In other words, on account of the bonding of the masonry, the load over such a joint would rarely act on the masonry in the joint enough to produce any lateral expansion counteracting its tendency to pull away from the overhanging stone.

As to the effect of the settlement on the face work, it is of course only at the faces that the effect of the long-continued settlement may be observed and measured. However carefully the face courses may be set to line and grade, subsequent testing will show that on the plumb upstream face they are full and low, *i.e.*, have been pushed slightly outward and settled. On the downstream face they have settled and probably drawn in somewhat according to the batter. It may be claimed that the possible effect of such settlement upon the pointing vitiates the argument in favor of thick face joints rather than thin and points to the thin joint as desirable, but an analysis of the entire case justifies the thick joint. Thus we may admit that the less settlement the better, but it must be remembered that this should apply to the entire structure as a homogenity. A 2-in. face joint is much nearer comparability with the thickness of the joints that it is feasible or economical to obtain in the interior work, and it conduces to a condition such that face work and interior work will settle together. Notwithstanding the intimate adhesion of the face work to the interior work and proper bonding by headers in the former, it is entirely conceivable that serious stresses between the two classes might exist in a plane below which were say fifty ½-in. joints in one and fifty 2-in. joints in the other. If the bond between the two classes of work is not effective there will be some separation; if it is effective the result will be that the face work is carrying more than its share of the load, somewhat as in the Washington Monument.

Without attempting to assign a figure to the total amount of settlement in any given case or to the rate of settlement at any time during the total period, it may be safely assumed that the rate is constantly diminishing. If the time element is plotted as abscissa and the settlement as ordinates, the result will be a curve which continually approaches some horizontal line as the limit of settlement. A tangent to this curve at any point indicates the rate of settlement at that time. In the earlier and steeper portion of the curve two rates, say six months apart, will be quite different. It follows then that if a column of masonry, say 50 ft. high, be built and allowed to stand for several months and if a second column be then built alongside of and in contact with the first, the subsequent settlement of the two would be at different rates. The result would be a broken and imperfect contact between the columns, with the settlement of any point in the second column, below that of the corresponding point in the first column. Whether the total settlement at all points in the column is in exact proportion to the height of the point above the base depends on the effect of age on the compressibility of the mortar, the effect of the added weight and the uniformity in rate of building. However, the answer is immaterial to the argument. Now let us see what appreciable effect varying rates of settlement may have in building a high masonry dam.

The exigencies of actual construction often require that progress be made more or less in vertical steps instead of continuous horizontal layers. Then any and all portions are alternately higher and lower than adjacent portions. The derricks at any given time are actually building on one-half of the area while they occupy the other half. A derrick will build up say 10 ft. or 12 ft. above the level upon which it sets, next it will move up onto the higher portion and fill in the depression it came from and then in turn it will carry that portion higher than the first. It is not to be doubted that this procedure results in unequal rates of settlement and more or less distortion in the vicinity of the contact of the various parcels. However, as the differences in height and age are not very large and as the distortion occurs when all the masonry may be called fresh and better able to accommodate itself to distortions, the effect is generally so small as to be entirely negligible or even inappreciable.

It often happens, however, that it is necessary to leave a low place in the dam for a considerable time, during which it is desirable that work on the masonry should be in progress over the greatest possible length of the dam. This necessity will usually be in connection with the river control and should be a most compelling one because such a break in the continuity of the work is to be avoided if possible. At the Wachusett dam two flumes were carried across the work and were maintained until the general level of the masonry was considerably higher than the flumes. A small flume which carried water to supply the Metropolitan District was operated until the permanent pipes and valves were installed and the foundations of the lower gate chamber built. A large flume carried the flow of the river until such time as progress on the dam and a low stage of the river combined to present a favorable opportunity for discontinuing it.

At the Roosevelt dam the river was turned over the north end of the masonry at elevation 9 while permanent gates were installed in the outlet tunnel at elevation 0. Again a gap was left at elevation 106 to carry the flow of the river while repairs and alterations were made to the gates, and while the tunnel lining was being completed. The first gap was 50 ft. below the top of the masonry when the necessity for it had passed, and the second one was 90 ft. Another situation may occur which would in effect result in a gap, or rather in one side of a gap. Namely, when excavation for the dam up the hillsides interferes with plant or with progress on the masonry, there may be a tendency to defer it for a time and bring the dam up instead of extending it. This situation is entirely avoidable as the masonry should run out horizontally to the foundation instead of stepping down to it.

When a gap is left in the masonry, its form (inclination or rack of the sides) is a matter of some moment, the more so the deeper the gap, especially as when a gap is discontinued it is usually desirable, if not absolutely necessary, that it be filled with masonry as quickly as possible. Hitherto accepted practice has been to rack the masonry up away from the gap in a series of steps equivalent to a 1 to 1 slope or flatter if possible, and if the gap is to be a deep one. In the 50-ft. gap above mentioned the rack attained that height in 85 ft. horizontally, the 90-ft. one in 170 ft. horizontally. In the latter case much of the rack was equivalent to 1 to 1, a wide step being left in the middle.

Aside from the fact that the flatter rack is desirable on account of the character of the work, consideration must be given to the question of how derricks can be placed to fill the gap later. If the bottom of the gap is so narrow that derricks cannot set in it and readily build the masonry, it will be very convenient if not absolutely necessary to leave a wide step part way up the rack.

However, assume that a gap has been filled. The result will be that a joint following the rack from top to bottom will open along the junction of the fresh masonry and the older masonry. On the faces the cracks will appear distinctly, showing thin at the outer corners of the rack and thicker in the re-entrant corners. Although the character of the interior masonry is different from the regularly coursed face in that the different layers are bonded together, still no one can doubt that these cracks penetrate the entire mass. In fact, for several reasons, they may be much more pronounced in the interior than would be apparent from the appearance of the finished face because more settlement occurs in the early stages. As the face joints are raked out the crack is not so easily detected and observed, and it is only the settlement subsequent to pointing that causes the crack that shows in the completed dam. It is not argued that such a crack is liable to be a serious matter. Still it is undesirable and should be eliminated if practicable. The following method would appear to be better practice: Make the sides of the gap nearly vertical in steepness and eliminate the steps entirely as they interfere with the settlement, and it is the interference that causes the crack. Make the sides straight, with forms if necessary, so that the intersection of any vertical plane (across the gap) with the side would be a straight line. Of course, any horizontal plane which cuts the side could show as many skew backs or bonds as desired. This would allow the masonry to settle without any rupture and provide ample break in the continuity of the joint. In fact, this scheme would be analogous to the expansion joint introduced in so many recent dams.

The procedure suggested might be criticised in the small particular that it was not adaptable to the usual method of building the faces, *i.e.*, in regular courses. True, if the face courses are stepped up or toothed up the same cause for rupture exists and the same crack will appear at the face, but it will be no worse and the whole interior will be sound. Or, unless the gap is one that could not be foreseen and planned for, pilasters could be built on the face to cover the joint. Building vertical sides to a gap would also result in one great advantage in the direction of not cutting down the area available upon which masonry could be built except for just the width necessary for the bottom of the gap.

Uplift Pressure.—Water from the reservoir enters under pressure the masonry of a dam and also the rock foundation underneath. Under certain circumstances it may produce an uplift pressure sufficient to affect seriously the design of the dam. The proper solution of the problem will be a matter of debate as long as certain engineers are more conservative than others. Again, the nature of the solution will depend more upon the engineer's natural cast of mind than it will upon the circumstances of the particular case.

Whether or not prevention is better than cure; whether to eliminate the pressure by taking means to prevent the entrance of water or to counterbalance the pressure by adding masonry, will be decided largely according to the degree to which the engineer is willing to trust his reputation to his judgment. The limit of possible pressure is known, *i.e.*, full reservoir head applied to the entire area of the plane under consideration. Further, an entirely adequate remedy lies in making the masonry section heavy enough. That safe and expensive method will always have its advocates. Not only is this an entirely adequate remedy but, if adopted, the necessary masonry section is susceptible of precise determination by the designer in advance of any data obtained during the preparation of the foundation and entirely independent of the judgment of any person as to nature of foundation or quality of masonry. Some reason and excuse for the conservative design lies in the fact that the judgment of the constructing engineer is often a questionable or unknown factor. It should at least always be possible to secure the services of a man of competent judgment to oversee and advise, and of a resident engineer who may be depended upon to carry out the prescribed measures faithfully and intelligently.

A rational view of the entire situation has been obscured by various loose and confusing ways of thinking about and stating the conditions and by a misapprehension as to just what it is desirable and possible to accomplish by certain preventive measures. For instance, there is the old familiar and perfectly correct proposition that any amount of leakage, no matter how infinitesimal, may produce a pressure equal to the reservoir head. In conjunction with this proposition it is pointed out how practically impossible it is to prevent some leakage through either masonry or foundation. Whereas the only pertinent conjunctive statement is the equally true one, namely that be the leakage "to" a certain point or plane of application a drop, a gallon or a thousand gallons per minute, day or year, the pressure at that point can be only that which is necessary to force such quantity away to an outlet at an equal rate. In other words, if leakage "to" exceeds leakage "from," pressure will result; if leakage "from" is relatively free and easy, pressure is impossible. Substitute money for water and affluence for pressure and we have Micawber's formula for happiness or misery.

Another source of error lies in considering a dam (masonry or foundation or both) to be a homogenous mass, or rather of equal permeability throughout; also in considering such a condition desirable and attempting to realize it even approximately. We may indeed treat an entire foundation in such a manner as to cut off 99 per cent. of the leakage, but when it has been accomplished we do not know whether the leakage has been cut off at upstream side, centre or downstream side, and the remaining 1 per cent. may have just as much destructive effect as the original 100 per cent. It is the same with the masonry. We cannot exclude the last drop of water, and in proportion as we succeed in making every part equally impermeable we succeed in making it uncertain where the water is cut off and whether or not any pressure can exist. We may realize a very high degree of impermeability but if we utilize the entire width of the dam in doing so we have accomplished absolutely nothing in the way of limiting or controlling the pressure. The actual amount of the leakage (barring streams of such a size as to cause erosion) is of small moment compared to the location of the limiting section of the channel upstream from which the pressure exists. With the problem thus stated it will be seen that the solution lies not in aiming for a tight dam or foundation but for a dam or foundation one part of which shall be much tighter than the remainder. One condition is uncertain and impracticable if not impossible; the other condition is perfectly feasible and certain of realization. We may be uncertain as to absolute tightness but we can be certain as to relative tightness. Possible horizontal seams below the masonry present no difficulties but rather tend to facilitate a satisfactory treatment.

In connection with the subject of uplift pressure some observations as to actual existing pressures will be of interest, although naturally they will settle no controversies. Indeed, arguments in support of widely differing views may be drawn by different engineers from the same observations.

In *Engineering News* for July 31, 1913, is an abstract of a paper which appeared in *Zeitschrift für Bauwesen*, Vol. 63, 1913, page 102, giving some observations as to pressure under two German dams, the Oester and the Neye. The Oester has a maximum height of 131 ft. and a thickness of about 90 ft. Upon each of four sections across the dam three pipes led from the foundation to galleries where a pressure gage was applied. The foundation was of graywacke and shale varying from firm to rotten, with some clay crevices. The stratification was nearly in vertical planes, up and downstream, *i.e.*, crossing the dam. Excavation in the bed rock was 13.1 ft. to 14.8 ft., with no sign or indication of a cut off trench and no mention of any special treatment like grouting. The gates were closed Feb. 16, 1907. On May 6 water was going over spillway and pressures were observed.

The observations indicated that pressures existed as follows: Section A–B on one side of the bottom of the foundation; full head at heel to one-half at toe; section C–D next in order, full head at heel to about one-fourth at toe; section E–F next, three-fourths of full head at heel to about one-fourth at toe; section G–H at the other side of the bottom of the foundation, from full head at heel to zero at toe.

Observations in 1910 showed in most cases a reduction in the pressures, although in one pipe there was a slight increase and in one (the downstream pipe of section G–H) a considerable increase to about one-fourth full reservoir head. Holes 19 ft. from heel showed in 1907, 85 per cent. of full head and in 1910, 71.2 per cent. In one of the crevices, which had been excavated about 8 ft. to 13 ft. lower than the general foundation, a spring (just upstream from the upstream pipe of section G–H) had been piped. This spring ran 19 gal. per hour with reservoir empty and 870 gal. per hour with reservoir full.

The foundation of the Neye dam was about 26 ft. below original surface in graywacke and shale which was on an inclination of 15 deg. to 35 deg. toward the downstream side, and likewise without cut-off trench. In February, 1910, after the reservoir had been full for some time, pressures were observed in nine pipes, water in the reservoir being 102 ft. above the general level of the foundation. The two lines agreed very closely, the pressure at 14 ft. inside the heel being about 57 per cent. of the total and decreasing to about 32 per cent. near the toe.

An erroneous conclusion was drawn from the Oester observations: "That deeper foundations conduce to decrease in pressure as more solid and impervious rock is reached."

Section G–H was deeper than section A–B and the other two sections came between, both for elevation and pressure. However, the effect of the tapped spring at the upper end of G–H would account for the difference in pressure. We also see a confusion of ideas in that pressure is considered dependent upon quantity of leakage whereas there is no necessary relation between the two. From the observations on the two dams two conclusions were drawn:

First.—"That full static pressure does not act over the entire base but the resistance, which the water encounters in its flow toward the downstream side, gradually uses up the head and makes the pressure at the toe less than at the heel." Precisely what was meant by this is not clear; probably something indicating an analogy to the gradient taken by water flowing through a homogeneous material. This analogy if it exists at all is entirely too precarious and accidental to deserve any weight whatever.

Second.—"Deep foundations are conducive to decreasing the uplift pressure." Here the deeper foundation and less pressure at the Neye was a coincidence, and is made the basis for an unwarranted conclusion in connection with the fictitious (from ignoring the effect of the spring) result at the Oester. The conclusion is unwarranted in any case but all the more so in this from the ignored fact that at Oester the stratification was vertical and that at Neye it approached the horizontal.

Deep foundations conduce to a decrease in quantity of leakage. Pressure, however, depends upon the extent to which any amount of leakage is confined.

To state the case again. The head at any point (A) under a dam subtracted from reservoir head gives a certain difference in head which is required to force a certain quantity of water from the reservoir to point A. The head at point A minus the head at the toe gives the difference in head required to force the same quantity of water from A to the toe. It is seen that the head at A does not depend upon the quantity of leakage but upon the relative freedom of the channels upstream and downstream from A.

If the difference between A head and reservoir head is small compared with that between A head and head at toe, the tightest position of the channel is downstream from A. Read large instead of small in the above sentence and the tightest part of the channel is upstream from A. The entire treatment of a foundation should have for its aim the location of point A as near the heel of the dam as practicable, and the making of the difference in head upstream from it as great as possible and the difference in head downstream from it as small as possible.

An observation at the Roosevelt dam must be prefaced by considerable explanation. The foundation was a hard fine-grained sandstone, the strata dipping upstream at an angle of 29 deg. and at right angles to the dam. Some seams carried water but the widest seams were filled with a material resembling clay (so fine as to betray no grit whatever) and were absolutely watertight. The issuing water generally was at the river temperature but at two or three points the water was considerably warmer (110 deg. Fahr.). These warm springs were piped together and led to the basement of the power house at the toe of the dam; the other springs were grouted.

The rock was so sound and suitable that it was not necessary to excavate much. Over portions of the bottom practically none was excavated and the average for the entire area was less than 5 feet. Just inside the heel of the dam and extending across the canyon was a fault in the rock with a displacement of about 8 in. Part way up the sides of the canyon the dam curved away to the downstream side of it. Where the fault came inside the masonry lines advantage was taken of it to form a cut-off trench or V 6 ft. to 8 ft. deep. The rock on the upstream side, being solid, was left in place and the rock on the downstream side readily broke to the fault with a minimum shattering or disturbance of the foundation. The elevation of the bottom ranges from −20, to −35, with a lowest point of −38, elevation 0.0 being the original low-water elevation in the river. The exact location of the warm springs was not made a matter of record, but a close enough approximation would be 50 ft. or 60 ft. from the heel of the dam. Previous to any work upon the dam, warm springs existed on the river bank a mile upstream from the dam. As these were covered by the slowly rising waters of the reservoir, warm springs appeared 6 miles upstream from the dam where none had been observed before. During the construction period the water in the reservoir was held for a year or more around elevation 100. During that time warm springs appeared between 400 ft. and 500 ft. downstream from the dam. Water of the same temperature (110 deg. Fahr.) emerged from crevices in both canyon walls at elevations between zero at river level and 20 ft. above, and very likely also under the river. The streams, visible above river level, although scattered so as to be difficult to estimate, might have amounted to 2 sec. ft.

Three years after the dam was completed the pressure was observed on the pipe leading from the warm spring under the dam. Water in the reservoir was at elevation 115. The pressure gage was put on at elevation about 20. It was kept on for one day and read 14 lb. One can hardly do more than speculate upon the significance of this observation as the nature of the connection between the reservoir and the warm springs is shrouded in uncertainty. Probably there is some connection; but it must be remote and indirect or the great difference in temperature would not persist between adjacent springs. The observation was an afterthought as the spring was piped into the power house simply to furnish a supply of warm water.

The area over which the pressure is effective is as important as the pressure itself so that it is only with caution that one may assign areas as being applicable to certain pressure observations. In order to form a reasonably safe conclusion regarding the pressures under a dam there must be available observations at a large number of points, and the observations must be consistent.

However interesting it may be to speculate upon the structure of the rock foundation and the conditions under which water may pass through it from the reservoir, the proper course is to control those conditions instead of being controlled by what we imagine they may be. The entire solution is in the line of realizing the condition, elsewhere stated as desirable, that a masonry dam should properly be two things: first, a zone as nearly water tight as practicable; second, a structure for the support of that zone.

Two requirements must be met to eliminate uplift pressure as completely as possible, whether in dam or in foundation: first, the upstream portion should be tight; second, the downstream portion should be relatively open. In the foundation the first requirement is met by the cut-off trench, so called, under the heel of the dam. As a general proposition it may be assumed that seams diminish in number and width as bed rock is penetrated. Therefore a cut-off trench of whatever depth filled with masonry will intercept and prevent leakage, except possibly such minute quantities as would readily escape through the seams downstream from the trench. To extend this tight zone and intercept leakage at greater depths than would be economical, necessary, or even attainable with the cut-off trench, grout may be forced into the seams through holes drilled for the purpose. The process, which has been described elsewhere, is simple. If conducted with care it should be most effective.

Drainage of Foundations.—To accomplish the second requirement, noted in the preceding paragraph, *i.e.*, to facilitate the escape of whatever water does leak by the cut-off trench, it has become the practice in recent years to lay drains under the dam from the cut-off trench to an outlet below. The best practice would be to start the drains from the downstream side of the *bottom* of the cut-off trench rather than from any point further downstream in the path of the water. This may be argued as follows: Water passing the cut-off trench conceivably might exert some pressure in coming up the downstream side of the trench in its search for some outlet seam. Then when it enters a seam it might be that the tightest part of its entire channel is downstream, at or toward the toe of the dam, in which case there would be a resulting uplift pressure. To start the drains from the downstream side of the bottom of the cut-off trench would be the cheapest and most effective method of draining the foundation to that depth. In case it is desired to extend the open zone to a depth corresponding to that of the tight zone produced by grouting, holes may be drilled and left open with free connection with the drainage system. In several recent dams provision has been made for the future drilling of such holes should it be considered desirable. To be certainly effective such holes should be drilled only after all grouting operations have been completed in the adjacent tight zone.

It need hardly be pointed out that with any or no drainage system the uplift pressure on the bottom of the dam cannot be less than that due to the head from any backwater that may stand at the toe of the dam. In studying any results obtained from testing holes in the foundation (as described on pages 19 to 23) and in devising and carrying out the entire treatment, it should be borne in mind that the weight of the masonry will tend to close or reduce any horizontal seams in the foundation; also that after the dam is built there is a redistribution of pressures consequent upon raising the water behind the dam.

A drainage system like that outlined above, if intelligently devised and faithfully constructed, will so certainly prevent uplift pressure as to eliminate that question entirely from the realm of practical consideration.

Drainage of Masonry.—A feature in the design of several recent dams (Elephant Butte, Arrowrock, Olive Bridge, Kensico, Farnham,

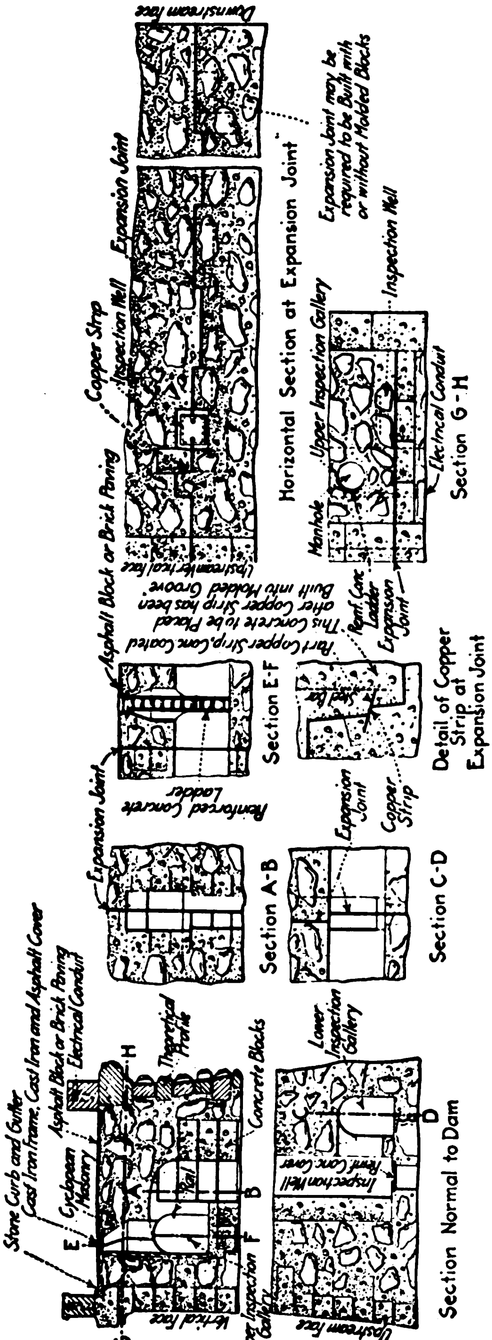

FIG. 22.—Details of expansion joint Kensico dam.

Barker, and others) is a system of drainage in the masonry near the upstream face to intercept and remove any water which may enter the masonry. (See Figs. 22 and 23.) Leading to an open drainage and inspection tunnel are many channels made by building in the masonry vertical chimneys of very lean (and hence readily pervious) concrete blocks. Such a system, like the one in the foundations, establishes, just downstream from the tight zone, an open zone for facilitating the escape of water to an outlet under the refill at the toe.

Nearly all masonry dams have no such drainage system. While a dam with absolutely no leakage is so rare a thing that it is held

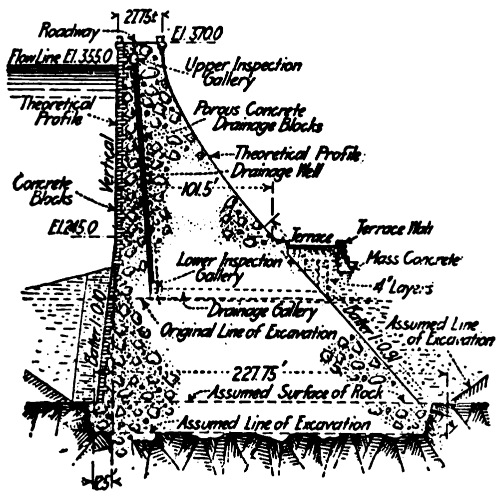

FIG. 23.—Cross section Kensico dam.

by many to be non-existent, still very few of them show water upon their downstream face in any objectionable quantity. Even if the system is not expensive, it requires some attention, will probably interfere somewhat with progress on adjacent masonry and is certain to be regarded more or less as a device to hide inferior construction, all for the matter of an appearance which can be kept so slight as to be insignificant. Even if the appearance of the downstream face is such as to suggest the use to which the dam is being put, there is vastly more precedent for it than there can be objection to it.

Previous to the advent of cyclopean concrete it could have been said of masonry dams that conditions are such, or can easily be made such, that no apprehension need be entertained regarding a possible uplift pressure on a horizontal joint in the masonry. The masonry of the upstream face usually is or can be made so much tighter than the remainder that any water which penetrates the dam can go on through without causing any pressure. The upstream face of a dam is 90 per cent. to 95 per cent. stone as compared with 50 per cent. for the interior, and the pointing of the upstream face should be much tighter than the mortar or concrete of the interior. Then through the mass of the masonry there should be nothing analogous to a continuous joint or seam such as may exist in the foundation.

It is significant that such drainage systems made their appearance very soon after the advent of cyclopean concrete. To date they have been introduced only in dams of cyclopean concrete with faces of concrete or concrete blocks. It is quite possible that the first one or two cyclopean concrete dams, built while the method was somewhat experimental, may have been accompanied by results which indicated that the drainage system was a necessity. Whether or not the drainage system was indicated by experience, its desirability in connection with cyclopean concrete may be reasoned as follows:

First.—Cyclopean concrete may and should be less permeable than the mortar formerly used.

Second.—The upstream face, if of concrete or concrete blocks, is more permeable than the stone formerly used.

Third.—Thus in both respects the difference between the upstream tight zone and the downstream relatively open zone is reduced; and while the total leakage may be no more, or even less, it is accompanied by conditions more conducive to the existence of an uplift pressure.

Fourth.—In addition to the foregoing it may be fairly said that the building of cyclopean concrete tends to the introduction of horizontal construction planes with less bonding across the planes, thus furthering the application of an uplift pressure and making it more effective.

Fifth.—One recent feature of dam design, the expansion joint, which will undoubtedly persist, seems to require that an intercepting drain be introduced immediately downstream from the upstream recess where the metal strip crosses the joint. While a drainage system in the masonry was entirely unnecessary for the dams built under the older specifications, still such a system appears to be a very desirable feature for future dams of cyclopean concrete or concrete, and it will undoubtedly become standard practice.

CHAPTER VIII

MISCELLANEOUS FEATURES (Continued)

All long masonry structures which are exposed to appreciable changes of temperature will contain temperature cracks. In a masonry dam they manifest themselves principally at or near the top where the section is comparatively thin; if they extend down the faces to where the dam has a thickness of 30 ft. or more it seems to have been demonstrated that they seldom extend entirely through.

Cracks may show on the surface down where the dam is much thicker but they seem to be analogous to cracks produced in mud from drying out in the sun, namely, the cause operates with diminishing force as the mass is penetrated and the resulting crack diminishes in thickness until it dies out entirely. Along the top of a dam of uniform cross-section the cracks may be expected to appear at fairly uniform intervals of 30 ft. to 40 ft. Even a slight change of section such as produced by a buttress or pilaster or an abrupt change in elevation of the foundation, will fix the location of the crack at that point, and probably tend to localize the effect, *i.e.*, make that crack wider than adjacent ones. A crack may also be started as a settlement crack and continue as a temperature crack.

Effect of Temperature Changes.—Temperature cracks in a dam need never be regarded as a serious matter. As above stated they exist chiefly in the upper and thinner part. In a dam without overflow that part of the crack below water level is thinnest and is small in extent. The water against the upstream face of the dam limits to a very large extent the range of temperatures acting on the masonry under water and consequently limits the thickness of the cracks. Exposed to the sun and atmosphere the temperature of the face masonry might range during the year 130 deg. Fahr. or 140 deg. Fahr., while water in the reservoir might range from freezing to say 70 deg. Fahr., or far less than half as much.

Temperature fluctuations diminish rapidly in extent as the mass is penetrated for the thicker the mass the nearer the temperature of the center comes to being constant at the annual mean; even so the effect of the water is most beneficial. An overflow dam is usually thinner at the top, and in that respect is more susceptible to cracks, but the effect of the water is exerted at the top and (if the water is flowing over) also over the top and down the other face. A temperature crack is in a vertical plane at right angles to the line of the dam. Hence even if water did penetrate some distance into the dam it could exert pressure only in such a direction that no harm could result. The worst that can be said of a temperature crack is that it allows the water to penetrate the upstream face masonry which it is desired to keep impervious; but at any such depths as are necessary for any considerable pressure temperature cracks are insignificant if indeed they exist at all. In a curved masonry dam the pressure of the water tends to keep the dam in compression and consequently to close the cracks. Indeed the temperature variations in length of the dam are largely taken up by deformation of the arch, the radius of the curve being longer in cold weather and shorter in hot.

Leakage—Expansion Joints.—Practically all masonry dams show leakage upon their downstream face; in fact only in the case of one curved dam is it claimed that absolutely no leakage can be detected. Some dams show only a slight sweating, which may disappear on a dry sunny day, while others may leak quite appreciable streams.

Leakage may result from temperature cracks or by percolation through the body of the masonry. In all cases, probably, it results from these two causes although in different and unknown proportions. Speaking generally and with some caution, it might be said that in dams of rubble masonry the leakage is somewhat more attributable to percolation or to the presence of numerous and minute cracks; while in the more recent dams of concrete or cyclopean masonry it seems to be due to a smaller number of wider cracks. It was probably this localization of cracks observed in the earlier cyclopean dams that led to two features of design, namely, longitudinal reinforcement near the top and expansion joints. The latter seems to have become standard practice, in theory at least, even if the details are yet subject to improvement. Actual figures regarding effect of temperature changes, and leakage, are quite scarce, and what follow are taken from a paper by Charles S. Gowen (with discussion by George C. Honness, Thaddeus Merriman, William L. Brown and Charles S. Gowen) on "The Effect of Temperature Changes on Masonry" which appeared in the *Transactions* of the American Society of Civil Engineers, Vol. LXI.

The Boonton dam is 2150 ft. long, with a maximum height of 114 ft. It is constructed of cyclopean masonry approximately in the proportions of 50 per cent. stone and 50 per cent. of concrete the average mixture of which was 1–2¾–6¼.

"Across the entire top of the dam extend thirty-three cracks, seventeen of which total 2½ in. in thickness, and the remaining sixteen average $\frac{1}{32}$in. each. In addition to the above are thirty-three more which do not extend across the top but are confined to one-half or less of the crest width, and with a total thickness of ½ in. Thus the total width of all the cracks was 3½ in. and the largest ones could be traced down the face of the dam for a distance of 60 ft. from the top. The larger cracks occur at very regular intervals of about 100 ft. except near the ends of the dam. Through the main cracks is considerable seepage, naturally greater in the winter than in the summer. Through the two principal cracks, one near each end of the dam, the seepage was measured on March 17, 1908, and found to be 23,000 gal. per twenty-four hours."

From a somewhat incomplete series of thermophone observations on the temperature of the interior of the masonry Mr. Merriman deduces, subject to future verification, the following formula for range of masonry temperatures between depths of 0.5 ft. and 20 ft. from the surface. If R be the total range in temperature at any point in the mass, in degrees Fahrenheit, and D be the distance in feet to the nearest face of the dam, then

$$R = \frac{135}{3\sqrt[3]{D}}$$

where 135 is the total atmospheric range observed during the period.

In the New Croton dam were five large cracks, one measuring ¼ in. while the others averaged ⅛ in. each. In addition were a number of smaller cracks, but their number and size were not determined. Mr. Gowen presents some interesting curves showing the variation in width of cracks corresponding to changes of atmospheric temperature. One or two of the principal cracks were treated with grout and later caulked with lead. Since then one of them, which extends down the face of the dam for 70 ft., has shown in winter some spraying seepage at certain elevations. During warm weather this seepage reduces to a sweating or dampness too slight to create a flow.

In the Cross River dam (completed in 1907) the upper 30 ft. was reinforced with 1¼-in. square twisted rods or rods of equivalent area. No temperature cracks were discovered during the winter of 1906–07. Work was resumed on the dam in March, 1907, and it was completed by the end of August. Thus that part of the section of the dam which is affected by temperature cracks was constructed during warm weather. During October, 1907, four cracks were discovered, which by the following March had developed so that three of them extended down the face of the dam for 70 ft. from the top and the fourth for 42 ft. The aggregate maximum width of the cracks was $\frac{7}{8}$-in. The dam was apparently cracked entirely through for a distance of about 43 ft. from the top at which elevation the section is 31.75 ft. wide.

"No leakage or seepage through the dam or through the cracks was noticed until January 16, 1908. On this date the water stood at a point 53 ft. below the top of the dam, and the leakage was noticed at a distance of 70 ft. from the top of the dam." An illustration "shows the crack at Station 14 89, which extends through a header block. A slight amount of leakage continued until February 7, when it suddenly increased, indicating that there was a free passage for water through the dam section at the cracks. On February 10, a measurement showed a leakage of 6.6 gal. per min., and by February 17, this had increased to 23.9 gal. per min., which is the maximum amount of leakage measured at any time. As soon as the marked increase in leakage was noticed, steps were taken to control it. This was accomplished by caulking the crack at the upstream face with lead wool and grouting the crack. These measures were successful to the extent of reducing the leakage to 4.5 gal. per min., and this has continued to diminish since the warm weather set in."

The Assouan dam contains many abrupt changes of section, both on account of considerable steps in the foundation and from the sluices through the masonry. While much of it was built during very hot weather, the annual range appears to be no greater than at the Boonton dam. Six or seven cracks appeared, the largest occurring at one of the most sudden changes of section and extending from the top of the dam down almost to the rock foundation. It was about $\frac{1}{4}$ in. thick at the bottom in cold weather, rather more at the top, and extended through the dam from side to side. The leakage amounted to only about 5 gal. or 10 gal. per hr. and decreased very much after the reservoir had been full for some time.

The paper from which the foregoing is abstracted contains progress profiles of the New Croton and Boonton dams showing the location of the cracks. A study of the cracks shows, as would be expected, that the number and extent of the crack is governed largely by the atmospheric temperature during the construction period as compared with the winter temperature. The conclusion is most plain that in order to minimize the cracks the masonry built during hot weather should be covered with a thick layer built during cool or cold weather. In short, construction should proceed steadily as far into the winter months as practicable, and the top of the dam should be finished during a period of low temperature.

At the Roosevelt dam permanent provision was made for keeping the exposed faces wet during hot weather after the completion of the dam as well as during the latter part of the construction period. At this dam the minimum temperature would be not far from 20 deg. Fahr. and the maximum in the sun certainly over 160 deg. Fahr. A pipe, 2 in. reducing to 1 in., was run along the downstream face about 25 ft. below the top. A small hole was bored every 10 ft., sufficient to keep the face of the dam continually moist. (See Plate VIII, Fig. B.) The pipe was supplied with water by gravity from a canal slightly above and independent of the reservoir level although not high enough to cover the crest of the dam. Through another pipe water can be supplied by pumping to serve similarly all of that portion above the gravity system, including the roadway on top. The thin film of moisture on the face of the dam was not intended to convey to the masonry the temperature of the water, but to produce an even lower temperature from the effect of the rapid evaporation which is a feature of that arid climate. The temperature of the face masonry under these conditions should be many degrees, quite possibly 100, below the maximum that it would otherwise attain. The top of the dam was constructed, except for portions of the concrete roadway and stone parapet walls, during the summer of 1910. Above the level of the gravity system, it had not received such a beneficial effect from sprinkling. During the latter part of October cracks averaging perhaps $\frac{1}{16}$ in. in thickness appeared across the top at quite regular intervals of 40 ft. to 50 ft. In but one case (and that at a pronounced change in section) did the crack extend more than a few feet down the faces. After having tried reinforcement in the tops of the Cross River and the Croton Falls dam, without however much apparent effect on the cracks, recent practice has been to govern the number and location of cracks by introducing expansion joints (or rather to substitute joints for cracks). Expansion joints are provided at intervals, varying from 40 ft. to 80 ft. in different dams. They consist of smooth joints constructed across the dam, the first face constructed being covered with a plastic waterproof material and the second being built against it. See Plate XI, Figs. A and B, also Fig. 22. At one or more points along

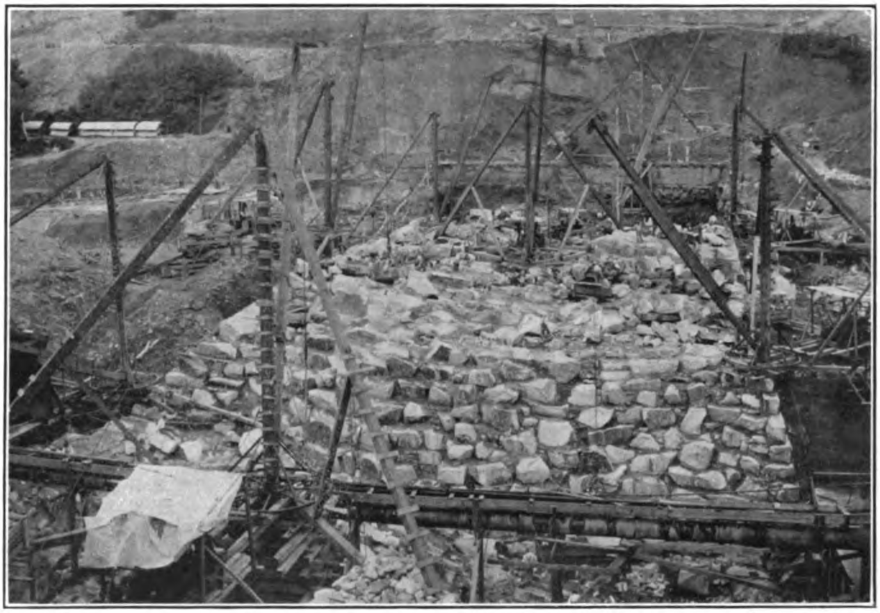

FIG. A.—Rubble masonry Wachusett dam. Small flume across far end of pit

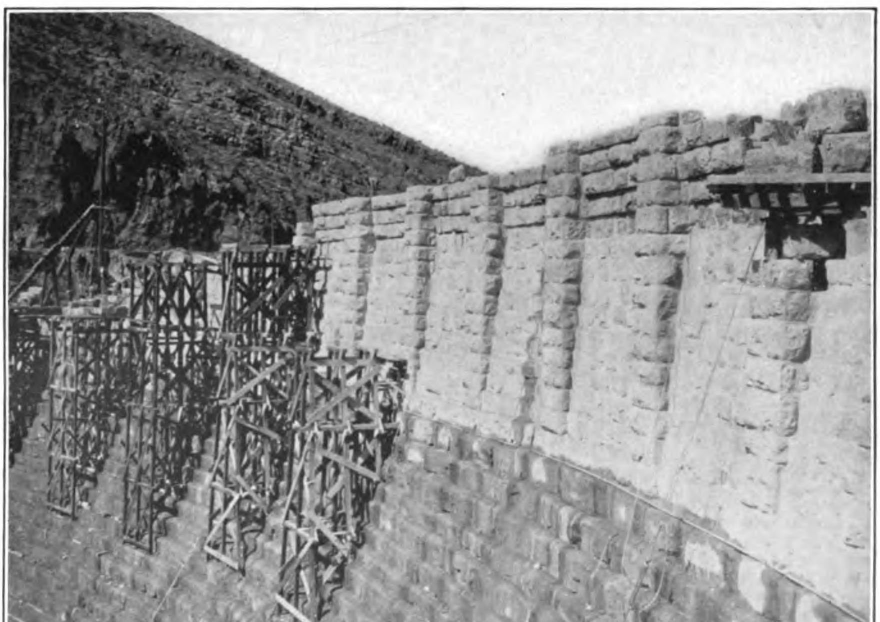

FIG. B.—Roosevelt dam near the top, showing landing stages and supports for derricks; also the two-inch water pipe for keeping the face of the dam wet.

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PLATE VIII

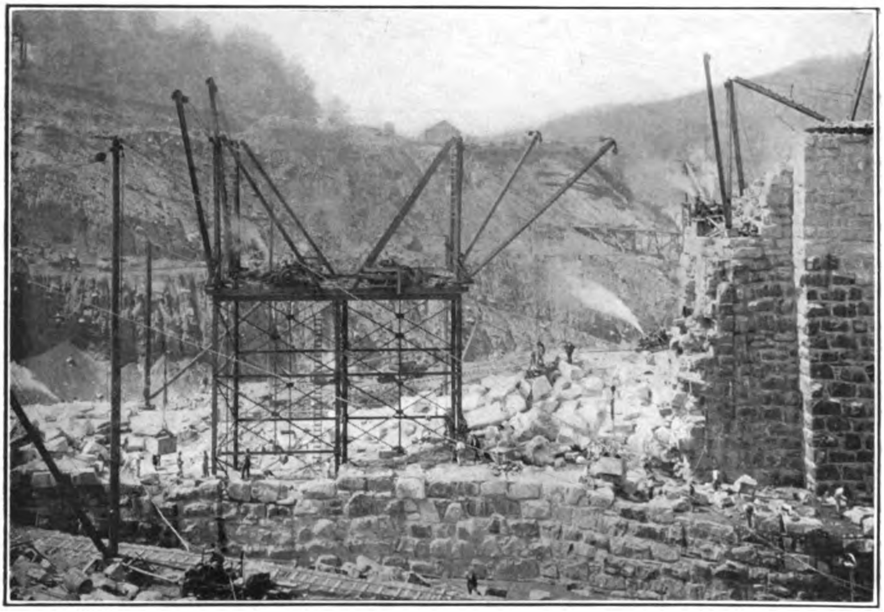

FIG. C.—New Croton dam. Steel towers on which derricks were mounted.

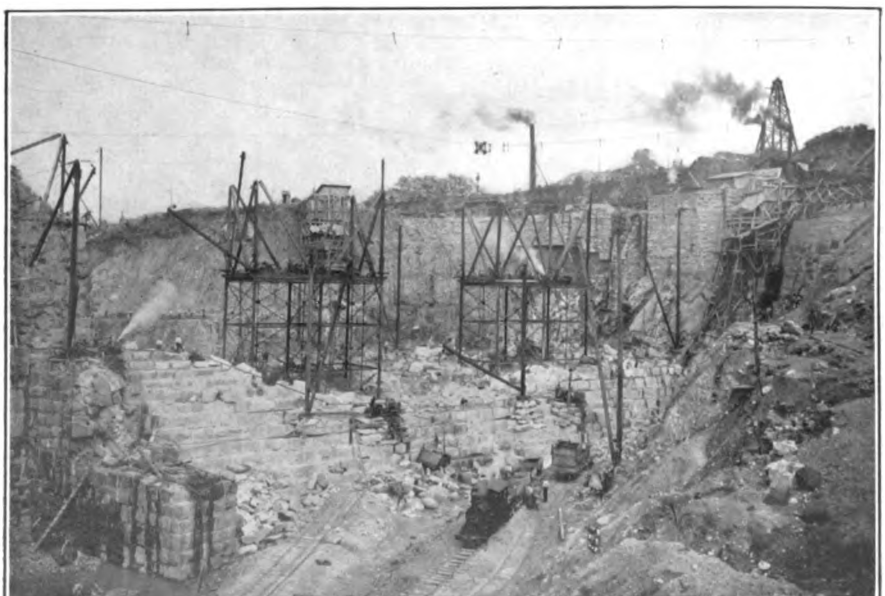

FIG. D.—New Croton dam. These steel towers were built into the masonry.

this joint the masonry upon one side contains a rectangular recess which is filled by a corresponding projection in the masonry of the other side, forming rectangular bonds, such that movement in the direction of the length of the dam will not tend to open or close the joint upon two sides of the rectangle. In the Kensico and Farnham dams a metal strip crosses the joint with an edge embedded in the masonry upon each side. The joint immediately downstream from the metal strip is intercepted by the drainage system previously described. (See pages 106 to 109.)

The method of constructing the joint is to build up a wall of smoth-faced concrete blocks, carefully pointing the joints. This wall is kept one or two courses in advance of the cyclopean concrete of the section being built, *i.e.*, of the section of which the blocks are a part. The section may be carried up almost any desired distance in advance of the adjoining section. (See Plate XI, Figs. A and B.) There is no apparent reason why the section face could not be built against a form instead of being built of concrete blocks; the form would certainly be much cheaper. Say that the materials entering into the concrete of the blocks costs the same as a like quantity of the materials entering into the cyclopean concrete. Then against the cost of mixing, transporting and dumping of the mass concrete, we have the cost of making blocks (which, including curing, storing and handling is about $1.00 per cu. yd.), transporting them to the dam, setting them with derricks in the same manner as face work and finally pointing the joints. Certainly this means a cost for the blocks which is many times the cost of a form to accomplish the same result.

It may be mentioned here that the introduction of such joints operates to reduce the amount of stone which it is feasible to introduce per cubic yard of masonry. The reason is that such stone, in order not to interfere with the construction of the joint, must be kept away from the joint upon one side and from the blocks forming the first side of the joint upon the other. It is probable that the Olive Bridge and Kensico dams would otherwise have contained a percentage of stone nearly or quite equal to that in the Cross River, viz., 35 per cent instead of 25 per cent. In addition to taking care of any longitudinal movement due to changes of temperature, these joints obviously afford a perfect adjustment for any possible unequal settlement of different sections of the dam, so that no cracks can be produced from that cause. A dam with such joints therefore may be regarded as a series of dams placed end to end with the leakage between them insignificant in amount and intercepted.

Expansion joints are in reality one of the most significant features of the entire advance in practice of dam construction, not solely nor even principally owing to their function in caring for settlement and temperature movements, but on account of the purely incidental function of making possible and even facilitating certain changes in construction methods. Briefly the change consists in a departure from the method of construction in horizontal sections to or toward a method of construction in vertical sections. This may become a most radical departure although so far it has not been made use of to the extent of modifying previous construction plant or methods.

For the purpose of handling water during construction, dams 40 ft. or 50 ft. high have been built in alternate sections with (in effect) similar joints between sections. If for any reason whatever any marked advantage will result, there seems to be no reason why the method should not be extended to its logical conclusion and be applied to the construction of consecutive if not alternate sections of a high dam. A writer has recently questioned the advisability of such procedure on account of the stresses which would follow possible extreme differences in temperature between the face of a completed section and the fresh masonry being built in the adjoining section. It can be shown, however, that such an objection is based upon conditions which may be entirely eliminated or the importance of which has been exaggerated.

Thus, assume that a vertical face is a certain number of degrees colder than the new concrete that is being deposited against it; still the temperature of each will be affected by the other during the interval between the deposition and the hardening of the fresh concrete. The old concrete will be warmed for some distance on its side of the joint and the new concrete cooled on the other side, so that in effect the transition will not be sudden but gradual. Even if some subsequent adjusting motion along the joint might be necessary, the joint will accommodate transverse motion even better than longitudinal. The purpose of the joint is to allow the masonry to move without rupturing itself in the endeavor. At the worst the conditions would be better than under the usual procedure in building dams. As pointed out in the paragraph on Initial Stresses the same conditions have always been encountered and they should be less inimical if confined to a vertical transverse plane than if occurring in a horizontal one where no joint is provided. If any motion is possible, or is anticipated, that would render two vertical bonds or recesses in the joint inadvisable, it should be remembered that one is all that is necessary and that one is better than two.

If expansion joints at right angles to the axis of a straight dam are feasible, so also would be radial joints in a curved dam. Not that the radial joints would be necessary as expansion joints but they may be advisable because they would adjust settlement and permit the above suggested departure in construction methods. The extent of the revolution in construction methods to which such joints may give rise will be discussed in the chapter on Probable Future Methods.

At the Farnham dam (see page 243) the transverse expansion joints are 80 ft. apart, with 6 in. recesses 8 ft. apart along the joint. Each joint is intercepted by a drainage well on the upstream side of which a copper strip crosses the joint. The first side of the expansion joint was formed with concrete blocks, and the face was covered with paper between two coats of pitch before the cyclopean masonry of the other side was built against it. During the last of February, 1913, with reservoir full, the drainage wells intercepted and carried away leakage amounting to 18,000 gal. per day. During the following summer, with reservoir drawn down 10 ft. or 12 ft., this leakage had decreased to 200 gal. per day.

Ice Thrust.—The provision in the design of dams for a possible thrust from a field of ice is a question which has for some years been recognized, and regarding which considerable controversy has been waged. The board of experts who advised in the design of the Quaker Bridge dam recommended that provision be made for a thrust from the ice, at full reservoir level, of 43,000 lb. per lin. ft. of dam. In the dam as later built some distance upstream at a new location, and known as the New Croton dam, no provision for ice thrust was included. Several dams constructed since then however have included the provision. The dams, and the amount of the allowance in pounds per lin. ft. are as follows: Wachusett 47,000; Olive Bridge 47,000; Kensico 47,000; Croton Falls 30,000 and Cross River 24,000.

These allowances must have been based upon two assumptions, namely, the amount of the crushing strength of ice and the maximum thickness of ice expected in the particular locality. Although the figures for these assumptions have not been stated, it may be interesting to observe that 47,000 lb. per lin. ft. of dam corresponds to assumptions of strength of ice 200 lb. per sq. in., and a thickness of $19\frac{5}{8}$ in. The *American Civil Engineers Pocket Book* quotes U. S. Engineer Corps experiments showing a crushing strength of 100 lb.

to 1000 lb. per sq. in. Doubtless, such wide variation depends entirely, upon the conditions under which the ice was formed.

If it is proper under any given circumstances to include in the design some allowance for the thrust of ice, 200 lb. per sq. in. may be a close enough approximation to crushing strength of ice formed under actual conditions—but against what is the reaction applied? A certain gentleman once called attention to the fact that a fulcrum was as necessary as the lever in order that he might move the world; a perfectly sound principle whatever the weights involved.

In current discussion regarding the matter of ice thrust, there is much that is superficial and ill considered. Thus the question of what is to hold the ice while it is exerting the thrust seems to have been either misapprehended or ignored entirely. Apparently the question of thrust has been confused in some minds with the idea of motion of the sheet as a whole. For example, in the *Transactions* of the American Society of Civil Engineers, an engineer advances the following as one of the reasons why no provision was made for ice thrust in a recent important dam: "Because the point of rock projecting into the reservoir in front of the dam will protect it in such a way as to make this thrust small." This line of reasoning would seem to lead to the conjecture that a rocky point opposite the entire length of the dam, and but a short distance from it, would afford complete protection. As a matter of fact, it should be obvious that the shorter the ice column and the more unyielding the abutments, the more likely is the ice to exert an effective thrust.

Only one case, that of a low overflow dam, is on record where the failure of a dam was unmistakably attributable to ice thrust. Opposed to the upstream side, and at a short distance, was a masonry foundation wall. A considerable thickness of ice was formed, and the fluctuations of water level were such that the ice acted with a toggle-joint effect between the dam and the foundation wall.

On a pond the ice will expand and crowd into or slide up onto the shores. The movement may be considerable, and the causes and process may be analyzed as follows: As the underside of the sheet of ice is in contact with the water, it takes its temperature from the water and is constant at 32 Fahr. While the ice is thin the temperature through its entire thickness may be approximately the same, but as the ice grows thicker the top surface of the sheet gets more and more away from the influence of the water and is subjected to that of the air. As the air gets colder the upper surface contracts and cracks. Water enters the cracks and is immediately frozen.

Then when, as during a sunny day, the ice expands from the rise in temperature, it undoubtedly opens the same or additional cracks on the underside of the sheet which are in turn filled with water to be subsequently frozen. Thus the process may repeat itself with each marked fluctuation of temperature. The movement of the ice and the unequal resistance offered by different portions of the shore doubtless acts to increase the width of the cracks, or a portion of them, and this effect is in turn the cause of additional filling, freezing and movement.

The Boston Metropolitan Water Board made observations for some two months one winter on the motion of the ice around the shore of a pond. The pond was about ½ mile in diameter, the ice 12 in. to 15 in. thickness, the water level practically constant and the surface generally was free from snow. At seven or eight places around the pond distances were carefully measured between points on the shore and points opposite on the ice. During the period of about two months the points on the ice all moved toward the shore by amounts varying from 4 in. to 7 in., and apparently depending principally on the steepness of the bank and the amount of resistance that it offered. No attempt was made to measure the force exerted.

Disregarding ice which is in motion as a sheet, which is subject to wave action or violent fluctuations of elevation, and considering only ice under such conditions that it may produce thrust as here discussed, it would be a fair statement of the case to say that if ice can ever exert a crushing thrust against a dam it could crush against itself. Yet this phenomenon was never observed and never will be. Ice moves into or slides up onto a shore simply because the shore offers less resistance to the ice than the ice itself offers to compression.

The experiments, as to crushing strength, noted above, also yielded figures varying from 6 per cent. to 30 per cent. as to amount of compression before crushing. Assuming any intermediate value we like as being applicable to an actual case, and assuming the worst possible location for the dam, it is almost impossible to conceive that the ice can exert more than a small fraction of its crushing strength as thrust against the dam. It should not be difficult to devise nor expensive to conduct under actual working conditions a series of experiments to ascertain: first, what the expansion of a field of ice would be; second, the amount of thrust that the ice could exert when confined between immovable opposing shores. In all probability the results of such experiments, though doubtless interesting, would show that an allowance for ice thrust is very seldom, if ever, necessary in the design of a masonry dam.

Initial Stress.—There is one condition which necessarily accompanies the construction of every large dam which is never taken into account in any discussion of the principles of design. It may be that this neglect is because of the difficulty, if not impossibility of assigning a correct value to it, or of suggesting a remedy. Nevertheless the condition may be recognized. The condition is that the construction of a large dam extends over several seasons. Now

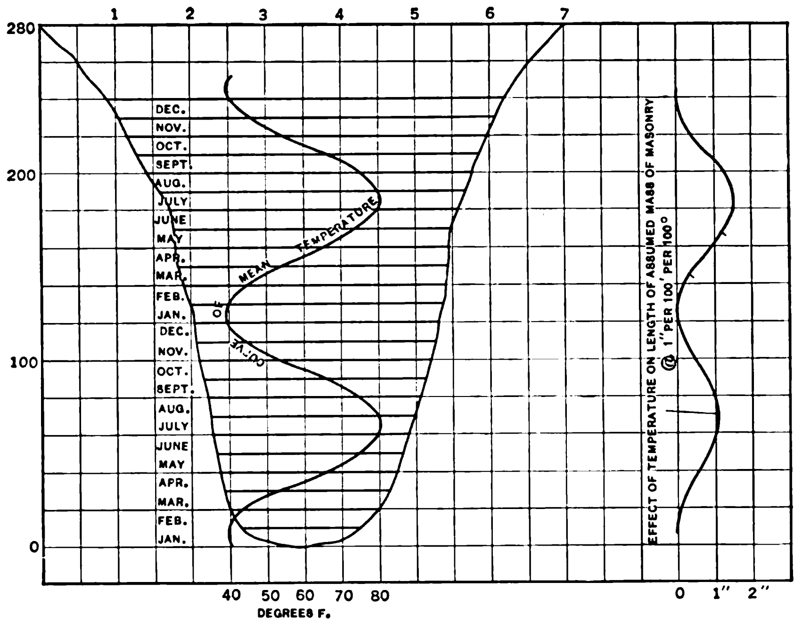

FIG. 24.—Showing possible range of temperature during construction, and its effect.

while in localities of severe winter climate construction is suspended during cold weather, it is easily possible that portions of the dam are built during mean temperatures 40 deg. Fahr. or more different from that prevailing during the building of other portions. It follows then that certain layers of masonry built in cold weather actually contain more masonry per unit of length than others built during warm weather. The measure of the amount is $L \times$ coefficient of expansion; and whatever the subsequent mean temperature of the mass, stress will exist between those layers. In Fig. 24, though a dam is not built in such regular layers, the principle is correctly illustrated.

Suppose the assumed dam is built under the temperature conditions shown. Then the curve of effect shows the differences in length at the various elevations, whatever may be the subsequent temperature of the masonry, and stresses sufficient to produce those differences should exist. It may be argued that the maximum stress acts between two levels 60 ft. apart, and that the comparatively fresh masonry will tend to accommodate itself to some mean temperature, being assisted thereto by the force of gravity and resistance of side walls. This is undoubtedly true to a certain extent. However, the conditions assumed in this case are far from being the most severe that it is possible to encounter. The assumed range of temperature is moderate. Masonry is often built at 10 deg. or 15 deg. lower temperature. Further, the temperature for periods, even if short, might be over 80 deg. with a corresponding change in the curve of effect. Again some interruption to the work might sharply accent the curve. Thus a suspension of work for two months, say April and May or September and October, would bring three-fifths of the maximum effect to act between planes but a few feet apart. As an extreme case conceive of a 6-in. granolithic surfacing being built in January on top of masonry which had been built in July. Then subsequently the surfacing will be a detached mass, simply laying on the masonry, unless the temperature cracks in the masonry are carefully duplicated by expansion joints in the surfacing. Carefully devised expansion joints might relieve much of the stress, but only in the direction of the length of the dam; it is a remedy that could not be applied of course, to the stress across the dam. It is only intended here to call attention to conditions which seemingly must exist in some degree. Any intelligent measure of them must be prefaced by more accurate knowledge of the temperature changes in a mass of masonry.

Instructions to Inspectors.—Some of the following instructions are applicable only to dams built under specifications similar to those of the Wachusett dam, *i.e.*, vertical joints filled with hand-laid mortar and spalls. It is probable also that not all of these instructions are applicable to any one dam. Where necessary the reason for the instruction is given. Under the old method of masonry construction, an inspector should be stationed at each derrick. Where concrete is used for bedding the large stone and for filling vertical joints, an inspector can properly look after more than one derrick.

First.—Be on the work in the morning before the workmen. This is especially necessary in case there have been any operations during the night such as bringing material (stone or spalls) onto the dam. A stone might be hit and its bed broken. Rather than take it up a workman will in thirty seconds with a trowel obliterate all evidence.

Second.—See that no foundation is built upon unless it has been accepted by the engineer, and cross-sections have been taken.

Third.—Even after acceptance of the foundation there are still details that the inspector must be responsible for, *i.e.*, to see that all loose or unsound fragments of rock are removed, also that it is clean.

Fourth.—If there is water in the foundation see that it is diverted, and that any necessary pipes are inserted either for handling the water or for future grouting operations. This is a most important feature of the work, and few inspectors are entirely capable of handling it. The chief inspector or the engineer in charge should be present and be responsible for it.

Fifth.—See that the stones are sound and free from open seams or dirt. Have any thin or feather edges knocked off. See that it is to set on its proper bed and is not too tall for its bed; and that it is wet.

Sixth.—See that the mortar is well tempered, and of proper consistency for the particular stone; that the bed is properly prepared; that the stone is lowered onto its bed in such a way as to expel all air; that after being lowered it is floated in position without developing any hard spots in its bed; that superfluous mortar is shoveled up and saved for use elsewhere, being careful that the remaining mortar does not fall away from the bed of the stone at any point.

Seventh.—See that the filling of the vertical joints is such that there is no unoccupied space. If of mortar and spalls the mortar should be thinner than for derrick stone; if of concrete and spalls the concrete should be fluid enough to flow into all parts of the joint. See that this operation is assisted by shovels or bars so that no nesting of the aggregate occurs. The spalls should subsequently be thrown in and pressed down into the concrete. The spalls must be sound, clean, and wet. See that the largest practicable percentage of spalls is introduced.

Eighth.—See that exposed faces are smoothed up and left with a neat workmanlike appearance.

Ninth.—When building on previously laid masonry see that there are no loose stones in the bed. Carefully and thoroughly remove all dirt or inert cement.

Tenth.—Use plenty of water to keep the work in a moist condition and prevent any drying out of mortar or concrete. This is to be watched most particularly in hot or dry weather.

Eleventh.—See that the bond is such that there are no continuous joints in any plane or direction.

Twelfth.—See that face stone meet the same requirements for soundness and cleanliness; also that they conform to the specifications as to dimensions, number of headers, bond, etc. See that they are properly set to correct line and grade, preserving proper joint thickness. See that the joints are properly raked out for subsequent pointing.

Thirteenth.—Carefully preserve any lines or grades set by the engineer until they have served their purpose. If any doubt exists as to whether a line or grade is correct or may have been knocked out, have it checked before using it. In general see that any point or line established for guidance is consistent with adjacent points and with adjacent completed work.

Fourteenth.—If any grout pipes are being brought up through the masonry keep a note as to their number and location. See that none are overlooked and covered up.

Fifteenth.—Keep a small diary in which record each day the following:

Location of derrick.

Name of foreman.

Force employed under it.

Any unusual incident or accident.

Any lost time and the cause for it.

Carry a 2-ft. rule and record the dimension of each large stone delivered to your derrick for its use.

Record the number of boxes of spalls received, and number of batches of mortar and concrete.

Record any important order received from engineer or chief inspector or given to the foreman.

Record full particulars of any work done under such circumstances that it may be claimed as "extra" work.

Sixteenth.—While it is the duty of the foreman or derrick engineer to see that the equipment is in safe working order and that no condition involving danger to any of the force is allowed to exist, see to it yourself, for your own sake if for no one else's.

An accident which might have been prevented reflects on the inspector as much as on the foreman.

Seventeenth.—Remember that you are not concerned with the quantity of work performed but with the quality. Hence do not give orders that encroach on the province of the foreman, as you may furnish a basis for a claim on the part of the contractor.

Eighteenth.—Be tactful. Do your duty firmly and justly but as far as possible in a manner that will further the foreman's efforts. You can accomplish more by working with him than against him. If you show the proper spirit almost any foreman is open to suggestions in furtherance of the interests of the work where he would not tolerate orders. In any case do not have a fight. If the foreman must be disciplined let the chief inspector or engineer do it.

Nineteenth.—Make use of all reasonable and proper assistance in forming an accurate estimate of your own knowledge and importance. Do not be above learning anything from anyone.

TO THE CHIEF INSPECTOR

Observe all of the foregoing instructions to inspectors and

First.—See that the inspectors do likewise.

Second.—Keep in close touch with every feature of all operations upon which the quality of the masonry depends.

Third.—At any stage of the work there will be features which will require your particular attention and a larger share of your time. This will no doubt always apply also to certain foremen or certain inspectors. Some will require more watching than others.

Fourth.—Study the characteristics of the foremen and inspectors in order to assign the inspectors to the best advantage. Some men who are mutually antagonistic would get on very well if paired off with others.

Fifth.—Be present at any operation requiring special knowledge or skill, and direct it as far as possible through the inspector assigned to it. If it is necessary that you take charge of the operation or actually perform some important detail, do so, remembering that practically all workmen of whatever grade are conscientious and will receive in a proper spirit any instruction tendered in the same spirit.

Sixth.—Preserve your mental balance. Don't be stampeded over anything. Consult with the engineer over any ambiguous clauses in the specifications.

Seventh.—Keep a diary the same as mentioned in Rule 15 to inspectors. Each night assemble in your diary all the numbers and figures kept by the various inspectors. See that they are reasonable and consistent. Compare the figures as to mortar and concrete turned in by the masonry inspectors with those kept at the mixer.

Regarding the figures as to stone, each inspector can readily keep the number of boxes of spalls delivered to and used by him. In case a night force is employed to deliver large stone upon the dam it may be necessary to have someone present to measure it. Stone coming on through the day can be measured by the inspector to whom delivered. Each stone should be measured and recorded thus $2\frac{1}{2} \times 3 \times 4\frac{1}{2}$, $2 \times 3\frac{1}{2} \times 5$, etc. A stone can be measured and recorded in thirty seconds. Recording say twenty to forty stones will not interfere with the inspector's other duties. The desirability of keeping these notes requires some explanation.

For batches of mortar and concrete it is well to have a check on the account kept by the inspector at the mixer.

The relative number of yards of concrete and spalls is a direct measure of the efficiency of the force under a particular derrick. Up to a certain limit it is economical to put spalls into the concrete, although to be sure the question of whether it is to the interest of the contractor or your employer depends upon who is paying for the cement. For instance, 40 cu. yd. of concrete to 9 cu. yd. of spalls shows one thing and 38 cu. yd. of concrete to 11 cu. yd. of spalls shows quite a different thing. The latter derrick has substituted 2 cu. yd. of spalls for 2 cu. yd. of concrete at a saving of say $5.50.

Regarding the large stone: Though it would be well to keep separately the quantities going to each derrick if it can be done readily, still the purposes for which this information may be desired are such as to render this segregation of less importance.

A.—As the mortar, concrete and spalls are accounted for it would be well to keep also the remaining element (stone) in order that your total may be complete in itself and furnish a rough check on the engineer's cross-sections, not only from time to time as for monthly estimates, but for the final estimate.

B.—If your totals are carefully kept and show a proper correspondence with monthly estimates, they might in case of necessity be used as a basis for a monthly estimate.

C.—Accidents are always possible, other notes might be lost or destroyed creating a situation such that the inspector's account might be very valuable if not invaluable. Unforeseen questions or contingencies might arise which could only be settled by such accounts.

D.—Some clause in the specifications might easily render such accounts absolutely necessary. For instance, that material excavated from the quarry, or certain quarries, should be paid for at an excavation price if it was not used in the masonry. Again, quarry conditions might be such as to render impracticable any measurement of stone and waste by taking cross-sections. Under such circumstances it might be necessary to keep an inspector in the quarry to account in a similar manner for everything that went out of the quarry.

Eighth.—As an inspector should cooperate with the foreman, so should the chief inspector cooperate with the superintendent. Study the larger conditions affecting the progress as well as quality of work, such as amount, condition, and capacity of plant, arrangement and proposed future arrangement of derricks, quality and quantity of quarry output, etc., not only to plan your own work intelligently, but to advise with the superintendent if occasion arises. You may have more time than the superintendent in which to study certain features, although the superintendent's province is quantity and yours quality, it will be to the advantage of the work if your relations are such that you can freely discuss with him all sides of any question.

CHAPTER IX

PLANT AND POWER

Details as to location, layout or arrangement of plant are so involved with the topographical and other conditions peculiar to any particular dam that it is almost useless to lay down any but a few of the most general rules. However, kind and amount of plant are susceptible of more definite treatment. The following discussion is intended to be of assistance to constructors who are engaged in working out actual problems as well as to furnish an intelligent basis for rapid preliminary estimates. Roughly speaking the necessary and advisable investment in plant for the construction of a masonry dam of some magnitude may be assumed as from 75 cents to $1 per cu. yd. of masonry. The salvage on plant at the end of a job, while a more variable quantity, may be assumed as from 20 per cent. to 50 per cent. of the original cost.

Plant as discussed here is held to include in addition to actual cost of machinery, the freight, handling, wagon haul and erection; also the materials (as lumber and cement) entering into machinery foundations and housing; also any necessary roads or railroads built to reduce cost of haul or to provide access to or communication between different parts of the work.

The first question is the distance from the work to the nearest point on the railroad, and the advisable amount to spend in reducing cost of haul. If a wagon road exists it may be desirable to spend some money surfacing it or to reduce grades. If it is necessary to build the road it may cost anywhere from $1000 per mile in easy country to $25,000 per mile for heavy rock work in mountainous, inaccessible country. With a good road, with grades generally not over 6 per cent., with a length of haul such that loading and unloading take but a small percentage of the time of the teams engaged, with freight coming along with good regularity so that a regular force of teams can be organized and employed without much lost time, a widely applicable and fair rule for cost of hauling ordinary freight like cement, fuel, hardware, supplies, etc., is 25 cents per ton mile. Special or heavy pieces of machinery which require special trucks or which involve much labor loading and unloading may cost twice that.

However, such pieces form but a small part of the total tonnage. The above price might be reduced somewhat by using motor trucks.

In considering how much saving in cost of haul might result from building a railroad, the following actual experience is pertinent. The *Reclamation Record* for February, 1914, gives the construction and operating costs for such an industrial road at the Arrowrock dam. The distance from Barberton (the nearest railroad station) to the dam was 17 miles. The cost of hauling freight over the rough wagon road was $8 per ton, or about 47 cents per ton mile. Seventeen miles of main line and 2 miles of spur track required the excavation of nearly 250,000 cu. yd. of material, which was done by contract. Culvert and bridge building, track laying and ballasting was done by Government forces working an eight-hour day. The maximum grades on the main line are 1½ per cent. and on the spur tracks 3 per cent. There are three wooden truss bridges on the main line, one 442 ft. long, one 166 ft. and one 366 ft. The total cost of construction was about $20,000 per mile. The rolling stock consists of two locomotives, two passenger cars, four flat cars, three box cars, miscellaneous hand cars, speeders, etc., and cost about $35,000. One train is run each way each week day, with double trips Saturday.

During two years operation by the Government January 1, 1912, to December 31, 1913, 30,337 train miles have averaged a cost of $2.34 per train mile, the minimum for one month being $1.53 per train mile. The total ton miles of freight amounted to more than three-quarters of a million, and 45,000 passengers were carried. Thus in this case the cost of haul was reduced from 47 cents per ton mile to about 9 cents, a saving of $6.46 per ton over the 17 miles. Assume charged against this saving the entire cost of 19 miles at $20,000 or $380,000, plus say $20,000 depreciation on rolling stock, making a total of $400,000. Then for a total volume of freight of about 61,900 tons the investment in the railroad is justified to say nothing of the values attaching to prompt delivery of freight and to good passenger service. Only surveys and a careful examination could determine the feasibility and cost of such a railroad. Allowable grades might be obtainable only at prohibitive expense and the cost of grading and bridging might vary widely from the above example. In addition to the grading and bridging the cost for ties, rails and laying would be $8,000 to $10,000 per mile.

In connection with a possible railroad consider the question of quarry location, and whether, if the quarry be at some distance from the dam, one railroad will not serve to transport stone as well as other freight and thereby furnish a further economic justification for the railroad. It might be that the choice between two or more possible quarry sites might be influenced by the possibility of combining the two reasons for building a railroad. As the total tonnage bears on the question of road or railroad, so does the yardage to be handled influence the amount and kind of plant at the site of the work. Other considerations are the manner of doing the work, which in turn depends upon the general design, the amount of water to be handled during construction, the length of the working season between interrupting floods or winter weather, the total time allowable for the work, etc. Generally speaking it would be silly to put as much plant upon a 50,000-cu. yd. dam as would be advisable upon a 300,000-cu. yd. dam. On the other hand if there were but two months in spring and two months in fall between severe winter weather and a summer season of floods, it might well be necessary to put on a plant of many times the capacity that would be advisable if there were no interruptions from winter or from floods.

Another question which might affect the amount of advisable expenditure for plant is the amount of salvage to be reasonably expected in the particular case, considered in connection with the reduced operating cost which might follow from certain proposed additional equipment. Before discussing this question, however, let us briefly examine the principles governing depreciation and net plant account chargeable to similar jobs under different conditions of location and accessibility.

Depreciation of Plant and Other Factors.—Plant charge must include not only the depreciation on equipment, but freight and handling to the job and back to where the equipment can be sold or used on another job. It is often the case that return freight and handling may sometimes amount to more than the depreciated value of certain portions of the equipment so that it does not pay to remove it. Let us see how the matter may work out actually. We will assume two jobs precisely similar as to apparent plant requirements. One of these, however, is located near a center of supply and demand, and is easy of access; the other in a remote location and more difficult of access. The machinery and equipment has been kept in ordinary working condition, has been used five years and has been rated as worth 50 per cent. of first cost. As some portions will be scrapped, assume that four-fifths of the original weight is removed from the work. Naturally the materials which enter into the erection will cost more in one case than in the other.

	A In remote location rather difficult of access	B Located near supply center, easy of access
First cost of machinery and equipment at center of supply	$150,000	$150,000
Freight to railroad station nearest the work	25,000	10,000
Wagon haul and handling	10,000	2,000
Materials and labor of erection	45,000	38,000
Interest five years at 6 per cent	69,000	60,000
Total plant cost	$299,000	$260,000
Salvage value 50 per cent	$75,000	$75,000
From which deduct freight and wagon haul on four-	20,000	8,000
fifths original weight, back to point of disposal	8,000	1,600
Actual salvage to credit of job	$47,000	$65,400
Plant cost	$299,000	$260,000
Minus actual salvage	47,000	65,400
Plant account chargeable to job	$252,000	$194,600

The difference in plant charge, depending on location is $252,000 minus $194,600 or $57,400.

Although additions to plant investment may seldom be so definitely convertible into terms of reduced operating expense, let us assume that a purchase of 25 per cent. additional equipment will reduce the cost say 15 cents per cu. yd. on 300,000 cu. yd. of masonry. Now let us see how the plant accounts work out in the two locations. Freight, handling and erection are assumed to be increased in proportion.

	A plus 25%	B plus 25%
First cost of machinery and equipment at center of supply	$187,500	$187,500
Freight to railroad station nearest the work	31,250	12,500
Wagon haul and handling	12,500	2,500
Materials and labor of erection	56,250	47,500
Interest five years at 6 per cent	86,250	75,000
Total plant cost	$373,750	$325,000
Salvage value 50 per cent	$93,750	$93,750
From which deduct freight and wagon haul on four-	25,000	10,000
fifths original weight, back to point of disposal	10,000	2,000
Actual salvage to credit of job	$58,750	$81,750

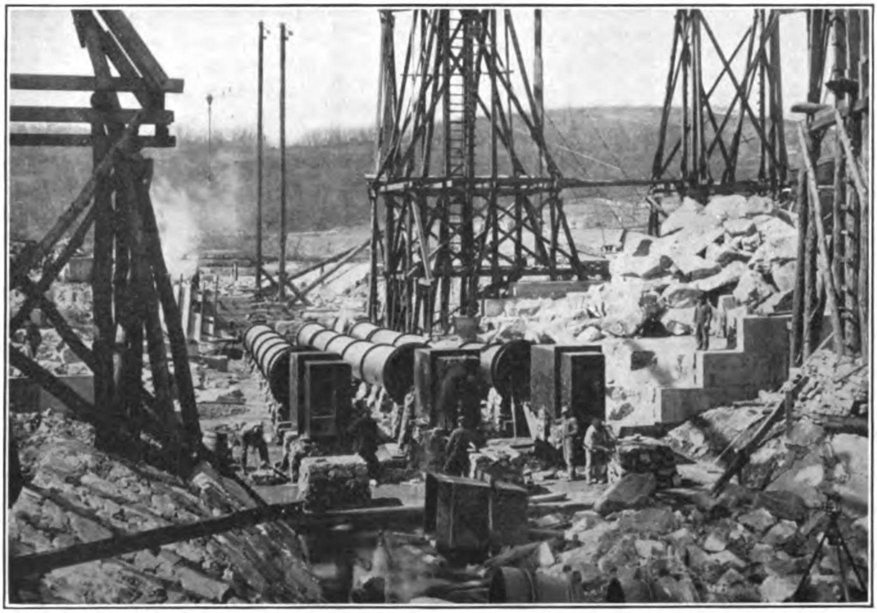

FIG. A.—Croton Falls dam. Setting the outlet pipes.

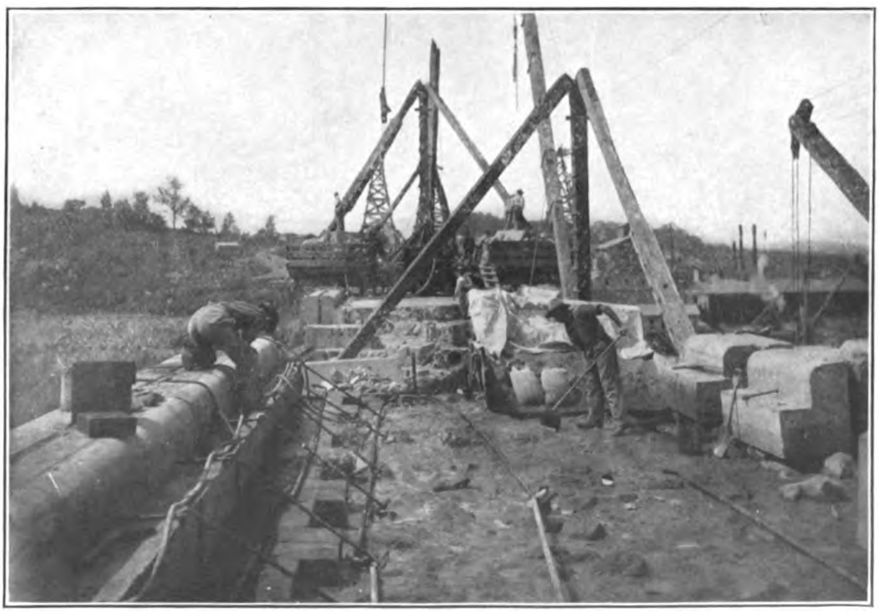

FIG. B.—Cross River dam, showing reinforcing rods and the anchoring of the coping course.

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PLATE IX

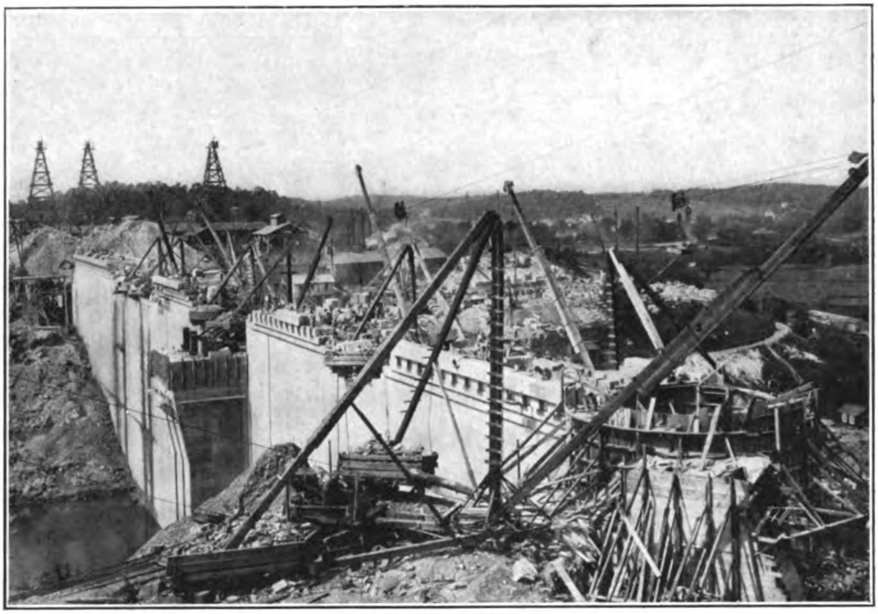


FIG. C.—Cross River dam nearing completion. Mixing plant and block-yard in background.

FIG. D.—Concrete block-yard at Cross River dam.

Plant cost	$373,750	$325,000
Minus actual salvage	58,750	81,750
Plant account chargeable to job	$315,000	$243,250

The difference in plant charge, depending on location, is $315,000 minus $243,250 or $71,750.

This shows that the contemplated purchase of 25 per cent. more plant in order to effect a saving of $45,000 in building cost would be inadvisable in the case of A and about a standoff in the case of B. The resulting additions to plant account chargeable to job are $63,000 for A and $48,650 for B. Of course, if the increased plant resulted in greater daily output, thus reducing the time required for the work, it might be that the expenditure would be justifiable in either case. Assume, for instance, that interest and other overhead charges might amount to $75 per day. Then a reduction in time of 240 days in the case of A and forty-nine days in the case of B would balance the apparent deficit above.

Source of Power.—Both the source, and the manner of distribution of the power are matters of increasing importance with increase in magnitude of the work. In addition to mere magnitude there may be many individual factors which should be considered in arriving at the best selection. The various conditions are here discussed and supplemented by some information gathered from certain examples of large dam construction. The questions of source and distribution are so inseparable that they will be discussed together.

For a very small dam, the mere yardage involved may indicate that it should be handled by one or two hand derricks or by derricks operated by a team of horses. If the size of the work indicated this method it would also mean that the dam would be a simple structure of rubble masonry comprised largely of one-man stone, spalls and mortar hand-laid; or if some natural deposit of concrete aggregate is available, a structure of hand-mixed concrete into which might be put any suitable one-man stone that could be readily gathered and brought to the job. The natural power for a dam considerably larger, but still in the class of small dams, would naturally be steam even at rather a high cost of fuel. This would require separate units of boiler and engine for each derrick, crusher and mixer, boiler to run steam drills in the quarry, etc. Such dams as these are so small that the various operations are not and need not be sharply differentiated. The work consists in bringing the materials to the site by teams in the simplest way possible and of putting them into a dam by the use of the most inexpensive and most easily transported plant.

When we get above the limit in size for which such simple procedure is advisable we encounter more and more conditions which should be more and more closely scrutinized. Steam used in small units or distributed over portions of the work from one central boiler station soon becomes out of the question. Recourse must be had to a central power plant and distribution by compressed air or electricity. In some locations or under certain conditions it may be that hydro-electric power can be economically generated for construction purposes. A casual consideration of conditions may answer the question or at least indicate that a detailed study of the question should be made.

The conditions favoring hydro-electric generation are:

First.—High cost of fuel. This may be owing to scarcity in the immediate vicinity; distance from a fuel supply with consequent high freight charges, or distance from a railroad involving expensive wagon haul.

Second.—An available quantity of water in combination with a stream slope such that the necessary channel for conveying the water from the stream to the wheel would be within bounds as to expense.

Third.—The possibility of diverting the water from the stream with an inexpensive dam.

Wood at best is an expensive fuel. Even if nothing is paid for the wood it must be cut, gathered and brought to the boiler. The cutting alone would make it equivalent to coal at $3 per ton. The haul to the boiler must usually be largely over the poorest roads or no roads at all. If the job is of some length and magnitude and the wood thin and scattering, it must soon be brought considerable distances. Finally the wood is usually green and of low fuel value.

After obtaining prices of coal or fuel oil delivered at the nearest point on the railroad, figure on the cost of haul, if any, to the work. Use say the previously mentioned basis of 25 cents per ton mile. Remember that if fuel oil is used only about one-half the tonnage must be hauled; *i.e.*, six 42-gal. barrels of oil weigh about 1 ton and when used to generate steam have a fuel value equal to $1\frac{1}{2}$ to 2 tons of average coal. While the use of fuel oil involves an initial outlay for a storage tank, this tank need not be very costly. In operation the fuel oil can be handled and the boilers fired more economically than with coal because the use of coal involves expensive ash disposal, etc. Internal combustion engines and gas producer plants, however economical of fuel and eminently worthy of consideration in permanent plants, are so costly to install that they are usually outside the range of discussion for temporary plants for construction purposes.

If it is necessary to haul the fuel because the work is not accessible by rail, it may be cheaper to build the power station at the railroad and transmit electric power to the work. The determining factors are the cost of hauling the total amount of fuel required, and the cost of a transmission line including necessary transformers, etc. Should the completed project contemplate the generation of electric power and its transmission to a market located (as is almost necessarily the case) on a railroad, it may be advisable to anticipate somewhat the construction of the transmission line even at the expense of interest. In this event, power would be sent back from a steam plant at the railroad to the work during the construction period.

In figuring upon the relative merits of steam and hydro-electric generation, the determining consideration may very likely be the length of time that the plant will be in service. A hydro-electric installation might require a much larger initial outlay than an equal steam installation but its operation and maintenance would be much less; while the much lower first cost of the steam installation is offset by its greater labor and fuel costs. Thus a steam plant might be much cheaper for a two-year job and much more expensive for a four-year job. In estimating the plant charge for each method make proper allowances for interest and depreciation. These items will usually be heavily against the hydro-electric installation, as in addition to higher first cost and larger item of interest, the depreciation on items of diversion dam and channel for conveying the water must generally be taken as 100 per cent.

In connection with the depreciation of the hydro-electric plant, study should be made as to whether the plant may not have a value after the construction period as a permanent plant if continued in operation at the same place. It may indeed be planned as a desirable permanent feature of the project for which the dam is being constructed, in which case an agreement might be embodied in the construction contract. Or, as was the case in one prominent instance, the installation might be made by the owners of the project and power sold to the construction company at a stipulated price. A still further possibility is that power might be economically purchased from some adjacent company engaged in the commercial production and distribution of power. A large electric light plant, for instance, with an installation equal to several times its day load might welcome such an addition to its day output as this would involve practically no additional expense except for fuel. A company in such a position might make a price for power sufficiently attractive to warrant the construction of the transmission line.

Distribution of Power.—Comparing steam with compressed air it may be said, if steam is piped any distance from the boiler, the loss from condensation is great, further, the hot steam pipes are in the way and costly to maintain for machinery whose location must be often changed. If small detached units are used the consumption of fuel is under most wasteful conditions for it must constantly be burned at a rate which will meet the maximum power requirements instead of being adjustable for the frequent intervening low-load periods. It is expensive to deliver fuel and water and remove ashes from a number of detached boilers occupying all sorts of positions with relation to the work. For each such unit an engineer or fireman must be upon the work for an hour or so each morning to get up steam before the laborers start their shift. Lastly is the consideration, important in many localities, that licensed engineers must be employed for such use of steam, whereas such requirement is not made in case of using compressed air.

With a centrally-located compressed-air plant much more economical arrangements can be made for handling fuel and ashes. An appreciable saving of fuel results from using compound condensing engines. A further great saving follows from the fact that the power generated may not be over half the sum of the capacity of the engines supplied, for not all the engines are in use at the same time. This point will be elaborated later. The air may be easily piped for long distances and connections maintained or readily changed. Licensed engineers are not required and it is a comparatively simple matter to train plenty of men as engine runners.

Within the last few years electricity has been successfully applied to the rather severe requirements of a service, where the weight of load to be handled varies within such wide limits. Direct current was first used as it is readily controlled to afford a wide range of speed, *i.e.*, a slow speed for a heavy load or for certain particular operations, and a much higher speed when light load or other circumstances permit. Recently, however, at the Kensico dam alternating current induction motors have been applied to hoisting engines on this class of work. The engines are constructed to obtain two speeds by a shifting of gears as in an automobile. Thus the derrick engines (in this case 75 h.p.) operating with one motor speed can obtain a rope speed on the drum of 250 ft. or 500 ft. per minute. This type of engine is said to give entirely satisfactory service.

As in any such change the amount of investment in old plant delays the adoption of the new, so the amount of steam and air plant in the hands of contractors has delayed the adoption of electricity. One argument, that a considerable part of the power must be in the form of compressed air (for drilling purposes) and that the remainder might as well be, has lost most of its force with the advent of the Temple drill. To be sure, this tool is air-driven but with a small electrically-driven compressor for each drill. The outfit complete costs about $1200, as compared to $300 for an ordinary air drill. On the other hand, the power cost is about one-fifth as much as for the ordinary drill.[1]

Doubtless there will always be some situations favoring the use of steam or compressed air, but electricity will come to be much more generally used as it often possesses very pronounced advantages.

First.—The power may be transmitted for very much longer distances, thus vastly increasing the range of possible location of the central generating station with respect to the work. This wide range will often render available cheaper fuel, *i.e.*, the location of the cheapest fuel may determine the location of the central station. It may permit the development and use of a water power too remote to be considered otherwise or it may bring within the range of possibilities the purchase of power from some existing company thus saving the expense of installing a central generating station.

Second.—For any distance of transmission the line losses in electrical transmission will be much less than the leakage of compressed air in mains and the smaller connections.

Third.—In the case of compressed air the distribution mains are

[1] At the Kensico dam twenty-one of these drills are employed. They are the Temple-Ingersoll Type 5F, with cylinder 5⅝ × 8 in.; with pulsators driven by 5-h.p. 220-volt motors, giving at full speed 400 strokes per minute. The starting bits are 3½ in. to 4 in. in diameter. For the longest (28 ft. 6 in.) steel the diameter is 1¾ in. With drills actually cutting for 51.1 per cent. of the time (the remainder being taken up in shifting drill, changing bit, and bailing hole) the power consumption was found to be 30 kw-hours to 40 kw-hours per drill shift of eight hours. The *Engineering News* of March 5, 1914, gives the drill records in that particular rock.

much more costly to construct and maintain than the distribution wiring for electric power, especially where so many of the machines (the derricks) are moved so often.

Fourth.—Air must be cooled during the process of compression and when used during cold weather it must be reheated at the point of use, at the cost of some fuel and some bother even if the engine runner is the only attendant.

CHAPTER X

INSTALLATION REQUIRED AND POWER CONSUMPTION

The total number of machines required of each kind, such as derricks, cableways, pumps, crushers, mixers, etc., should be estimated as closely as possible. A further estimate should be made as to the maximum number which it will be necessary to have in operation at any one time in order to figure on the size of the central station outfit. Bear in mind the sequence of the various operations, namely how they may or may not overlap or be made to dovetail with other operations, the machines required for each in their various stages, etc. Thus stripping and opening of quarries and sand pits can be done at a time when only a small amount of power is required, as on river diversion works. The power required to handle excavation probably decreases as the depth of the pit increases (not because of increased depth but because of slower progress) and is partially offset by the greater amount of water to pump and the higher lift. On starting the masonry in the bottom of the pit, more and more room is available for, and increasing power is required for, the masonry as it comes up at the same time that the pumping is decreasing. It may not be possible to estimate closely the amount of power required for pumping. The number of derricks required in the quarry is also more difficult to predict than the number which can be employed on laying the masonry. The number of derricks required in the quarry depends largely on the character of the quarry and is ascertained only when the quarry is well opened up. The number of derricks also depends upon the masonry specifications as affecting the percentage of large stone it is expected to introduce into the masonry, and also whether or not the quarry is to be worked in such a way as to save considerable stone to be cut for the face work.

If a dam is to be constructed of cyclopean masonry containing 25 per cent. or 30 per cent. of large stone and with faces of concrete blocks, a much smaller number of quarry derricks will be required than in the case of a dam containing 50 per cent. or more of large stone and with cut stone faces. In general it may be said that the number of derricks required in the quarries will be between one and one and one-half times the number employed in laying masonry. Unless there are some special and remarkable conditions it will be safe to assume that a central power installation sufficient for the purpose of maximum rate of progress on masonry construction will be sufficient for any stage of the entire operation. Now the requirement for this stage can be estimated with sufficient accuracy.

If we assume as necessary for the construction of a certain dam eight derricks on masonry and ten in the quarry with engines of 30 h.p., two cableways at 75 h.p., crushing and mixing plant at 200 h.p., transportation of materials to dam site 100 h.p., for drills, shop purposes, and miscellaneous 100 h.p., it does not follow that the central power plant should be the sum of the above. Indeed it will be surprisingly less on the same principle that forty rock drills may be run on about twenty times the air necessary for one. For example, a derrick is actually engaged in lifting loads for but a very small fraction of the time. Say the derrick sets 15 cu. yd. of masonry (weighing 30 tons) per hour and that the work performed in handling materials and setting masonry is equivalent to lifting that weight three times to a height of 20 ft., plus an equivalent amount for weight of boom, swinging the loads passing empty skips, friction, etc. Then the derrick has done work equivalent to its nominal capacity (30 h.p.) for but seven and one-fourth minutes during the hour. Assume that one cableway rated at 100 h.p. serves four such derricks engaged on masonry, that the work performed is equivalent to raising 120 tons of materials plus 30 tons for skips to a height of 75 ft., that the conveying is equivalent to raising it 15 ft. more and that returning with empty skips is equivalent to 50 per cent. of the work of taking the loads out. Then the cableway has been employed equivalent to its rated capacity for but twelve and one-fourth minutes. Some portions of the plant, as the crushing and mixing plant, screens, etc., undoubtedly do work equivalent to their rated capacity for a much larger percentage of the time.

Comparison of Performances.—To show the relations between central station plant, h.p.-hours actually delivered, amount of machinery operated and work accomplished, some actual experiences are here given. The performance at the Roosevelt dam was quite accurately kept during a month when masonry construction was at a maximum. For the other examples cited the conditions are not as accurately known. Some of the data either were not kept or are not available, nevertheless they are believed to be fairly stated approximations that may be used as a basis for rough estimates. At the Roosevelt dam the conditions were as follows:

Electric power at 2300 volts alternating current was metered and delivered to the contractor, a motor-generator set transformed it to 500 volts direct current at which it was used. A small air compressor to supply the drills was belted from the motor-generator set. The quarries were immediately at each end of the dam and at a level higher than the masonry. Quarry derricks passed the stone to points under the cableways, also to the crusher. Two cableways transported all materials to the dam, and a third cableway transported quarry waste some 700 ft. to a dump. A tramway 1700 ft. long brought the cement from the mill to the mixer. At an intermediate station the sand was picked up and transported about 1100 ft. to the mixer. The tramway was about level except for the sag between towers. The tramway worked twenty-nine and one-eighth day shifts and twenty-eight night shifts, taking to the dam 13,538 buckets of sand at 8 cu. ft. and 9822 buckets of cement at 5.2 cu. ft. Two cableways in twenty-eight day shifts took from the mixer to dam 1056 skips of mortar of 2.2 cu. yd. each, 2571 skips of concrete at 2.62 cu. yd., each; they also took from quarries to dam 1876 skips of spalls. The skips were of iron 7 ft. × 7 ft. × 2 ft. and weighed about 2200 lb. Two cableways in twenty-nine night shifts took into the dam 7900 cu. yd. of stone. One cableway in two shifts per day disposed of 3150 cu. yd. of quarry waste. Nine derricks in quarries produced 7900 cu. yd. of stone for dam and 2200 cu. yd. of spalls and passed it to the cableways; passed to the crusher about 2570 cu. yd. of rock, and to No. 3 cableway 3150 cu. yd. of waste. Five derricks on the dam in twenty-eight and one-eighth day shifts laid 18,328 cu. yd. of masonry. One Smith mixer in twenty-eight and one-eighth shifts mixed 2112 batches of 1.1 cu. yd. of mortar and 5142 batches of concrete of 1.31 cu. yd. The rated capacity of each machine, hours actually employed, and h.p.-hours of 100 per cent. load factor assumed, is as follows on page 140.

The motors actually installed and used would total about 980 h.p. The consumption rarely if ever went over 400 h.p. even on momentary peaks. The amount of power actually delivered to the contractor during the month as metered at 2300 a.c. before transforming, was 165,013 h.p.-hr. Assuming an efficiency of 90 per cent. for the motor-generator set, then 165,013 × 0.90 = 148,512 h.p.-hr., were distributed to the various machines, indicating a load factor for the plant as a whole of about 34 per cent.

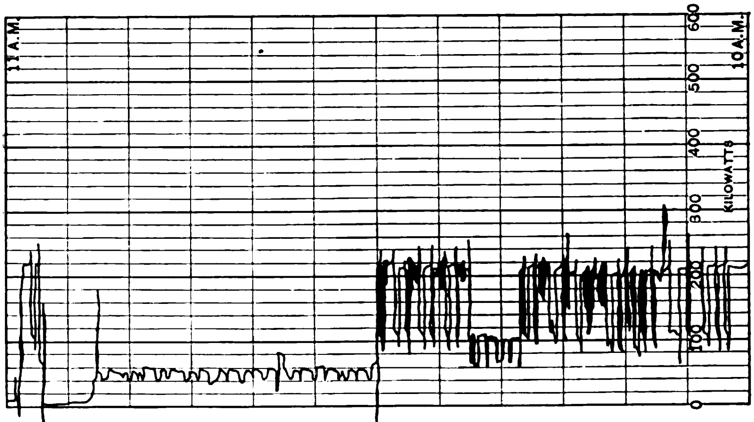
	H.p.	Hours employed	H.p.-hours at 100 per cent. load factor
Tramway	25	457	11,125
Crusher	55	209	11,495
Mixer	20	205	4,100
Two main cableways	100 each	456 each	91,200
No. 3 cableway	50	456	22,800
Nine derricks in quarry, day	8 at 40 each	2,081 total day	160,500
Nine derricks in quarry, night	1 at 35 each	1,988 total night	
Five derricks on masonry	35 each	1,073 total	37,555
Compressor for drills	100	Equivalent to 600	60,000
Lights at dam and camp			
Shop and camp purposes		Equivalent to	40,000
Water supply, ice machine			
Total			438,775

The principle is beautifully illustrated at the Arrowrock dam The following information and load curves on that work are taken from an article in *Engineering Record* for Aug. 24, 1912. At the generating station, 17 miles from the dam, are three hydraulic turbine-actuated 625-kva. generators which deliver three-phase 60 cycle alternating current at 2300 volts. This potential is stepped up to 22,000 volts for transmission. At the dam the current is stepped down to 2300 volts for distribution to the various motors. The feeder panels are equipped with curve tracing wattmeters from which the very interesting curves shown in Fig. 25 are taken.

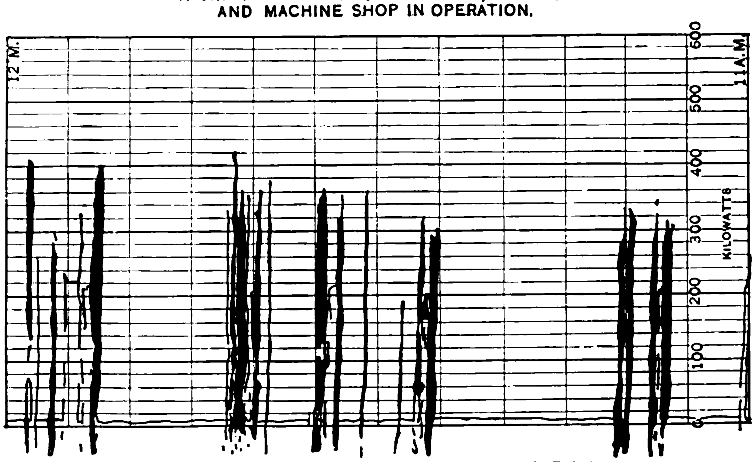
The apparatus contemplated for use at the dam comprises an installation of 3000 h.p. in rated capacity of induction motors, as follows: Two 300-h.p. cableways, 500 h.p. in various motors in the sand cement plant, 125-h.p. air compressor, 1180 h.p. in units of 50 h.p. to 125 h.p. for pumping, four 75-h.p. derrick motors and a large number of smaller motors for crushers, mixers, shops, etc.

At the Wachusett dam central power station were two 500-h.p. Rand-Corliss compressors. At their normal speed of 75 r.p.m. each compressor delivered 3310 cu. ft. of free air per minute raised to 90 lb. pressure. A 6-in. main ½ mile long led to the quarry and an 8-in. main 1¼ miles long to the dam. Air was supplied to the following: thirty-one 16-h.p. hoisting engines, two 50-h.p. cableways, fifteen to forty No. 3 Rand drills, three stone dressing machines, three 10-h.p. engines at mixer, screen, etc., three 3-in. to 4-in pumps, ten forges, one riveter in shop and one trip hammer.

With the above plant in operation the compressors were worked between 75 per cent. and about 90 per cent. of their full capacity depending on the number of No. 3 drills in operation. At one time the leakage in the air mains was such that with every machine shut off 11 r.p.m. of one compressor was required to maintain the pressure



A-CIRCUIT NO. 1 - AIR COMPRESSOR, DERRICK AND MACHINE SHOP IN OPERATION.



B-CIRCUIT NO. 2 - ONE CABLEWAY IN OPERATION.

FIG. 25.—Load curves on dam construction.

in the mains; corresponding roughly to 7 per cent. or 8 per cent. of the station output. Undoubtedly there was some additional leakage between the various valves where the machines were shut off and the machines themselves, so that when all were in operation the leakage must have been in excess of that observed when they were shut off. In other words some of the leakage must have been shut off when the machines were shut off.

The hauling of the stone and sand to the dam is not included in the above service. The stone was hauled by locomotive and cars on 2½ miles of standard gage track. The sand was hauled ½ mile by teams from the pit to bins near the mixer. At this time ten to eleven derricks on the masonry were setting 500 cu. yd. to 600 cu. yd. of rubble masonry per ten-hour day. At the quarry probably as much stone was wasted as was used.

The New Croton dam is one of the largest that has been completed to date. The quantities involved are 1,821,400 cu. yd. earth excavation, 400,250 cu. yd. rock excavation and 855,000 cu. yd. of masonry. The foundations were at a depth of 75 ft. to 100 ft. below the river and the pit was open so that pumping was necessary for about five years. The amount of water pumped when the pit was deepest varied from 5,000,000 to 8,000,000 gal. per twenty-four hours, depending on the weather and the stage of the river. (For a description of the river diversion works see page 34.) More than $100,000 was spent in opening quarries which for various reasons had to be abandoned. The quarry which supplied most of the stone was 6 miles from the dam to which the stone was transported by locomotives and cars on 36-in. gage tracks. Work was begun in 1892, but in addition to the magnitude of the work, various causes operated to delay its completion till early in 1906. Owing to a change of plans, an embankment and core wall section at one end was taken out and replaced by the regular masonry section, built however of cyclopean concrete instead of rubble.

Steam was generated and used in a large number of detached units. The principal items were eleven 10-ton, 36-in. gage locomotives, thirty-nine steam boilers 10 h.p. to 100 h.p. with total capacity of 1400 h.p., three steam shovels, dredge, fifty-five hoisting engines 15 h.p. to 24 h.p., eleven other steam engines 10 h.p. to 50 h.p., fifteen steam pumps total capacity 20,000,000 gals. per day, three 75-h.p. cableways, three stone crushing plants and eight mortar and concrete mixers. It cannot be stated how much of the above plant was in use at any given time. Undoubtedly an appreciable per cent. of it was duplication, renewals, reserve, in repair shops, etc., and could not be counted in average plant employed. Also some items as pumps, steam shovels and dredge were largely employed below river level and consequently do not figure in subsequent operations. The maximum force employed was 475 men on the dam

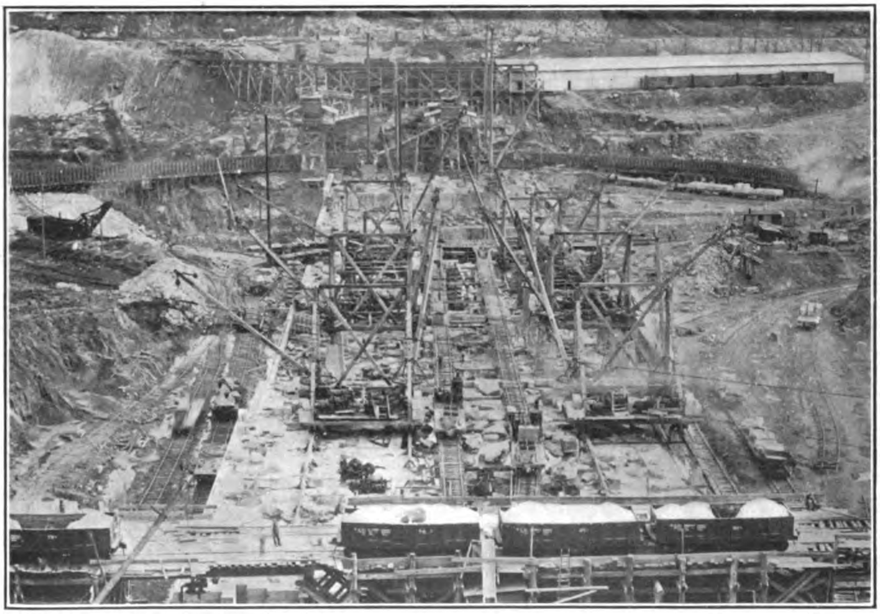


FIG. A.—Kensico dam.

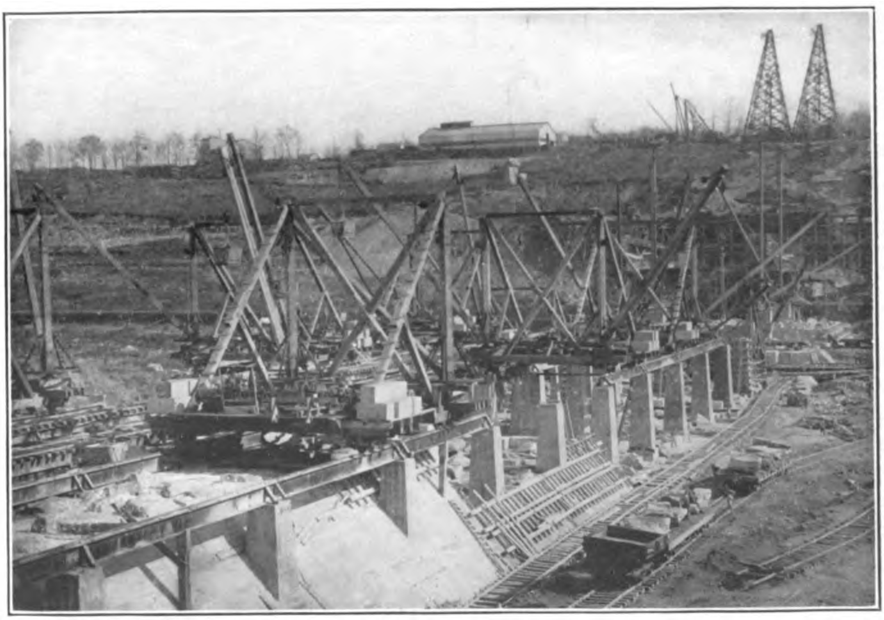


FIG. B.—Kensico dam, showing mounting of derricks and delivery of materials via tracks.

(Facing Page 142.)

PLATE X

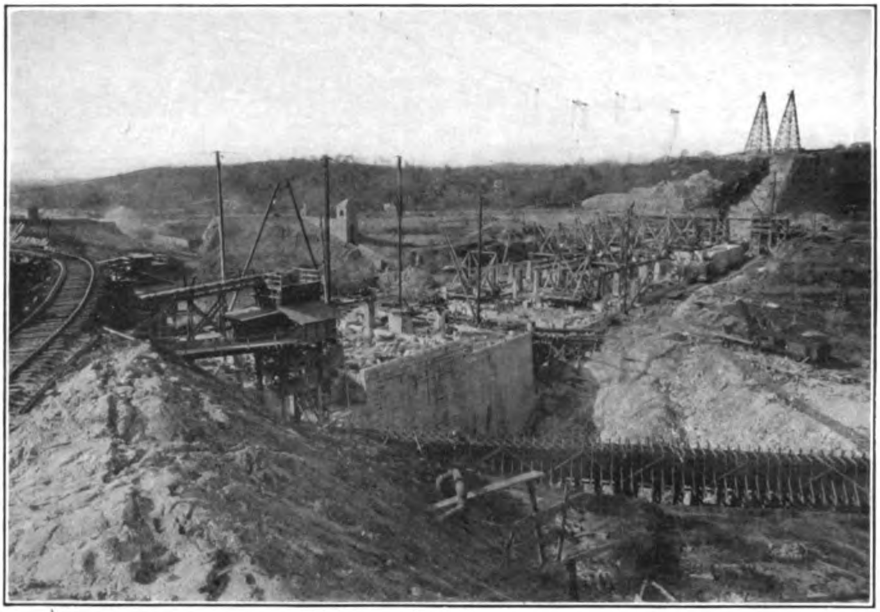


FIG. C.—Kensico dam, temporary diversion flume in foreground.

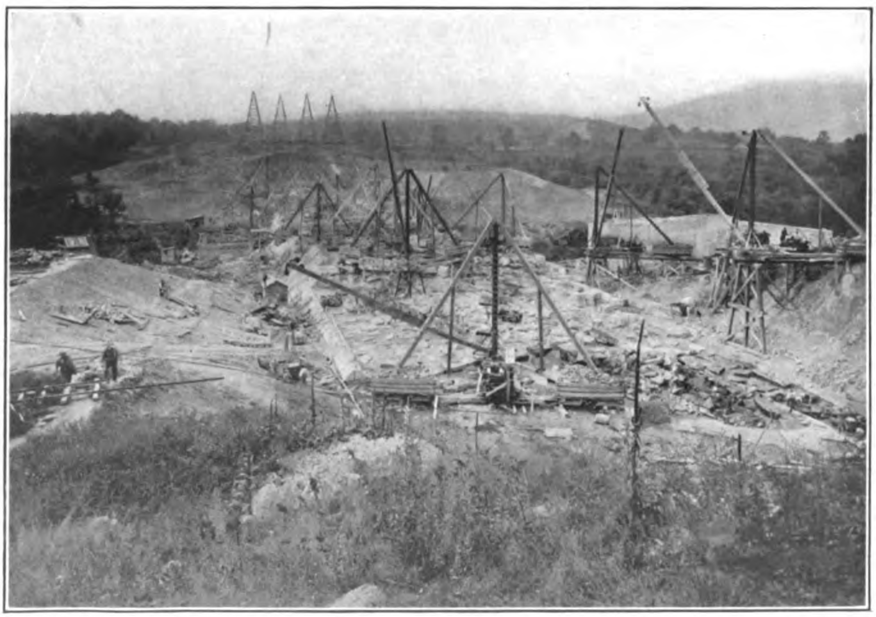


FIG. D.—Olive Bridge dam, showing arrangement of derricks.

and 376 men in the quarry. The maximum month (in 1898) laid 17,186 cu. yd. of rubble masonry. During the maximum month (Aug., 1904) on cyclopean concrete laid 17,000 cu. yd. Work was generally suspended in winter. When work was in full progress the consumption of coal was about 10,000 long tons per year. The total coal consumption for the entire construction period was 90,377 long tons.

At the Ashokan (Olive Bridge) reservoir (recently completed as an addition to the water supply of New York City) the water is retained by a large masonry dam and several dikes of earth embankment and core wall. The masonry dam is 1000 ft. long, and at the maximum is 250 ft. high and about 190 ft. thick at the base, containing 488,200 cu. yd. Each end is continued by a core wall embankment so that the total dam is 4800 ft. long. North and east of the main dam is a long dike, sections of which are known as West Dike, West, Middle and East portions of middle dike; further east is the East Dike and at its east end is the waste weir. North of the West Dike is a weir dividing two portions of the reservoir; this consists of 1100 ft. of masonry for an overflow section and about the same length of embankment with core wall. The total length of dams and dikes is 3.8 miles. The quantities of main items involved are 2,055,000 cu. yd. earth excavation, 425,000 cu. yd. rock excavation, 800,000 cu. yd. of concrete and cyclopean concrete masonry, and 7,360,000 cu. yd. of earth and rock embankment.

The main power plant, near the north end of the Masonry dam, consists of five 265-h.p. Babcock & Wilcox boilers, two 500-h.p. air compressors (formerly used at the Wachusett dam) each with a capacity of 3310 cu. ft. of free air per minute, two other similar machines each with a capacity of 2500 cu. ft. of free air per minute; also two generators for lighting purposes one 250 volt 140 amp. and the other 125 volt. 80 amp. The compressed air is piped to all parts of the work. From the dam the quarry is reached via about 3 miles of double-track standard-gage railroad with grades of 2½ per cent. down from the quarry and 1 per cent. up from the low point to the dam; nine 40-ton to 60-ton locomotives are in use.

The stone and concrete are brought under four 15-ton Lidgewood cableways of 1534-ft. span, mounted on towers 90 ft. high, which travel 600 ft. up and downstream. They deliver all of the materials to the derricks on the masonry. In the quarry are two crushers, a No. 7½ and a No. 5 McCully. At the main crushing and mixing plant near the head towers are one No. 9 McCully crusher and two No.

6 Austin crushers, four 2½-cu. yd. cube mixers, screens, elevators, conveyors, etc. Also near the head towers is the yard 500 ft. long × 200 ft. wide for the manufacture and storage of concrete facing blocks. The stone at the quarry is a bluish-gray sandstone occurring in horizontal layers up to 3 ft. or 4 ft. in thickness. The quarry face is some 1200 ft. long and 30 ft. to 40 ft. high, worked in two lifts by Rand air drills which are run by air from the central plant. The output is about 1000 cu. yd. per day. In the quarry are ten derricks, in the concrete block yard are ten, at the crushing and mixing plant two, and on the masonry sixteen. All derricks are run by double 7-in. × 10-in. cylinder double-drum engines.

Employed on the various dikes and weirs are seven or eight steam shovels, about a dozen steam rollers, two Western graders, two road machines, two traction engines, two narrow-gage locomotives, two cableways, several derricks, crushers, mixers, etc. Although air is piped to all except a few of the more remote parts of the work, coal is of course burned under a number of detached boilers in addition to the steam shovels, locomotives, rollers, etc. Power is supplied to carpenter, machine and blacksmith shops and to a number of pumps. The main pumping plant has a capacity of 800 gal. per minute with a lift of 230 ft.

During the month July 21 to August 20, 1909 progress was as follows:

Earth excavation	48,470 cu. yd.
Rock excavation	4,455 cu. yd.
Embankment and refill	114,230 cu. yd.
Concrete masonry in core walls	17,016 cu. yd.
Cyclopean masonry	17,000 cu. yd.
Concrete blocks set	1,010 cu. yd.
Concrete blocks manufactured	4,010 cu. yd.

At this rate of progress, the above-described plant, central station and detached units, consumed about 1400 tons of coal per month. During higher rates of progress attained since, the coal consumption was about 2000 tons per month.

At the Kensico dam the materials are all assembled and brought within reach of the derricks by means of standard-gage locomotives and cars. The quarry is situated about ¾ mile from one end of the dam, but in order to overcome differences of elevation through which the materials must be handled, there are several switchbacks in the system of tracks and the average total haul for the stone is some 2½ miles. The haul for the sand is about 4 miles additional. The sand cars are filled about two-thirds full and hauled to the quarry where they are further filled to their capacity by screenings drawn from bins at the main crushing plant. They are then hauled to the gravity mixers at the dam of which there is one at each end, and dumped into bins. The large stones are loaded by derricks. The material going to the crushers is largely loaded by steam shovel. There are some five or six quarry openings, totaling perhaps ½ mile of face and employing twelve derricks. At the crushing plant in the quarry is installed the following: one large jaw crusher with opening 5 ft. × 7 ft., to take 10-ton stone, swing jaw 13 ft. × 7 ft., two 12-ft. diameter 15-ton flywheels peripheral speed 3400 ft. per minute, overall size 24 ft. × 15 ft. × 13 ft. high, total weight 225 tons, run by a 300-h.p. motor; a second jaw crusher with opening 3 ft. × 6 ft. flywheels 8 ft. in diameter peripheral speed 5000 ft. per minute, overall size 13 ft. × 13 ft. × 9 ft. high, total weight 105 tons, run by a 150-h.p. motor; a set of 60-in. diameter × 30-in. crushing rolls operating at 50 r.p.m., capacity 300 cu. yd. per hour reducing from 4-in. size to 2-in., run by a 100-h.p. motor; elevator and screen run by a 50-h.p. motor and two conveyors 5 h.p. and 15 h.p.

In the quarry are employed both Temple drills and ordinary large sized pneumatic drills, the latter supplied from three compressors with a combined capacity of 4500 cu. ft. of free air per minute, in the central power station, at which are also transformers to step down from the transmission line voltage. At the concrete block yard is an elevated platform, mounted on rails, spanning the space occupied by block forms and carrying several small mixers which discharge directly into the forms. A traveling crane stacks the blocks when they have hardened sufficiently to be handled, and also loads them on cars for transportation to the dam. Installed on the dam for the purpose of setting masonry are thirteen derricks of the same type as those in the quarry each with a 75-h.p. induction motor handling the fall and boom fall lines and an additional 11-h.p. motor for operating the swinging gear. The derricks on the masonry lift the stone and some of the concrete from cars running along each side of the dam. At present the lift is 20 ft. to 25 ft. The purchased alternating current operates the drills, the derricks in the quarry, the crushing and screening plant, the concrete block yard, the derricks on the dam and doubtless some miscellaneous services as shops, water supply, lighting, etc. The current performs no part of transporting the materials from sand pit and quarry to the crushing plant or to the dam. The two 1861-ft., 10-ton, traversing cableways with 400-h.p.

engines, although at one time used on the excavation, are now used only to handle and move plant. The mixers are of the gravity type. For the month ending Sept. 24, 1913, 760 cu. yd. mass concrete, 49,850 cu. yd. cyclopean concrete, 2630 cu. yd. concrete blocks or a total of 53,240 cu. yd. of masonry was laid, using about 312,000 kw.-hours of electric power as metered at line voltage. During the above period the drills performed more than a corresponding amount of work as many holes are drilled ahead of the other quarrying operations. The block yard also manufactured blocks ahead of requirements. On the other hand, the output of the crushing plant was somewhat below the normal corresponding requirement as a large stock of crushed stone that was on hand early in the season was being exhausted.

A comparison between any of the above performances must necessarily be very rough. Nevertheless they may be of interest and as fair to one performance as to the other. Thus at the New Croton there was a large amount of pumping but it was probably completed before the time when the work was in full progress. The stone was hauled nearly or quite twice as far as at Olive Bridge, and a large part of it was delivered on cars at such an elevation that the derricks had to raise it considerably; on the other hand (until later when the embankment and core wall were replaced by cyclopean concrete) no crushing was done as the rubble was entirely laid up with mortar. Assuming that during the winter season little work was done and only a small amount of coal used, then the 10,000 tons per year might mean 1100 to 1200 tons per month during the busy season. At Olive Bridge dam we cannot say just how much of the coal consumed was chargeable to work not connected with the masonry dam, but it was certainly considerable. Roughly it might be said that at Olive Bridge nearly twice the masonry progress was accomplished as at New Croton with the same amount of fuel. Something like the above difference would be expected from the fact that at the New Croton the fuel was burned in a large number of small detached boilers instead of at a central power plant; also from the fact that the materials were delivered at a low elevation. With the quarry 6 miles distant and outside of the reservoir, it would seem that a route might have been selected such that the stone might have been delivered at the top of the dam, and a comparatively small amount of power subsequently used to deliver it via cableways to the derricks. However, at that time the cableway was not appreciated or worked to the extent that it has been since. One further difference in conditions should be noted, that at the New Croton the quarrying methods had to be such that a large percentage of face stones could be gotten out. This resulted in a higher cost of quarrying and quite possibly accounted for additional power more than would be used at Olive Bridge in the block yard.

To compare the two dams where electric power has been used: At Roosevelt the quarry derricks quarried the rubble and face stone, did necessary handling and turning for the cutters, passed the stone and waste to points under the cableways. Cableways delivered all material to the masonry derricks. Mixing was done by power instead of gravity. Cement and sand were brought to the dam via tramway. At Kensico the electric power may be credited with quarrying, loading cars (except that much of the loading of crusher material is done by steam shovel), crushing, making the face blocks and setting the masonry. All transportation was done by steam locomotives. The shop, camp and miscellaneous services to the credit of the electric power in each case may be roughly taken as proportional. Then at Roosevelt quarry and masonry derricks, crushing, mixing, cableways, cement and sand tramway compare with Kensico quarry and masonry derricks, crushing and concrete block yard. At Roosevelt were laid 18,328 cu. yd. per month with 148,512 h.p.-hours of power delivered to distribution system; at Kensico were laid 53,000 cu. yd. of masonry with 312,000 k.w.-hours purchased, which at say 97 per cent. transformer efficiency = 405,538 h.p.-hours delivered to machines. In brief, Kensico laid 2.89 times the Roosevelt masonry at a consumption of 2.73 times the power.

At the Roosevelt dam the quarries were much more favorable but on the other hand more large stone was used, including the face stone —40 per cent. of the masonry as compared with 27 per cent. at Kensico. Roosevelt mixed the concrete and transported all materials. Kensico did no mixing (except in block yard), no transporting and not all of the loading in the quarry. Some power was lost through the system of delivery of materials whereby they have to be subsequently elevated again, although as yet the masonry is not high enough to make this a serious item.

When considering either of the above comparisons it should be borne in mind that the New Croton masonry contained 50 per cent. of large stone, the Roosevelt 40 per cent., and the Olive Bridge and Kensico 25 per cent. to 27 per cent. Other things being equal, the larger percentage of large stone involves additional quarry cost and probably power; also in setting the masonry it involves additional cost and undoubtedly more power. It is more work for the derrick to set a cu. yd. of stone, as frequently stones have to be lifted and set a second time; probably also concrete blocks are more easily set than face stone.

CHAPTER XI

ASSEMBLING MATERIALS, CRUSHING AND MIXING

For a small dam the assembling, handling and placing of the materials may well be merged into what might be called two operations, hauling materials to site, and building the dam. To elucidate: Several derricks may cover the entire work, unload the materials from teams, transfer to and from mixer and crusher, pass from one to another along the work and do all the placing in final position. In fact the small yardage, and the limitation thus placed on the amount of advisable expenditure for plant, points to some very similar procedure. However, with increase in size of dam it becomes successively questionable, then advisable and finally imperative that these various operations should be sharply differentiated, and that each should have its plant and methods suitable for the particular case and for the yardage involved.

Whatever the origin of the materials (stone, sand and cement) entering into the construction of the dam, they must be assembled at one point before being sent onto the wall. It is true that the large stone may be sent direct from the quarry to the dam irrespective of the manner of transportation of the other ingredients. However, it will be found almost invariably that economy of plant and operation require that the large stone join the concrete and mortar at some point of assembly, and that it take the same route thence to the dam.

At some point between the quarry and the dam a certain portion of the quarry product must be crushed for concrete. At the same or another point this crushed stone must be joined by the (pit or manufactured) sand and the cement for the purpose of mixing concrete. While the details of crushing and mixing plants may vary widely, and indeed not be identical upon any two pieces of work, still certain general rules and considerations may be borne in mind in laying out the routes for the materials and the details of the process.

First as to the point of assembly of the materials: This should be kept up at such an elevation relative to the dam that gravity can play a large part in the passage of the materials through crushers, screens, storage bins, measuring hoppers, mixers, etc., and that the emerging concrete or mortar will require the least subsequent elevation on its way to the dam.

One factor which naturally enters into the problem is the location of the points of origin of the materials, *i.e.*, of the quarry, sand pit and cement storage, and of the routes from such points. The assembly point might conceivably be a deciding factor in the choice between alternate quarries or sand pits; a longer distance might be economical in order to obtain easy or descending grades. If at the point of assembly, or between it and the dam, the materials must be elevated there is little choice as to the exact stage at which it may be done. Thus stone brought on cars is dumped onto the crusher platform—the crushed stone may be screened and elevated or elevated and screened. If the former, an elevator is required for each size product; if the latter, one elevator may have sufficient capacity and the screen is light enough to be easily mounted on top of any storage bin.

Again, if the introduction of a bucket elevator between the crusher and the mixer will reduce the height through which a cableway must hoist the concrete before conveying it, introduce it by all means as the time of the cableway it too valuable, to say nothing of the weight of the concrete buckets or skips.

If considerations of available room render it necessary, the operations of crushing, screening and storage may be performed at the quarry. In this case, however, there enters the question of probable duplication of bin capacity, as there must be storage below the crushers and also storage above the mixers. But one storage bin is required if the two operations are performed at one point with or without intermediate elevation. The performance of the two operations at two points means an interruption to the continuity of the motion of the material, the interposition say of an additional train service between the storage at the crusher and the storage at the mixer. The actual lengths of haul involved probably are so short that such an interposed train service would be equivalent to doubling the transportation cost of the material thus handled.

A point that is occasionally overlooked is that the crushing and mixing plant is the point of restricted channel in the flow of the material. The product of many units along an extensive working face in the quarry converges at this plant to be later distributed among many units over the working area on the dam. The disablement of a quarry or masonry unit is a local matter, insignificant in its effect upon the progress of the work, but a breakdown at the crushing and mixing plant is liable to be a more serious matter. Ample crushing and mixing capacity is indeed usually provided, but the provision of this capacity in one unit is apt to lead to the weakness that a breakdown stops the entire work. Pertinent to this situation is the old adage "Don't put all your eggs in one basket." Assume that the estimated required crusher capacity is 50 tons per hour. Then it may be better to provide two crushers of 35 tons or 40 tons capacity each than one of 60 tons or 70 tons. Important elements of the problem are the amount of storage capacity, the design of the plant with regard to rapid repairs and the completeness of the stock of repair and duplicate parts kept on hand. If the material must be elevated between the crusher and the mixer, and from economy in bin capacity the storage is all upon one side of the elevator, then the continuous operation of the elevator is as vital as any more prominent feature of the plant.

On pages 204 and 205 is discussed the manner in which unit costs increase with decrease of output, and how the cost of certain operations is nearly in proportion to the time instead of the output. As this is peculiarly the case with the operation of a crushing and mixing plant, the aim in the design of the plant should be to make the fullest use of mechanical devices and to reduce the number of attendants to the lowest practicable point.

Gates controlling the flow of material, measuring and dumping devices should be made, as far as possible, to operate automatically; or else bring the various indicating and controlling mechanisms together so that they may easily be handled by the minimum number of operators. The possible saving may be easily figured thus: Assume that at the various estimated rates of progress the operating force will be required for say 700 working days and that their labor may cost $2.00 per man per day; then a plant expenditure for automatic or labor-saving devices of $1000 for each man displaced is easily justifiable. The design and arrangement of such a plant particularly in the minor features of conveyors and elevators depend upon so many circumstances, like capacity, topography, access to and between different parts of the work, design and location of other plant, etc., that one design could never be copied for another job. Still one very extensive plant may be here illustrated and described, particularly as its cost is given.

Description and Cost of Modern Plant.—The plant (see Fig. 26) is at the Elephant Butte dam, and is complicated by the inclusion

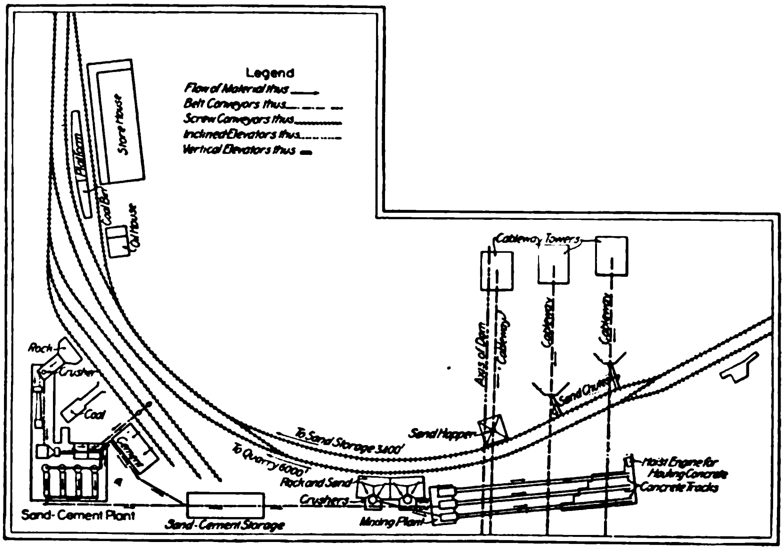


FIG. 26.—General lay-out of construction plant at Elephant Butte dam.

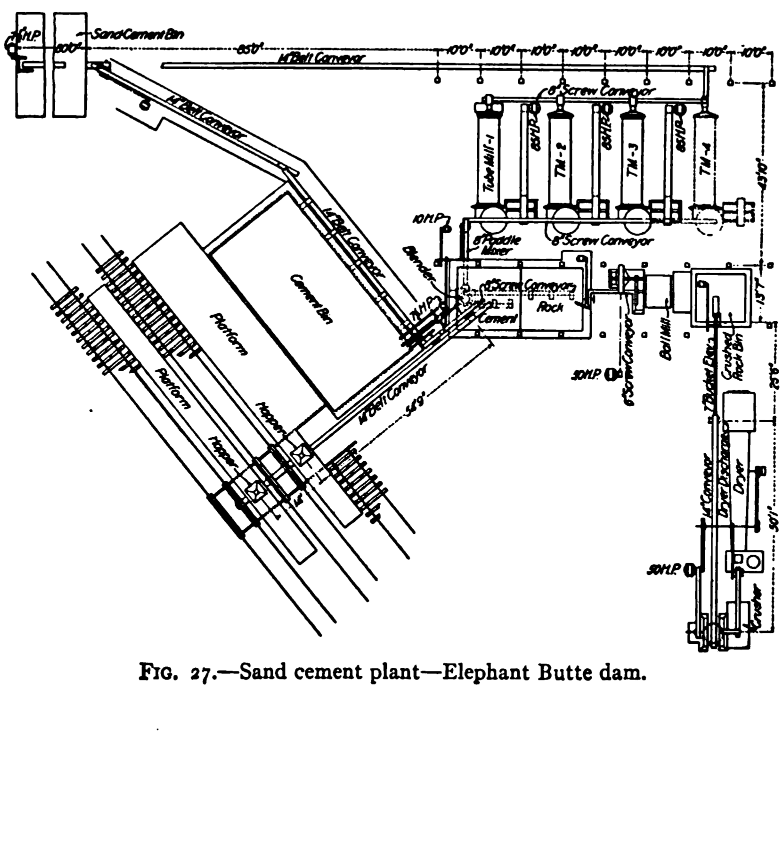


FIG. 27.—Sand cement plant—Elephant Butte dam.

of a plant for the manufacture of sand cement. The description is abstracted from an article which appeared in *Engineering Record*, October 4, 1913.

Sand Cement Plant (see Fig. 27).—Rock from the sandstone quarry goes to a gyratory crusher and thence, either directly or through a rotary dryer, to a bin above the ball mill. The product of the ball mill goes to the ground rock bins. The cement is received in sacks on cars, the sacks emptied into hoppers and the cement conveyed to a storage bin next to the ground rock bin. From these two bins the

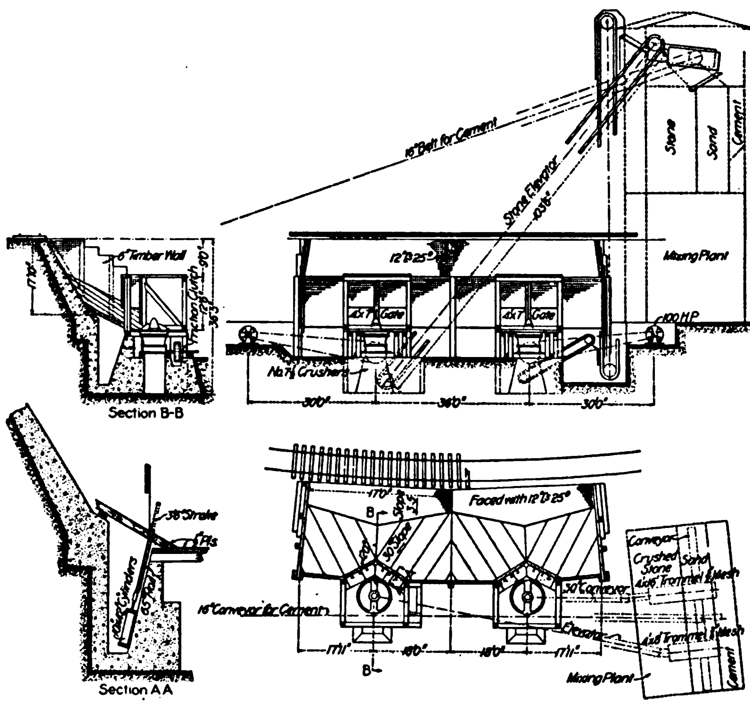
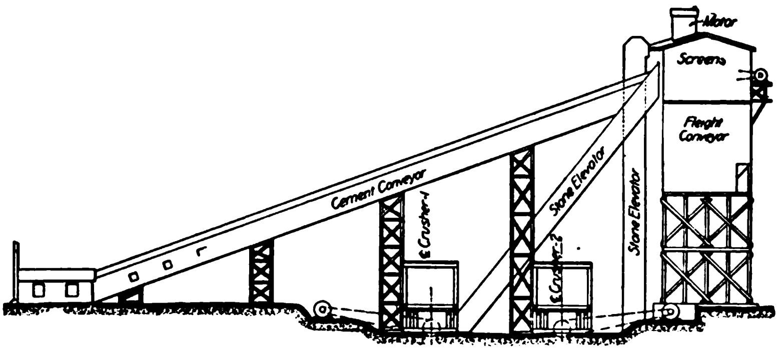


FIG. 28.—Crushing plant—Elephant Butte dam.

ground rock and the cement are taken via screw conveyors to an automatic mixer which can be set for any desired proportion; thence by a mixing conveyor to a long conveyor delivering to four hoppers which feed the four tube mills where the final grinding is done. The product of the four tube mills goes onto a conveyor which delivers onto an inclined belt conveyor leading to the top of the storage bins. The cost of the sand cement plant, including machinery, buildings, erection and overhead charges was about $56,000.

Crushing Plant (see Fig. 28).—This consists of two No. 7½ Symes gyratory crushers, elevators, screens and bins. Each crusher has a capacity of 100 tons to 150 tons per hour and, with its elevator,



Elevation of Crusher and Mixer Plants and Conveyors

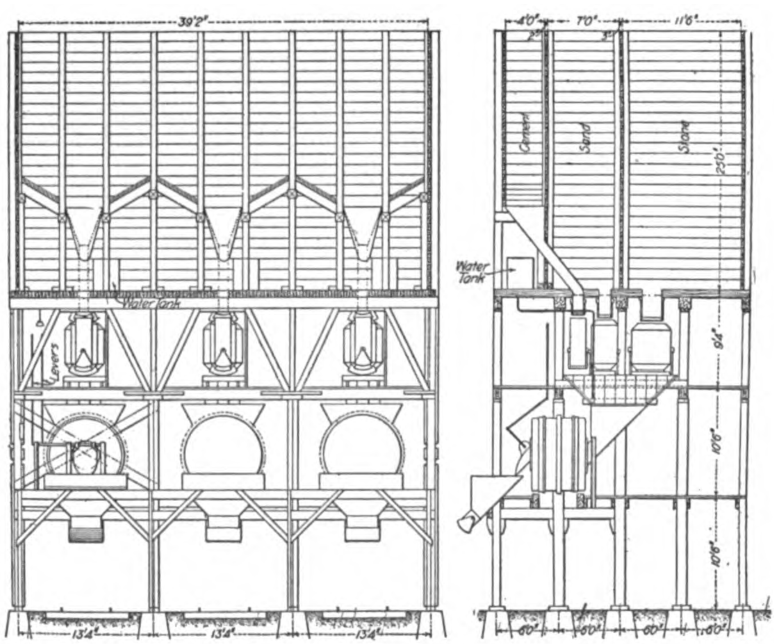


FIG. 29.—Mixing plant—Elephant Butte dam.

requires a 100-h.p. motor. Trains of 6-cu. yd. cars dump onto the crusher platform 20 ft. below the track. Gates, operated by plungers underneath, regulate the delivery to the crushers. One crusher discharges direct to an inclined bucket elevator, the other to a short platform conveyor which discharges into a vertical bucket elevator. The two elevators deliver to screens which separate the sand and stone, delivering them to their respective bins. A house was erected over the crushers. Two men are required for each crusher. The total cost of this crushing plant, including bins completely installed ready for operation, was $31,500. The depreciation cost charged against crushing rock, on the assumption that the plant is scrapped when the job is finished, is 10.6 cents per cu. yd. The labor cost of production is approximately 12 cents per cu. yd.

Concrete Mixing Plant (see Fig. 29).—Three 80-cu. ft. Milwaukee mixers discharge their product into hoppers holding two or three batches. This practice is followed in order that the operation of the mixer may not be interrupted by the non-arrival of the skip, and (which is as important) that the travel of the skip need not wait on the operation of the mixer. Above each mixer are three measuring hoppers like truncated pyramids connected by a rectangular section which telescopes to change the capacity. An automatic hydraulically operated interlocking mechanism opens and closes the gates (a) from bins to measuring hoppers, (b) from measuring hoppers to receiving hopper and (c) from receiving hopper to mixer. The amount of water per batch is determined by the setting of a siphon attachment to the supply tank.

Three tracks run from the mixing plant at right angles to and beyond the line of the cableways. At the opposite end of the tracks is a three-drum hoist handling a car on each track. With this arrangement it is possible for each mixer to be operated by one man though two seem to be necessary to secure uniformity of product. The total cost of the mixing plant, including all machinery, storage bins, elevators (not including crushing plant) and haulage system, is $57,800. The depreciation cost is estimated as 13 cents per cu. yd. The operating labor cost at full capacity will be 8 cents per cu. yd., although operation to date, at a small fraction of the capacity, has cost 14 cents.

CHAPTER XII

TRANSPORTATION OF MATERIALS

In any system for transferring the materials to the dam, the points to be desired are the following:

First.—That the motion be as direct and continuous as possible because change of direction or interruption to continuity involves multiplication of plant, loss of time, loss of power and increased expense.

Second.—That it shall interfere as little as possible with the operation of portions of the plant whose proper functions are something else.

Third.—That its capacity be ample for the rate of progress desired, that one installation may serve the entire work from beginning to end without renewal, moving, remodeling, or any radical change of methods.

Depending largely upon the length and height of the dam, and to some extent upon the point of origin of the materials, either one of two common methods may be employed, namely, cableways spanning the valley or a system of tracks along one or both sides of the dam. We will discuss the conditions determining the choice of either method and also the pertinent and allied matter of some conditions governing the capacity of derricks.

Cableways.—Cableways have been employed on spans exceeding 2000 ft., and they are the most economical machines possible in a large number and variety of cases. Besides handling the materials with celerity and a minimum consumption of power they are available for many incidental operations such as erecting, moving, loading or unloading heavy items of equipment or material. For moving derricks on the masonry the "sky hitch" is the acme of simplicity and efficiency; in fact, it pays for itself in performing that one class of service. It is needless to say that the road or railroad which gives access to the work should run under the cableway.

Cableways may have fixed anchorages with end supports or towers of a height depending on conditions. Again, if the service seems to require and the topography at the ends of the dam permits,

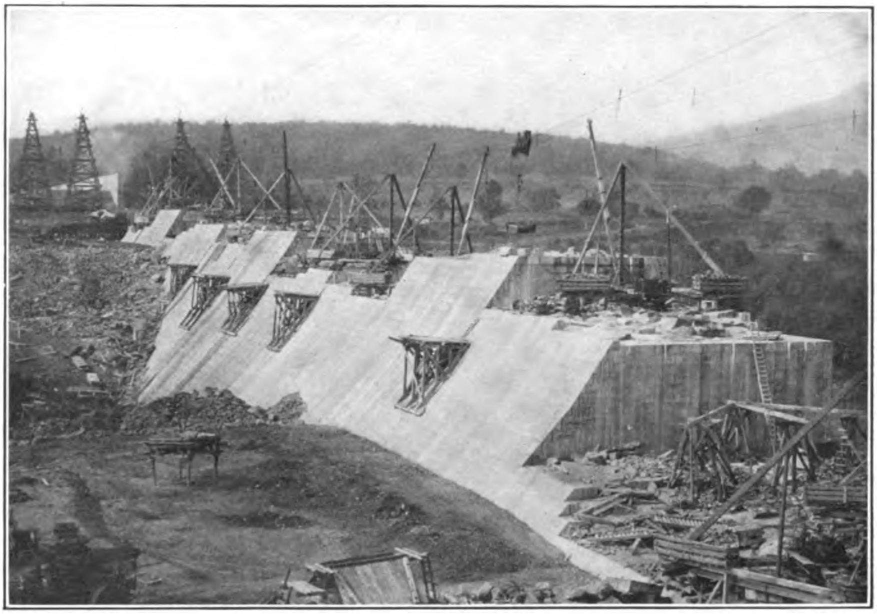


FIG. A.—Olive Bridge dam. Note vertical steps at expansion joints, also landing stages on down stream face.

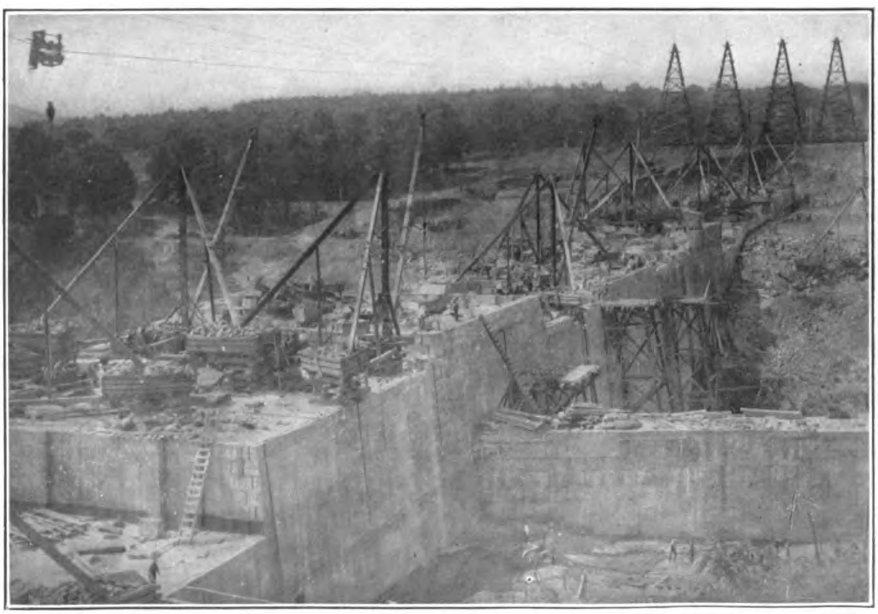


FIG. B.—Olive Bridge dam. Note expansion joints, arrangement of derricks, and landing stages on upstream face.

(*Facing Page* 156.)

PLATE XI

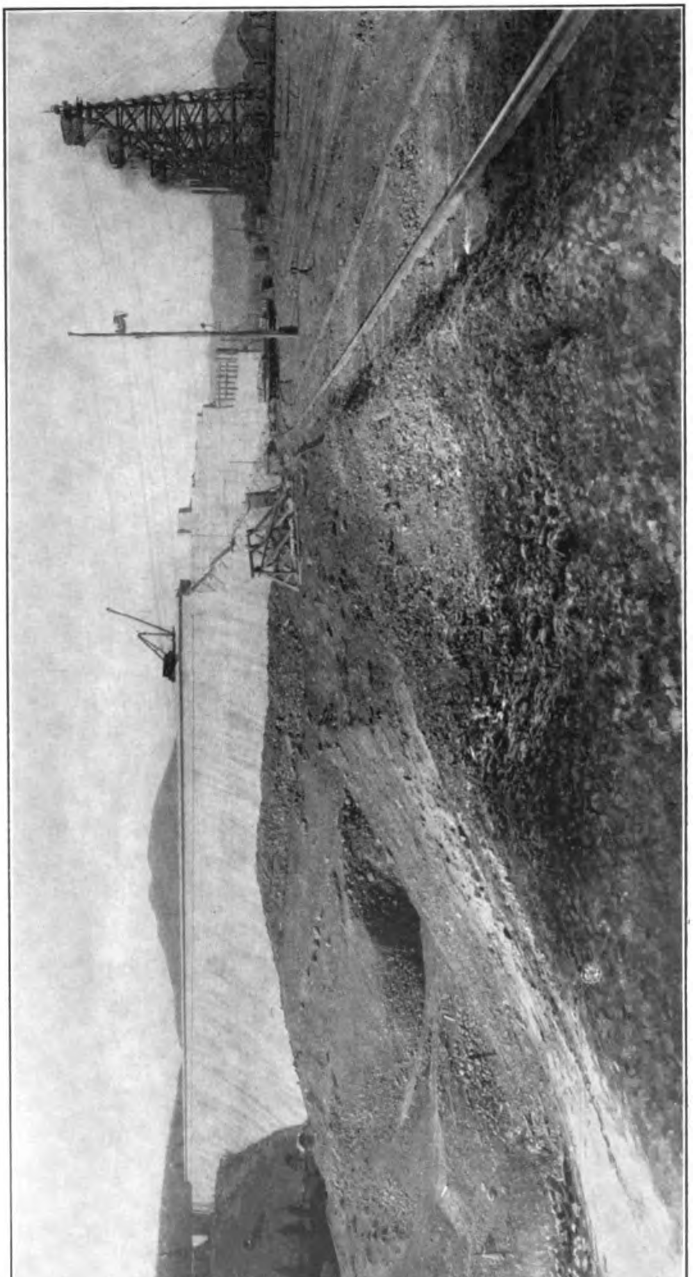


FIG. C.—Olive Bridge dam, practically completed.

they may be arranged so as to traverse up- and downstream, for which purpose the towers and anchorages are mounted on trucks running on several tracks. If it is possible with any reasonable height of end towers the cableway should be high enough to clear the top of the proposed work, as it is not only a nuisance and a delay, but also adds to the cost of the work, to let a cableway go out of commission and have to devise some other method to handle materials for the top of the dam.

The sag of a cableway is about 5 per cent. of the span. To this add about 25 ft. for carriage, fall blocks, hooks, extra sag under load and margin. From these data may then be figured the required height of towers. As the sag is 5 per cent. of the span, tower height can be saved by setting the tower further back when the ground rises steeper than 10 per cent. An accurate profile is necessary in order to figure intelligently on the desirable span and tower heights. In case it is desired to figure on cost of grading off a bench at each end of the dam for the purpose of installing a traversing cableway, it will be necessary to have, in addition to the profile, plans showing the topography for such area as might be affected by the benches.

A width of bench approximately equal to the tower height will be necessary. For the dam proper, traversing cableways will hardly be necessary. The number of cableways necessary to accomplish the desired rate of progress, placed side by side, will adequately cover the work. However, it will be well to figure on a traversing cableway when there are appurtenant structures like gate houses or other adjacent work involving the handling of large quantities, and if a saving sufficient to more than pay for grading the benches may be effected by handling those quantities by cable. This is especially true if there is a large amount of material to excavate from the foundation.

It may be mentioned that for the purpose of handling the ordinary materials of construction it is not practicable (although possible) to move the cableway laterally while engaged in transporting longitudinally; but when necessary this may be done in the case of moving plant or other special loads. In ordinary operation the traversing cableway can simply occupy one of a wide range of positions, and for any given destination of load the cableway occupies a position over it, and the stone, concrete, etc., are brought to a point under the cable in the same manner as to a fixed cable.

The matter of adequate anchorage should receive careful attention from an experienced man. The details and cost of course may vary widely with different material anchored in, for a rough estimate say $100 to $400 per anchorage. Anchorage in the case of traveling cableways is of course to the rear of the moving base upon which the

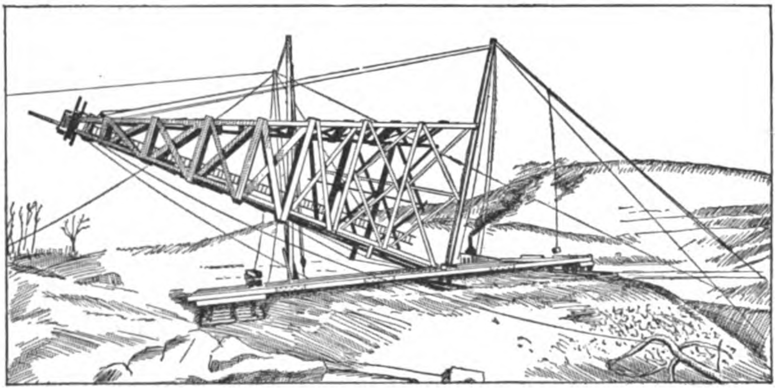


FIG. 30.—Erection of cableway tower.

tower stands, which must be weighted to hold it. Details of such anchorage, together with the necessary weight, should be shown on a plan of tower usually furnished with the cableway.

Obviously several towers may be mounted on the same set of

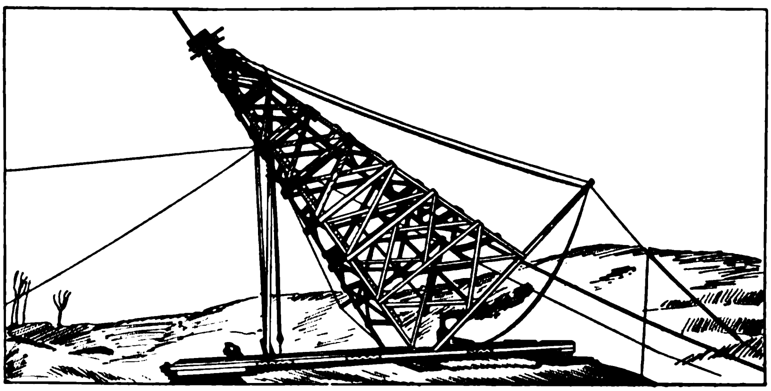


FIG. 31.—Erection of a cableway tower.

tracks, so that the item of grading and tracks is the same for one cableway as for several. Usually the towers are framed laying on the ground, to be erected in assembled position later. (See Figs. 30 and 31.) While the framing and putting together may be done by any good carpenter, the erection of the towers, stringing and rigging the cableway should be under the direction of an experienced rigger; but it is a short and inexpensive operation. It is necessary to bear in mind the required spread of tower base in order to give lateral stability as this governs the minimum distance apart at which the cables can operate. In some cases involving tall towers it is conceivable that one tower must be placed behind an adjacent one in order to bring the two cables nearer together than would otherwise be possible. In special cases two or more cables may be mounted on one pair of towers.

A recent very ingenious mounting of cableways in Germany is illustrated and described in *Engineering News* for November 20, 1913. Briefly, the cableways are mounted on masts instead of towers, and some range of lateral position is secured by manipulation of the side guys which stay the mast. Practically the same scheme was tried some fifteen years ago with a cableway used on some fortification work at Tybee island, Savannah. It is possible that this scheme may be developed so as to become quite useful in certain situations, where only a limited lateral range is necessary, or where a cramped situation at the terminals might preclude the use of ordinary towers. It should be far less expensive than grading off a bench and constructing traversing towers. The masts might be of steel, trussed as required for the length, and mounted on a foundation suitable for such a concentrated load.

In figuring upon a cableway installation the required capacity in two respects should be carefully borne in mind—*first*, the maximum load which it may be desired to handle, *second*, the yardage per month, or in other words the number of trips at the proper average working load, which is usually assumed as much less than the maximum.

The heaviest load may be in connection with moving some item of the construction plant, or it may be some valve, machine or casting entering into the permanent work. A very ordinary safe capacity for a cableway is 20 tons; loads up to 35 tons may be safely carried by slinging them between two cableways working side by side.

Cableways with two cables close together and one carriage have handled 35 to 50 tons when especially designed for such service.

In the case of an occasional special load which it is desired to pick up from or deposit at some lateral distance from the cable, lateral guys or even another cableway may be used to pull the load over. Depending on the weight of the load it may be pulled over till the fall line is 15 deg. or 20 deg. out of plumb laterally. The operation, however, should be conducted by some person familiar with cableways generally and that one in particular. Generally speaking, ordinary traveling towers on the same tracks may be brought near enough together so that the two cables may be safely Siamesed on a maximum load provided the load is slung below the cableways a distance equal to the height of the towers; lighter loads may be lifted much higher. By using some special sling or spreader, or if the load is such that an end may be carried by each cableway, the hooks may be kept far enough apart so as to favor the conditions materially. The number of trips per hour depends very largely upon the celerity of the attendants who hook and unhook the skips. In connection with the possible number of trips per hour or per day, and the cubic yards possible to handle, following are some records of actual performance:

With the two Lidgerwood Cableways at the Wachusett Dam the expert rigger attained a record of 235 trips in 10 hours, or a trip each 5 minutes 6 seconds. Subsequent performances of the regular crew never exceeded 185 trips per 10 hours, or a trip each 6½ minutes, and the average performance required by the practicable rate of construction was less than that. For instance, a day's timing of a cableway showed 86 trips in 627 minutes; average trip 7½ minutes; ten shortest trips in 36 minutes, ten shortest consecutive trips in 52 minutes. At the Roosevelt dam during the month of March, 1909, two Lidgerwood cableways in twenty-nine eight-hour night shifts took onto the dam 7900 cu. yd. of large stone (number of trips not known), and also did the necessary moving of derricks on the masonry. Obviously they could not be and were not worked at night up to their daytime capacity. In twenty-eight eight-hour day shifts they made 5003 trips with concrete mortar and spalls. This was an average of one trip each 5.37 minutes, without making any allowance for some unavoidable small miscellaneous services. The average day trip was approximately as follows: hoist 75 ft., convey 350 ft. to 400 ft., lower 125 ft. and return with an empty skip. The materials thus handled during the month were sufficient to build 18,328 cu. yd. of masonry, of which 50 per cent. was stone and 50 per cent. mortar and concrete.

At the Cross River dam two 1250-ft. span cableways handled in one month material for 18,500 cu. yd. of cyclopean concrete masonry composed of 33 per cent. large stone and 67 per cent. concrete, supplying six derricks, the performance of which was 10.8 cu. yd. per derrick per hour. The maximum ten-hour record for the two cableways was 257 2-cu. yd. buckets of concrete and mortar and some stone, amount not stated; or assuming 275 trips, it would be equivalent to about four and one-third minutes per trip.

At the Olive Bridge were four cableways of 1534-ft. span. The monthly record for the four cableways, eight-hour day, was 35,300 cu. yd. of masonry, of which 25.3 per cent. was stone and 74.7 per cent. was concrete. The eight-hour record for the four cableways was 404 2½-cu. yd. batches of concrete plus 160 cu. yd. of concrete blocks and 400 cu. yd. of stone. Assuming that 3 cu. yd. of blocks or stone were carried per trip, it would mean a trip each three and one-fourth minutes or 591 trips in 1920 minutes. The record for one cableway was 226 batches of concrete and 57 cu. yd. of stone, say 245 trips or at somewhat better than a trip each two minutes. This maximum day's record for one cableway may have been for a cableway where the trip was shorter than the average trip for the four. Hourly or even daily records, although interesting, are of less value than monthly records in estimating the amount of work that a cableway may be expected to accomplish. The foregoing records (disregarding the Wachusett dam where the methods were so different) show a remarkable agreement at about 9000 cu. yd. per month per cableway.

At all of the above-mentioned cases it seems to have worked out that it was found advisable to deliver some stone at night. This is natural as it results in economy of time if the cableway can work regularly with loads originating at one point, *i.e.*, no time is lost in traveling between a point where an empty skip has been left and a point where another load must be picked up. Thus in 100 round trips say with concrete from one originating point to the dam and return with an empty skip, there is involved hooking onto, hoisting, conveying, lowering and unhooking 100 loads and the same for 100 return empty skips; while if fifty loads of concrete alternate with fifty loads of stone there is involved, in addition to the above, 100 trips empty between the points of origin of the stone and concrete, or probably 25 per cent. more time required at best. The interference with the continuity or regularity of the operations of the attendants probably results in further loss.

Roughly comparing the above, we find that at Roosevelt four derricks in eight-hour shifts attained an output equal to six Cross River derricks in ten-hour shifts, or 50 per cent. more per hour. The Roosevelt cableways did a larger per cent. of their work at night, and not as much during the day as at Cross River. The Cross River and Olive Bridge cableways apparently made about the same performance. The Olive Bridge derricks, however, laid on an average 25 per cent. more masonry per hour than the Roosevelt derricks, and twice as much as the Cross River derricks.

The low Cross River derrick record for cyclopean masonry containing 33 per cent. stone is probably due to some peculiar condition attaching to that particular work as normally a less percentage of stone to set should operate to increase the yardage per derrick hour. It is probable that the capacity at Cross River was limited by the cableways. Regarding the time required by a derrick to handle stone and concrete, it may be said that usually more time is required to hook onto a stone than onto a skip of concrete. The stone must be held up by the derrick while being washed and occasionally while a loose fragment or a fin is being knocked off, some stone must be lifted and set a second time, particularly if they are being bedded in mortar; further, the average stone is undoubtedly smaller than the average skip of concrete, and so involves a larger number of operations for the same yardage. As illustrating how this may affect the yardage of a derrick, attention is invited to the following record on the Medina dam.

On the Medina dam the concrete was sent out in 2-cu. yd. bottom dump buckets and was handled by five derricks. The masonry contained only 10 per cent. of plums. During June, July, and August, 1912 (not including Sundays), in seventy-nine ten-hour days 107,550 cu. yd. of masonry were laid or 27.2 cu. yd. per derrick hour, with a best monthly record of 40,303 cu. yd. This is a remarkable performance. It is due undoubtedly to the small percentage of stone handled, and the consequent uniformity in the operations of the derrick; it also implies that the derricks were kept well supplied. The derricks were supplied partly by two cableways and partly by cars running on a double track laid along the toe of the dam. No figures are available as to the relative yardage handled by each, so no comparison can be made regarding the cableway performance.

At the Gatun locks of the Panama Canal were installed eight Lidgerwood cableways, mounted in pairs on four sets of traveling towers. The span was 800 ft.; the conveying distance when placing concrete in the lock walls was from 150 ft. to 500 ft.; and the vertical distance 40 ft. to 100 ft. below the origin of the load. "About 2900 cu. yd. of concrete have been placed in the locks in one day of twelve hours by the battery of eight (four duplex) cableways in addition to handling forms and ironwork for the day's work. Three duplex (six) cableways have placed 2700 cu. yd. in ten hours. One duplex (two) cableway has placed 64 cu. yd. in thirty-two minutes. Thirty complete round trips have been made in one hour with one cableway." The concrete was handled in 2-cu. yd. dump buckets.

Delivery by Tracks.—The Kensico dam now being constructed by the New York Board of Water Supply will contain about 1,000,000 cu. yd. of cyclopean concrete, of which about 200,000 cu. yd. con-

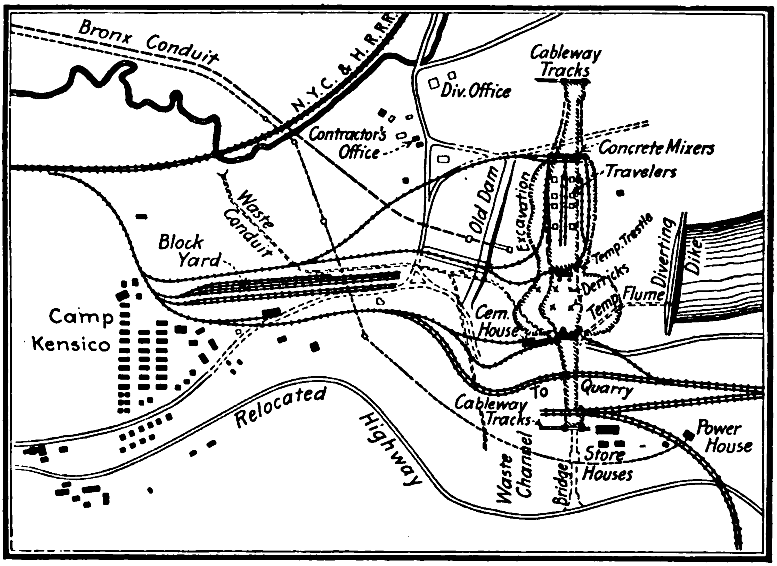


FIG. 32.—General lay-out of construction plant Kensico dam.

taining about 27 per cent. of plums have been laid. Here all of the stone and much of the concrete are delivered by cars and locomotives via tracks on both sides of the dam. (See Plate X, Figs. A and B; also see Fig. 32.) There is a mixing plant at each end. The stone (including all crushed stone) and sand originate at one end. The two 10-ton, 1861-ft. span traveling cableways are used solely for handling plant. The masonry record to date is 53,000 cu. yd. in twenty-seven days, or equivalent to the output of six cableways at previous records of performance.

This amount of masonry was laid in 1913, between Aug. 25 and Sept. 24 inclusive, or in twenty-seven eight-hour days. No account was kept of the actual working hours of the derricks; thirteen derricks were installed on the masonry, and assuming that they all worked full time they would have shown a performance of about 19 cu. yd. per derrick hour. It is probable, however, that the number of derrick hours of actual work was less than the above assumption. The materials were all passed to the derricks by means of cars running on tracks upon each side of the dam. From the mixers, some concrete was also run out via tracks extending out a short distance along the center of the dam from each end.

While in this particular case this method of handling the materials has resulted in a large monthly total yardage, still it is open to some general objections which it may be well to analyze.

First.—If the materials, stone and concrete, originate at the level of the bottom of the valley they may indeed be easily brought on tracks to one side of the dam, and while the dam is in its earlier stages may be brought through or around one end to the other side, so as to be available to derricks reaching over either side. As the masonry grows in height, however, the tracks upon one side must usually be discontinued unless one of several objectionable expedients are resorted to, namely:

(*a*)—Leaving an opening through the masonry.

(*b*)—Climbing around the end by a longer route, probably requiring switch-backs, and in any case involving a heavy labor and fuel cost, as well as a heavy investment in track on which the salvage is practically nothing.

(*c*)—Establishing two quarries with crushing and mixing plants, one above and one below the dam. Even if such a procedure is possible it would be a division of forces and multiplication of plant that could hardly fail to result in a sacrifice in economy.

Second.—In many cases it is desirable or necessary that water be stored in the reservoir long before the dam is finished, thus precluding tracks upon the upstream side.

Third.—When the masonry is started the tracks may have to be some distance away on account of the excavation slopes. As the masonry is brought up and the refill made they may be shifted up near the masonry lines. If the refill is of considerable depth the tracks may have to be shifted laterally or in elevation several times, and they may interfere with the operation of refilling. The change of elevation may involve much work on the tracks for some distance in order to preserve feasible grades.

Fourth.—After the masonry has been built to an elevation above the refill and above the tracks, the amount of work and time involved in reaching over with the derricks and lifting the loads is a consideration which becomes more and more serious as the masonry progresses in elevation above the tracks. On the downstream side, owing to the batter of the face, a derrick on the masonry (*i.e.*, in a position to build masonry) will soon find itself at a considerable lateral distance from a track on the refill. With the usual batter, a practicable length of boom and some allowance for clearance of track, etc., this method would be possible until the masonry was some 75 ft. above the track level. To continue the masonry above that elevation by that method involves one of two expedients: either a trestle along and upon the downstream face of the masonry upon which the cars can be run, and which may require the remodeling of a large part of the track system; or the interposition of derricks upon the downstream face for the purpose of passing material and skips back and forth between the cars and the masonry derricks. Either expedient is expensive and a nuisance, and is open to the objection that it might seriously mar the appearance of the face masonry through rust stains or grease drippings. On the upstream side the masonry derricks may indeed reach over to the tracks at any stage of the work up to completion, although the lift may become such that considerable time of the derrick is thus occupied to the loss of time in setting masonry. Thus for a lift of 100 ft. and a load of 8 tons, a 30-h.p. derrick will require about two minutes for the lift; or say it is desired to build during the day masonry requiring 250 tons of stone and concrete, and assume that the containing skips or buckets weigh another 50 tons (a moderate assumption). This would equal thirty-seven and one-half 8-ton loads and one hour and fifteen minutes actual hoisting time. Hence 16 per cent. of an eight-hour day for the derrick has been devoted to overcoming a difference of elevation easily to be encountered in the construction of a large dam.

Fifth.—If the origin of the stone and concrete be at the elevation of the floor of the valley and delivery be made by tracks and cars, there enters the question of how to come within reach of derricks working near the ends of the dam. It seems to involve a long piece of track, switch-backs, etc., or else cable inclines at each end. If the origin of the materials is at one end of the dam, one of the same two expedients must be resorted to, with the additional disadvantage that the element of original elevation of materials is wasted (*i.e.*, thrown away to be later expensively reacquired) or else high trestles must be built.

Sixth.—If locomotives are used to transfer the cars, it means (in addition to the first cost of plant) that much fuel is burned at very low efficiency, especially, if considerable range of elevation is overcome by grades in the system of tracks. The net load is a much smaller percentage of the total load that must be handled than is the case when delivery to the derricks is made by cableway.

Seventh.—Another very important difference in the operation of the two methods of supplying the derricks requires some explanation. The function of the cableway and the function of the derrick are two entirely different things; they do not conflict or overlap. Neither the cableway nor the derrick necessarily wastes any of the other's time. Although they work together the proper coordination of their duties does not require such coordination in point of time as does the other system. In other words, the cableway leaves a load at a certain point within reach of the derrick and picks up an empty which the derrick had previously left at that point. "That point" means some area accessible to both, which for the time is not being built upon, and which must be available under either system. The derrick, whenever its function of building masonry requires or permits, leaves an empty at that point and takes up the load left by the cableway.

Now under the system of delivery by cars on a system of tracks, the train or the car requires for the completion of its function the assistance of the derrick. The train cannot leave its load, pick up empties and return. If the assistance is rendered at a certain moment it may interfere with the function of the derrick, and if at some other moment it means that the train has waited; with a loss of economy in either case. Further, it means that there must be some unoccupied space upon the train, possibly upon each car, for otherwise the derrick must make its first trip to get the load and a later one to place the empty, making two trips instead of one. Such interference with the function of the derricks may not be serious on portions of the work. Very little time is required to dump a bucket of concrete or to set a large stone down in a concrete bed; but whenever the derrick is engaged in setting face work (as stone or concrete blocks) such operation requires more constant use of the derrick with consequent fewer intervals when it may attend to the train without delay to one or the other. In general, if for any reason a derrick devotes any considerable percentage of its time to other work than building masonry, it means a loss of output for the derrick, and if total yardage is attained it means a larger number of derricks.

Many of the foregoing disadvantages may be overcome by mere abundance of plant or by the use of more powerful machinery; but at the expense usually of high plant account and uneconomical use of power. It is not to be denied that there may exist situations where delivery of materials by cars on a system of tracks is the proper method rather than by cableway. The conditions indicating the advisability of such installation would be a valley too wide to be practically spanned by a cableway; a dam not so high but that derricks on it can easily reach the tracks, and consequently so low that its corresponding width would not permit enough cableways working over it to attain the desired progress; a comparatively small range of elevation through which the tracks must be built and the materials handled.

We have seen that 10–15 ton cableways 1200 ft. to 1500 ft. long have handled material for 9000 cu. yd. of masonry per month. Under the chapter on Probable Future Methods we shall inquire what if any feasible variations from existing cableway installations may be made in order to attain a larger monthly yardage either gross or per cableway.

Delivery by Chutes and Belt Conveyors.—This system is increasingly common in ordinary building operations. The concrete is elevated at some centrally-located tower, dumped into a hopper and then distributed by means of chutes, the chutes being suspended from guys or from an arm or boom revolving in a horizontal plane about the tower as a center. The advantages of the method are apparent to the most casual consideration. The cost of central tower and chutes and the greater height to which the concrete must be elevated are vastly more than repaid by the facility of lateral transportation, above and removed from other operations, requiring no runways or gang with wheelbarrows and readily reaching the tops of column or wall forms. Delivery by chutes has recently been applied with entire success to the construction of a large masonry dam.

The Lake Spaulding dam being built by the Pacific Gas & Electric Company on Bear River, California, is to be 900 ft. long on top with a maximum height of 305 ft. and is to contain 304,000 cu. yd. Fig. 33 and Plate XII illustrate the situation perfectly.

The main chute from the mixer is 24 in. wide, of wood, lined on the bottom with cast-iron plates and on the sides with sheet steel. It

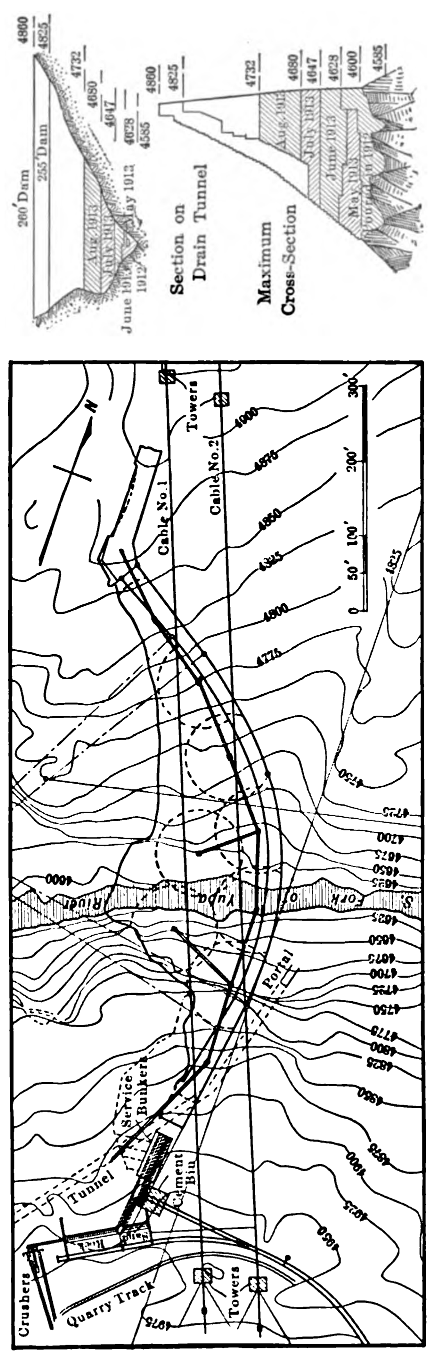


FIG. 33.—Lay-out of construction plant, Lake Spaulding dam.

has an inclination of 4½ in. on 1 ft. On the dam auxiliary chutes distributed the concrete as desired. In the lower portion of the dam the elevation was such that the chutes could reach every part of the work unassisted. Later as the masonry climbed up, reaching the minimum inclination at which the chutes could operate, it became necessary to boost the concrete in transit. This was done not by vertical elevators but by belt conveyors which accomplished both distance and elevation. Plate XII, Figs. B and C, show this operation, while Plate XII, Fig. D, shows a near view of one of them looking in the direction of the travel. Although the inclination of these belts has not been stated, the views seem to show it to be nearly as much as that of the chutes. By the belt system concrete has been delivered at the rate of 2000 cu. yd. per ten hours on a 24-in. belt traveling 400 ft. per minute.

The tendency of the concrete to adhere to and accumulate upon the belt instead of dumping clean is very ingeniously overcome by means of jets of compressed air. Just underneath the belt where it turns back over the last roller, after dumping, are several fan-shaped nozzles which deliver a continuous sheet of compressed air against the face of the belt. This device is entirely successful in removing practically all of the adhering concrete and maintaining the belt in good working condition. (Four 1-yd. mixers were employed.) That this system not only permits of rapid progress but is also very economical, is shown by the fact that in thirty-one ten-hour days (August, 1913) 40,485 cu. yd. of concrete were mixed and poured in the dam; and that this was accomplished with an average force of 313 men including not only those directly handling the materials, but all others as carpenters, engineers, derrick men, etc.

This system, as will be further shown in the chapter on Probable Future Methods, should be entirely applicable to almost any situation, even with much less advantage of natural elevation in favor of the inclines. In connection with such a system cableways are practically indispensable as a means of handling and shifting the chutes and conveyors together with their supports, also the face forms. When not engaged on such work the cableways could be utilized in putting large stone into the masonry.

On the Big Creek hydro-electric development in southern California concrete was delivered by chutes for the construction of several small dams. Dam No. 1, the highest, has a crest length of 800 ft., a maximum height of 135 ft. and contains 58,700 cu. yd. of concrete which was placed in sixty days by twelve mixers. A construction trestle (Plate XIII, Fig. C) the full height and length of the dam was built in twenty-six days just outside the upstream face, and coinciding with the line of the face so that the trestle timbers also served to support the forms. From cars running on the top of the trestle concrete materials were delivered to the twelve mixers installed within the trestle below; from the mixers wooden chutes conveyed the concrete to the dam. (Plate XIII, Fig. D.) The chutes were given a minimum slope of 16 deg. and alongside of each was a runway. Nine stiff-leg derricks with 125-ft. and 140-ft. booms handled the 81,000 cu. yd. of rock excavation for the foundation, and later put plums into the concrete to the extent of 15 per cent. of the total volume. Each 8-ft. lift of concrete was given twelve hours to set before again proceeding with the work. Hence the twelve mixers were operated in two batteries of six each. Two eleven-hour shifts per day were worked, and no cableways were employed.

This system seems more cumbersome and costly than the one employed at Lake Spaulding, and the performance though creditable was not nearly as good. The twelve mixers mounted on a trestle built to the full height and length of the dam was a costly plant for a 58,700-cu. yd. dam when compared with the Lake Spaulding system of chutes, conveyors and four mixers for a 304,000-cu. yd. dam. The four mixers at Lake Spaulding were capable of a much greater yardage output than the system at Big Creek permitted the twelve mixers to accomplish there.

Pneumatic Mixing and Conveying.—Another system of delivering concrete deserves mention, not because it has been applied to any large mass work such as a dam, but because it seems to possess possibilities for development in that direction. Briefly it is the mixing of concrete and forcing it through pipes by means of compressed air at one operation. Fig. 34 shows the hopper with plunger actuated top door and an 8-in. diameter delivery pipe leading from the bottom. A $\frac{3}{4}$-in. air inlet is near the top of the hopper and a 2-in. air inlet enters the elbow in the delivery pipe, pointed in the direction of the flow. The materials are dumped in any order, into the hopper from measuring hoppers, the door is closed and the air is applied. With $\frac{1}{4}$-cu. yd. and $\frac{1}{2}$-cu. yd. batches (the present development of the method) the forcing of the materials from the hopper into the pipe and through several hundred feet of pipe results in the discharge of a satisfactory mixture. To date, this method has been used in tunnel work up to something over 1000 ft. in length of delivery pipe, and in ordinary building work

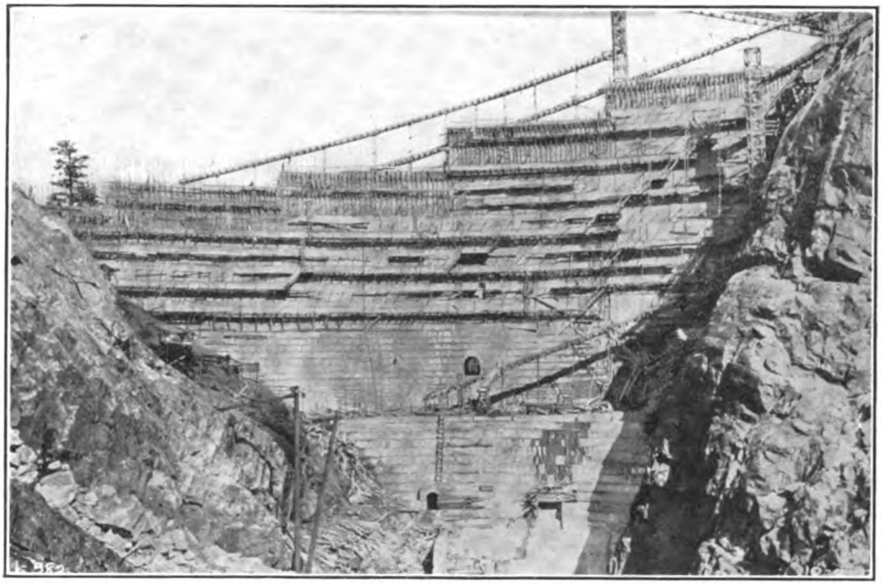


FIG. A.—Lake Spaulding dam, showing delivery of concrete by chutes.

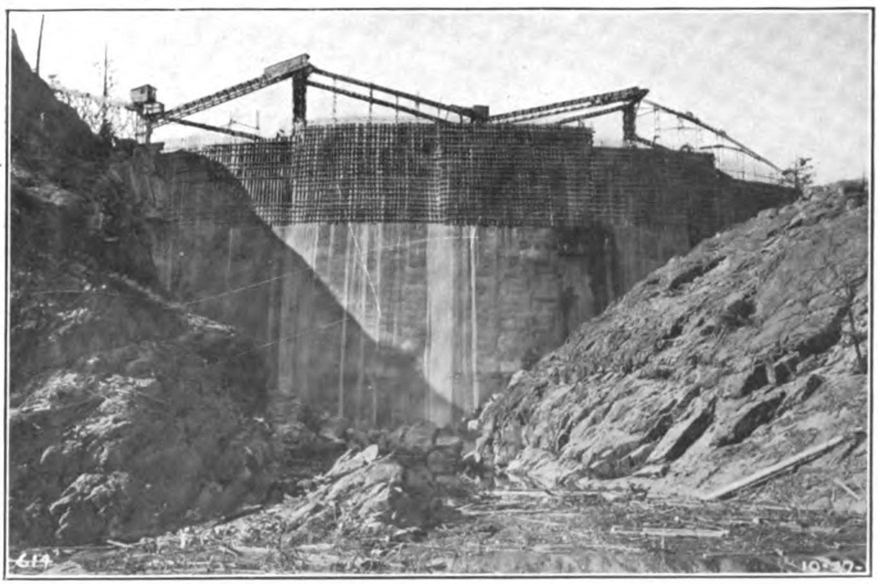


FIG. B.—Lake Spaulding dam, showing delivery of concrete by chutes and belt conveyors.

(Facing Page 170.)

FIG. C.—Lake Spaulding dam.

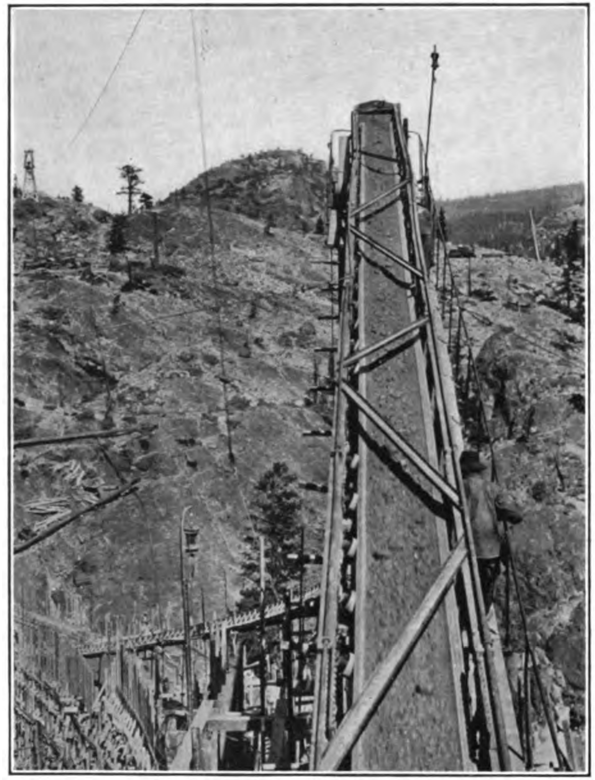


FIG. D.—Belt conveyor for concrete, at Lake Spaulding dam.

it has pushed concrete up 80 ft. vertically. Four ½-cu. yd. batches per minute have been delivered for limited periods of time, and 300 cu. yd. per ten hours is a common and easily maintained rate of progress. Two or more hoppers may be connected to one delivery pipe, resulting in practically a continuous issuing stream. The compressor capacity advised for a ½-cu. yd. machine operating at 30 cu. yd. per hour, with moderate distances, and few elbows in delivery pipe, is 500 cu. ft. of free air per minute raised to 80 lb. pressure. For maintaining the same rate at distances over 500 ft. add about 1

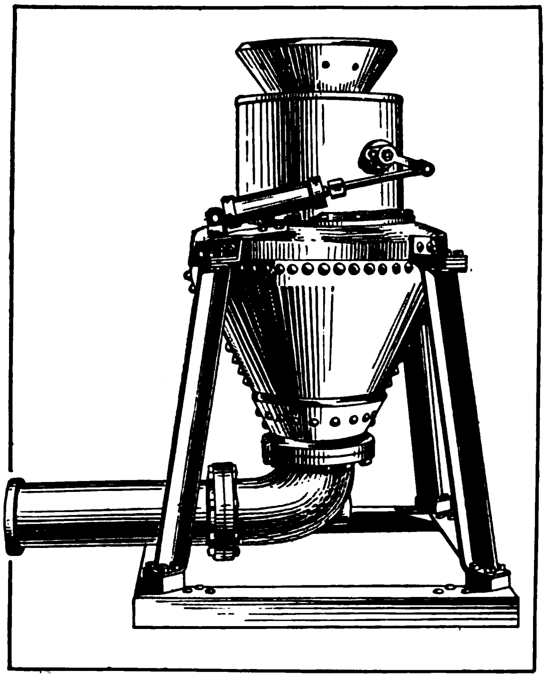


FIG. 34.—Hopper for pneumatic mixing and delivery of concrete.

cu. ft. of air for each foot of distance; also add about 2½ ft. of air for each foot of elevation to be overcome. Moderate receiver capacity should be provided, and the pressure will fall during the delivery of a batch from 80 lb. to 50 lb. or 60 lb. The 8-in. delivery pipe may be of thin steel weighing about 250 lb. per 20-ft. length, and the joints may include special gaskets permitting considerable flexibility. The machines are installed upon a royalty basis which naturally varies with the situation and the saving which may be made over any other system of delivery. In other words the charge is "what the traffic will bear."

Naturally the greatest advantage of the system lies in its application to a situation difficult of access by any other system, such as in a tunnel. As it accomplishes mixing, elevating (within limits) and horizontal transportation in one simple operation, and as the delivery pipe can easily cover a wide range of work that would require many elevators and chute equipments, this system should find a wide field for application. However, its advisability in connection with a large masonry dam would be a matter for some study. Thus, compared with the system of chutes and conveyors at Lake Spaulding the delivery pipe would be very much cheaper to construct and easier to shift and raise. It would require, however, say three times the power and much greater compressor capacity. The same rate of progress as at Lake Spaulding would require for pneumatic delivery some 2500 cu. ft. of free air per minute, which would mean 350 h.p. or more according to elevation above sea level.

Various devices are used for transporting some of the materials which enter the masonry. Attention will be briefly called to them and their possibilities.

At the Roosevelt dam, cement and sand were brought to the dam by a Leschen tramway. (See page 139.) The buckets averaged 8 cu. ft. of sand and about 5.2 cu. ft. of cement. The labor cost per cu. yd. transported was about $0.175 when supplying cement and sand for the maximum masonry progress, and about $0.243 when at two-thirds maximum masonry progress. Obviously the distance has a very slight effect upon the cost of transportation by such a method. It could have been ten times as far at practically the same cost.

At the Big Creek hydro-electric development (see page 169) pit sand and gravel traveled 154 ft. on a belt conveyor up a 35-deg. slope. At the Arrowrock dam, cement is being transported 1500 ft. by blowing it with compressed air through a 4-in. diameter pipe. The cement drops into the pipe through the riser of a 4-in. Tee into one end of which the air is introduced through a ¾-in. pipe. Several ¾-in. air nozzles are inserted along the line as boosters. The system is said to be entirely satisfactory.

Although in the present work the subject of excavation for the foundation of a dam is only incidentally mentioned, it should not be lost sight of in figuring upon the plant required for the masonry.

The amount of excavation may be enough so that a saving of a few cents per cu. yd. may materially reduce the plant account chargeable to masonry if the same plant is adapted to both operations. In general, it may be said that in a situation where a system of tracks is advisable for delivery of the materials for the masonry it will generally be found that the excavation can be disposed of immediately upstream or downstream from the trench.

Where cableways are indicated for handling the masonry it usually means a valley so narrow that the excavation must be disposed of in a dump paralleling the stream. In such case the cableways are probably the best means of conveying the material longitudinally to that station where it must be transferred to cars for lateral travel to or along such dump. It should be remembered that at the transfer station storage should be provided, as by bins or hoppers, into which the cableways may dump, and under which the cars may run. This is in order that one will not have to wait on the other, that coordination of functions may not require the wasteful coordination in point of time which was pointed out in discussing the delivery of masonry materials via train to masonry derricks.

For an account of the economical handling of a large quantity of excavation at the Arrowrock Dam the reader is referred to *Eng. News*, July 17, 1913.

CHAPTER XIII

PROBABLE FUTURE METHODS

The history of masonry dam construction during the past generation offers several illustrations of radical departure from previously accepted standards. These changes, as well as the advance in the size of undertakings, have led to most significant alterations in construction methods. In fact, growth in size, departures in design and the various developments in methods have been simultaneous, interdependent and contributing each to the others; an effect of one may be the cause of another. The general advance will be analyzed and an attempt made to show what changes may be expected in the future.

The earlier dams were built of rubble masonry consisting of some large stone and a certain percentage of one-man stone and spalls. The large stone was bedded in mortar and the vertical joints were filled with small stone and mortar. Derricks were used, but often they were operated by hand or by teams. Advance in the practice resulted in the universal use of power-operated derricks with bull-wheel and swinging-gear attachments (eliminating the old-time tagman) in the self-contained or stiff-leg derrick which was more readily moved, and which from absence of guys interfered less with the cableway delivery of material which had accompanied the evolution of the derrick. More recent advance has been in the direction of increasing the power in order to operate at higher speeds. It is probable that in only a few of the larger and more recent rubble masonry dams, did the magnitude of the work point unmistakably to distribution of power from a central power station.

Increase of Output.—The unanimity in rates of progress on rubble masonry dams is most striking. Thus on Dunning's dam, August, 1887, to November, 1889, 35,700 total cu. yd. "On one occasion one foreman with eight masons and nine helpers, with double drum steam derrick laid nearly 500 cu. yd. in seventy-six hours." This equals about 6.6 cu. yd. per derrick hour, probably higher than average rate. The derrick was probably overmanned or supplied with a large percentage of one-man stone. "Another foreman with seven masons and eight or nine helpers laid 375 cu. yd. in seven days" equals about 5.35 cu. yd. per derrick hour.

On the Sodom dam, February, 1888, to October, 1892, totaling 35,887 cu. yd., said to be the first one upon which a cableway was employed. "The largest quantity laid per month was 3000 cu. yd. with twelve masons and three derricks;" which figures out about 5 cu. yd. per derrick hour. On the Titicus dam, 1890–1895 "Six derricks and thirty-six masons averaged 3240 cu. yd. per month, the maximum being 5700 cu. yd." Assuming twenty-six eight-hour days per month this would be an average of 2.6 cu. yd. and a maximum of 4.6 cu. yd. per derrick hour. Though not under comparable conditions, it may be interesting to observe that on the Tausa dam, "The maximum progress was during January, 1891, when 700 masons laid 2600 cu. yd." Assuming twenty-six working days, this performance equals 1.43 cu. yd., per ten-hour mason. At the New Croton dam the average was about 5 cu. yd. per derrick hour. On the Wachusett dam, while occasionally 6 cu. yd., the average was not over 5.5 cu. yd. per derrick hour.

The foregoing figures may be taken to indicate that 5 to 5.5 cu. yd. per derrick hour is the normal rate of this class of work; that total progress on a work of some magnitude would be this rate multiplied by the number of derricks which mere available area would permit being installed; and that improvements and elaborations of plant were to reduce one of the elements of cost (transportation) and to supply a larger number of derricks.

The Roosevelt dam, as previously explained, was built under a different specification, the large stone being bedded in mortar and the vertical joints being filled with concrete and spalls. Progress under this specification was about 16.5 cu. yd. per derrick hour, accompanied by a corresponding reduction in cost, although the situation particularly favored a low cost for some of the elements of the process. It seems probable that this is about the limit of output that may be expected of any method which utilizes such a percentage (50 per cent.) of stone. Although the method realized two-thirds of the advance in output possible under the change to cyclopean masonry, it effected no saving in cement (using 0.76 barrel per cu. yd.) over the requirement for that practice, the mortar beds requiring just about the amount saved by displacing concrete with stone. Altogether the method, while interesting, seems to offer no advantages which would tend to perpetuate it. It is believed that this was the first large dam where electric power was distributed from a central station and used with anything like the universality, satisfaction and economy which seems to characterize it as the method of the future.

Strictly cyclopean concrete was first used on the Boonton dam, and since then upon the Cross River, Olive Bridge, Kensico, Salmon River Idaho, Barker, Delta and others. The development of this method seems to have been in the direction of using a smaller percentage of stone and attaining a higher rate of progress. Thus the Boonton dam is stated to consist of about 50 per cent. stone, the Delta dam 43 per cent., Salmon River and Southern Power Company 40 per cent., Cross River 35 per cent., Olive Bridge and Kensico about 26 per cent. The rate of construction in cu. yd. per derrick hour for the earlier of these dams, if indeed ever noted, has not been stated; in the last two above mentioned an average rate of about 21 cu. yd. per derrick hour was attained.

Such differences in amount of stone and rate of building seem to be a perfectly natural development and are too great to be accounted for by differences in size and shape of stone available. Indeed the increase in rate was made possible by the decrease in stone, and the increase in rate should effect a much greater saving in cost than would offset the cost of extra cement due to decrease of stone.

The construction of a large masonry dam was until recently an art or problem, the solution of which centered upon the dam itself. Recently, however, it has become more and more a transportation problem; simply a question of how best to get the materials from one place to another. The materials and the specifications have changed so that now the quality of the masonry very largely takes care of itself. The masonry, barring some face work, is now dumped by laborers instead of being built by masons; coincident with this change is the much greater rate of progress, which may be considered partly as cause and partly as effect.

Development of Existing Appliances.—When we contemplate the development of cyclopean concrete, the tendency (too pronounced and persistent to be accidental) toward a smaller percentage of stone, and the saving in cost of both plant and labor which would result from frankly discarding stone altogether, the conclusion is irresistible that the important dams of the future will be of concrete. It will be interesting to inquire how far present appliances and methods will be modified by this change.

The entire matter of putting in or leaving out stone is a matter of economy, of balancing value of cement saved against cost of production of large stone in a particular quarry. In other words, stone for crusher material can be produced, crushed and ground to concrete aggregate and the concrete can be mixed and placed in the dam for a certain price per cu. yd. The question is, will the cost of a certain percentage of the concrete be more or less than the cost of producing the large stone (at a modification of entire quarry methods) transporting and placing it; bearing in mind the additional plant required to handle the stone?

The quarry plant will be simplified and reduced in amount, quarry labor will be reduced, transportation from quarry to crusher and mixer, and thence to the dam will be developed to permit a much higher rate of progress, derricks on the dam will be eliminated except possibly for setting face work, and labor on the dam will be reduced very materially. In general, plant and methods will be devised strictly with the end in view of handling concrete. If any stone is introduced it will be because it can be produced in the particular quarry at no additional cost and can be readily put into the dam with a small additional amount of plant.

The incongruity in present practice is the immense number of derricks and attendants employed on the dam for no purpose at all except to move buckets of concrete and a small amount of stone a few feet from the place where left by the cableway, and to dump them or set them down. They perform no function that could not be as well performed by cableways except for the mere matter of a few feet in location, and that detail can be arranged. Masonry construction has been reduced simply to leaving the materials in a certain spot, and the derrick is simply a relic not yet discarded.

A very common cableway capacity is 10 tons, with a hoisting speed of 300 ft. per minute and a conveying speed of 1800 ft. per minute. Such speeds are high enough for every practical purpose; the time necessary for a trip depends principally or largely upon the time required for hooking and unhooking the loads. The possible gain in actual yardage handled that might be attained by increasing the speed is so small that it seems very doubtful if it will ever be found practicable to attempt it. There is a probability, however, that considerable gain may be made by increasing the average weight of load.

Thus for illustration take the previously mentioned eight-hour record for the cableways at the Olive Bridge dam. The loads of concrete were not more than 5½ tons including the containing skip.

It would seem not impossible to design the plant and the operating methods so as to be able to convey nearly or quite double the net load with slight if any decrease in number of trips per day.

In the case of a dam of such length as to at first sight seemingly preclude the use of cableways, and still of such a height, and in a valley whose side slopes are such, that the problem will not be satisfactorily solved by a system of tracks, the difficulty may be overcome by the installation of cableways with an intermediate tower, and by the sending out of materials from a terminal at each end. Of course, this would involve two points for assembling the materials, but such a dam would contain yardage enough so that the saving by use of cableways might easily be much more than enough to pay for the two terminals as compared with one terminal and any other system of delivery. The intermediate tower could be of steel and be left in the masonry. This tower would more than double the capacity of the cableway, as, in addition to working from two ends instead of one, the average length of trip would be reduced. The most striking example of an intermediate tower is in connection with the Tunkhannock viaduct now under construction on Delaware, Lackawanna and Western Railroad, at Nicholson, Pa. An intermediate tower 230 ft. high (later raised to 290 ft.) containing 101 M. ft. of timber supports two $2\frac{1}{4}$-in tandem cableways which are spaced 25 ft. apart. The span to one end tower is 1510 ft. and to the other one 1525 ft. It is not necessary to discuss here variations in operating procedure which may be followed in the case of the construction of a viaduct rather than a dam.

It has been previously pointed out that soon after the dam is started, from the time when the whole area of what might be called the bottom of the dam is under way, the available working area may not be materially reduced till some elevation near the top of the dam is reached. The area changes shape, becoming narrower and longer. Suppose an original installation of parallel cableways sufficient in number to cover the area fully. Then as the area grows narrow some of the cableways are put out of commission as they are no longer over the working area. To maintain the output more work must therefore come upon the remaining cableways.

A variation from the usual plan may be made, not only that more cableways may be used at any and all stages of the work, but also that if there is at some stages some inequality in performance still, they will be occupied till the dam is completed. The assumptions are the following: a dam 1500 ft. long; for 700 ft. in the center the masonry has a bottom width of 200 ft.; cableway towers may be placed 100 ft. from each end of the dam; span of cableways 1700 ft.; the towers (one at each end) to be such that the cables are spaced 25 ft. apart. The towers would be unique but not impossible structures.

Of the accompanying cuts, Fig. 35 shows an installation of eight cableways after the usual manner and how presently some of them become practically useless. Observe that for each 25-ft. decrease

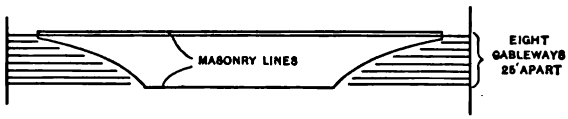


FIG. 35.—Usual scheme of cableway installation.

in thickness of masonry one cableway goes out of commission. However, if derricks are employed on the dam, the cableway may still be available through the device of building landing stages on the downstream side of the masonry; but there are several objections to such a method, and at best it would only render available about one cableway in addition to the number operating over the masonry.

Fig. 36 shows a suggested layout to obtain the above-mentioned advantages. It gives 50 per cent. greater cableway capacity at

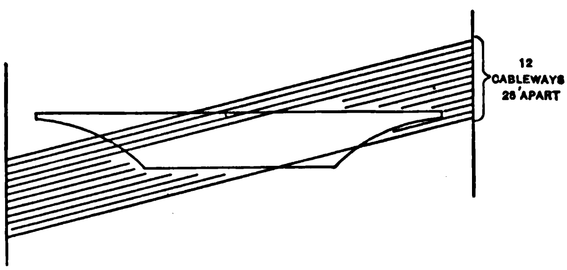


FIG. 36.—Possible installation scheme permitting more cableways.

the start, and maintains that original capacity up to near the top of the dam. In practical application the performances of the cableways can be kept more nearly equal than might be indicated from a casual inspection of the figure; the cableway spans are increased from 1700 ft. to about 1750 ft.

Using previous records of actual performance let us see what such an installation might be expected to accomplish. At Olive Bridge dam four cableways delivered 35,300 cu. yd. in a month.

Then let us assume that it is only a question of getting the material to the cableways, and that twelve cableways could put 106,000 cu. yd. per month onto the dam. Let us first see what this would involve if the present practice was followed of installing derricks on the dam to rehandle all the materials.

At the Medina dam, derricks set 27.2 cu. yd. per hour of masonry containing 10 per cent. of large stone. Assuming that rate per derrick (disregarding for the moment the question of how far it may be economical or necessary to sacrifice stone to gain yardage in output), about nineteen derricks would be required to set the masonry. Somewhat more than nineteen derricks could easily be placed on such a dam so as to work without interference. That the derricks could not maintain this rate in the upper 50 ft. of dam does not interfere with the general propositions that as many derricks should be employed as can be placed without interference, that they should do nothing but set masonry, that they should be supplied with materials up to their capacity, that it would undoubtedly be found economical to sacrifice much or all of the stone in the masonry if thereby the output per derrick hour could be raised from 21 cu. yd. or 22 cu. yd. to 27 cu. yd. and that it should be possible to operate such a plant so that 80 per cent. or more of the mass could be built at an average rate close to the above assumed maximum. It may be objected that the foregoing rate per derrick quoted for Medina was for masonry laid between forms, but it must be remembered that half or more of the material was delivered on cars along the toe of the dam. This method involves time of the derrick to reach its materials, and the moving of forms probably involves some delay as well.

If the materials were deposited directly by cableways, four-fifths of the derricks on the dam could be dispensed with and probably one-half of the labor. The working area would be unobstructed, conducing to rapid progress. Until the dam reached an elevation where it was comparatively thin, four derricks could set all of the face blocks. For that purpose the derricks could be smaller, lighter and more readily shifted from place to place.

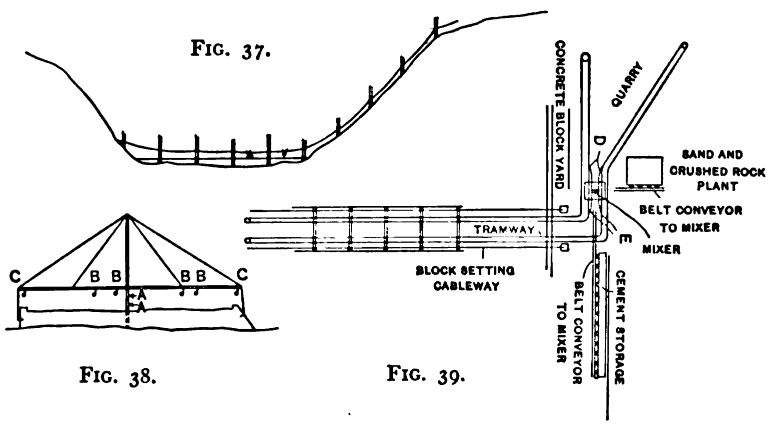
The cableways could deliver in place, as masonry, as much material as they now deliver to derricks. Without being unhooked from the cableway, a bucket of concrete could be dumped directly in less time than is required at present in unhooking from the load and hooking onto an empty. If the quarry was such that a certain percentage of large stone could be produced at practically no extra cost or with no modification of quarrying methods, such stone could be placed as readily by the cableways as by the derricks. However, it would not be economical to do so unless the stone were large enough, say 3 cu. yd. and upward, to warrant a trip of the cableway, as it would not be practicable to handle skips containing several stones. In case it is found that (along lines 25 ft. apart) is not sufficient distribution of the materials, it would be simple and inexpensive so to mount the cableways that they could traverse 12½ ft., and this would provide for amply covering the area. The additional cement required in case the stone is eliminated would be, say substituting 1-3-6 concrete for 25 per cent. of stone, somewhat less than 0.3 barrel of cement per cu. yd. of masonry at a cost which should often be much less than the savings in other directions resulting from the change.

While the cableway is a most admirable machine, suitable for a large variety of situations and operations, and while it will undoubtedly continue to be a very desirable item of plant for the construction of large masonry dams, it has one inherent feature which may tend to relegate it to the position of an accessory rather than of a principal machine. Briefly its limitation is that but one load can occupy it at a time. The cableway takes out a load, leaves it, takes an empty and returns, the round trip requiring three to five minutes. This involves motion in two directions, as well as raising and lowering at each end, hooking and unhooking, transmission of signals for most of the operations, and altogether an amount of lost motion, lost time, and breaks in continuity that are distressing to one who has analyzed the elements of the problem of transportation of materials.

Possible New Appliances.—Nearly five years ago the writer proposed the following plant and method for constructing a large masonry dam:

"Assuming the body of the dam to be of concrete with large stone embedded in it, and the faces of the dam to be of concrete blocks. For handling the material for the body of the dam, equip with two Leschen or similar tramways; each to consist of one standing cable extending across the dam and back, and one traveling line moving continuously and in one direction; the two to be supported by towers comparatively close together.

"The advantage of this tramway is that with a given speed of traveling line the capacity is limited only by the number of loads that can be attached per minute, whereas an ordinary cableway must make the entire length of the trip and return, coming to a stop twice for each load conveyed. The tramway is to be supported by towers say 150 ft. apart to permit good-sized loads to be carried on a moderate-sized standing line without excessive sag. (See Fig. 37 herewith.) Across the masonry at least, these towers should be something as shown in Fig. 38, made of regular rolled steel shapes. The masts are to be built into the dam and left there, although they would have to be shifted in position several times as the dam grew higher. Between the points marked "A-A" (Fig. 38) would be introduced sections in 10-ft. or 15-ft. lengths as the work raised, the upper part of the tower with the boom being jacked up while the



FIGS. 37–39.—Possible scheme for conveying concrete to a dam.

section was being inserted. The supports "B-B-B-B" (Fig. 38) for the tramway would be capable of being readily moved along the boom in order that the whole width of the work may be covered. In connection with the lateral movement of the tramways, the loads could be automatically dumped at any point in the length of the dam, thus covering the whole work.

"Referring to Fig. 39, it will be appreciated that this system is quite flexible. The tramway can be carried as far up the hill, also as far to one side as desired within reason, without cutting down the capacity, so that it permits considerable departure from the assumed relative positions of the dam, quarry, sand plant, mixer and cement storage. The empty buckets on reaching the points marked "D" would be automatically released from the traveling line and switched onto the side track under the mixer. After being filled they would be started down an incline of sufficient length and inclination to acquire the speed of the traveling line; at "E" they would go onto the standing line again and automatically grip the traveling line.

"At similar loading stations on the quarry the tramway could take up chunks of stone, using the same kind of bucket for small chunks as for the concrete. For large chunks eliminate the bucket and use chain and hooks, which can also be arranged so as to release automatically. One or two steam shovels in the quarry should deliver rock to conveyors running to crushers in the rock and sand plant. The finest portion of the output of the crushers will go to rolls for reduction to sand. The screen between the crushers and the rolls could probably be arranged so that the amount going to the rolls would furnish just sufficient sand to accompany the crushed rock; if so, the stone and sand could probably go to one bin, and from there go via one conveyor to the mixer. At the mixer should be devices for very rapidly measuring the cement and the product of the sand and rock plant.

"Two Hains (or similar) gravity mixers, one for each tramway, should be provided. The bottom hopper should have a gate to regulate the amount and time of delivery to the buckets.

"There should be ample cement storage, for cement either in sacks or in bulk as the case may be, with a belt conveyor to the mixer. Near the mixer should be the concrete block factory with tracks to the storage yard. The yard could be say 50 ft. wide and as long as necessary. It should be spanned by one or two traveling cranes of one-block capacity to handle the blocks coming from the mixer and the blocks going to the dam. A large part of the blocks should be made and in the storage yard before work on the actual masonry construction commences; the height to which they can be piled in the yard would be limited by the height of the traveling cranes. From the yard the blocks would be taken by platform cars or some form of conveyor to two cableways, one over each face of the dam. These cableways are shown at 'C–C,' Fig. 38. The carriage and hook travels out and back; out with a concrete block, stopping at the proper point, the block lowered into place and then back for another load. About two block setters will work on each face. Between trips they will prepare a mortar bed for the next block and fill up the vertical joint just made. Occasionally the cableway will make a trip with a small skip of mortar and return with an empty skip. As the blocks would be of uniform and accurate dimensions, they could be set rapidly, especially as the cableways (like the tramways) could be shifted laterally so as to hang over the center of the course it was laying. There would be no men on the dam except the face block setters, and a man or two to keep the work wet down. Slung between the towers at the level of the boom should be a light foot bridge. From this bridge one or two directors will watch the work and direct it by electric signals, calling for concrete or stone as may be desired, for any desired lateral shifting of the tramways and for any setting of the automatic dump.

"Assume on the tramways a speed of 200 ft. per minute for the traveling line. The loads could be spaced less than one minute apart, possibly forty to fifty seconds. Assume 1-cu.yd. batches of concrete and the same for an average load of stone. Then the two tramways should put onto the dam 1000 cu. yd. to 1200 cu. yd. per eight-hour shift. This rate could be maintained nearly to the top of the dam; for the top probably but one tramway could be used.

"Starting masonry with two or three towers in the bottom, others could be added in the desired positions as the work progressed. The booms would also be shortened. As the dam got higher, and the ratio of face blocks to filling larger, two shifts of block setting might be required to keep up with one shift of filling. The feasibility of many of the suggested devices has been satisfactorily settled in consultation with a prominent tramway manufacturer."

A possibly desirable modification of the above would be to space the towers closer and to use a heavier standing line,'or even a rigid track, for the purpose of handling heavier loads and to withstand the vibration incident to dumping the loads. Although this system could handle stone, it is not to be denied that it would be somewhat simpler and easier to construct and operate it to handle concrete alone. The cableways proposed for setting the face blocks would be necessary for handling forms in case it was desired to build the faces that way.

One other radical departure in method for building dams is inevitably and most forcibly suggested by a consideration of the expansion joint and its logical consequences. If vertical breaks in the longitudinal continuity of a dam are allowable, to say nothing of their desirability, why not take full advantage of them in construction methods? In other words, if the method offers any advantages, start at the top at one end and build across in a succession of vertical sections, finishing each to the top as advance is made. This would permit sending out the concrete in dump cars of large capacity and running on a level on top of the completed sections with the simplest possible installation of tracks.

Much more concrete per trip of containing vessel than by any other present or proposed method would then be feasible. The cars would be moved by an endless traveling cable, would dump without stopping and would be detached from the cable only while being filled at the mixer. As in the previous scheme the gripping and release of the cable would be automatic, and on being filled the cars would go down enough of an incline to acquire the speed of the traveling line.

The extension of the system as the work progressed would be most simple, involving only the shoving ahead of the dumping device and hopper, the introduction of a section of track and the shifting of a tension or take-up sheave at the land end. From the hopper into which the concrete has been dumped it will be delivered wherever desired in the section or sections ahead via one or more pipes or chutes swung from light guys. As concrete will run readily on an angle of about 25 deg. from the horizontal, the entire work can be readily reached for several sections in advance. Indeed, delivery of concrete and several other details of the work will probably be facilitated by working on several sections at a time. Such procedure of alternate delivery to several sections would be highly desirable in connection with the shifting of forms or the setting of face blocks. As in the previous scheme, a couple of cableways would be necessary to handle the face work, whether blocks or forms, and also the forms for the expansion joints. The same cableways or another one could put in whatever stone it might be found economical to introduce.

This method might be criticised in two respects. The first objection is that under the procedure of approaching the deepest and most critical part of the foundation by building down hill toward it, a proper regard for security as to character of foundation as well as the method of building would require that more of the pit would be open at one time, that it would be open for a longer time, and that consequently there would be more pumping and greater liability to interruption from high water. These objections, however, are more apparent than real, and in most cases would be more than balanced by accompanying advantages. Thus it would permit the work to be prosecuted by a larger force during the earlier stages, *i.e.*, during the time usually devoted to constructing the temporary and diversion works and to excavating the pit. If, as suggested, masonry construction was progressing on several sections in steps, high water might rise over the advanced lower step without causing any suspension of the masonry work for some time, or until the remaining steps had advanced to a plumb face at the water's edge, which situation would conduce to still more rapid progress when the pit was again available. If more masonry must be built while the pit is open, it may also be said that more masonry could be built by this method in the same time.

The second objection is that storage of water behind the dam could not begin until a much larger percentage of the total mass of masonry had been placed. While this is absolutely true, the resulting damages and benefits should be balanced as follows in arriving at a decision as to method: The value of stored water should be applied to the time between the date when storage could commence under any other method of construction and the date when it could commence under this method, which difference might not be as great as the relative masses of necessary masonry might indicate. It should be also borne in mind that after storage had once begun the water level could be permitted to rise very much faster under the proposed method of construction than under any method hitherto proposed.

To sum up: Cyclopean concrete has been accepted. The progress already attained with it is four times as fast as rubble masonry. Concrete will be substituted for Cyclopean concrete when it is fully appreciated that the result will be still more rapid progress, further reduction in cost and the elimination of much cumbersome and unnecessary plant.

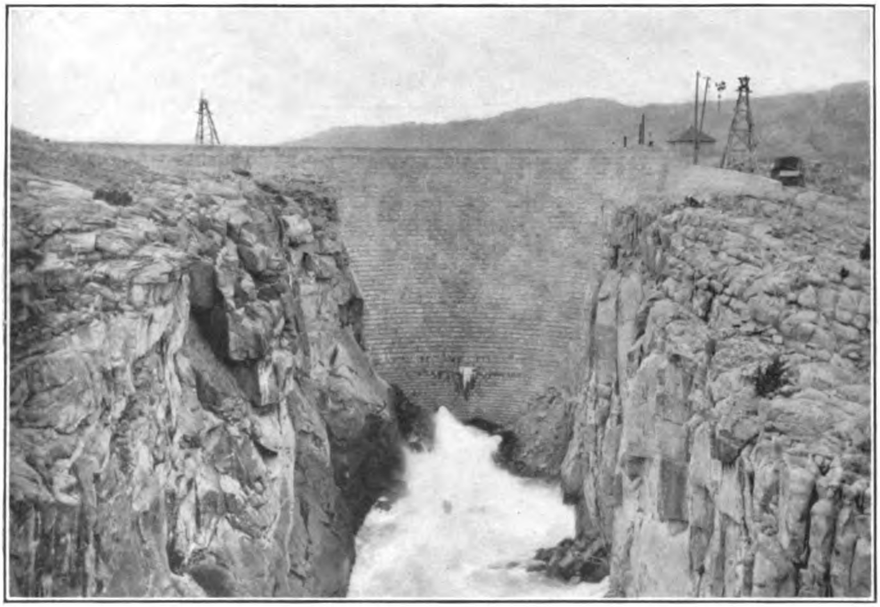


FIG. A.—Pathfinder dam.

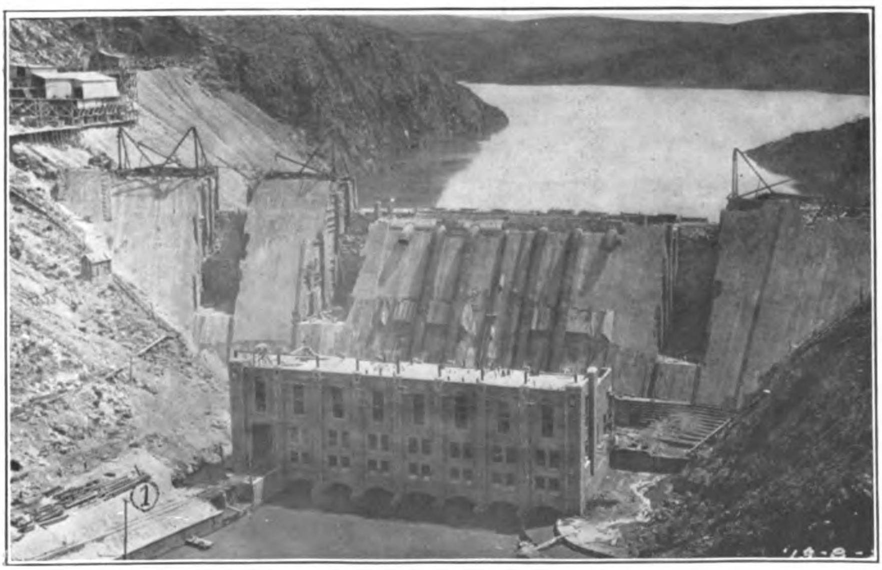


FIG. B.—La Boquilla dam.

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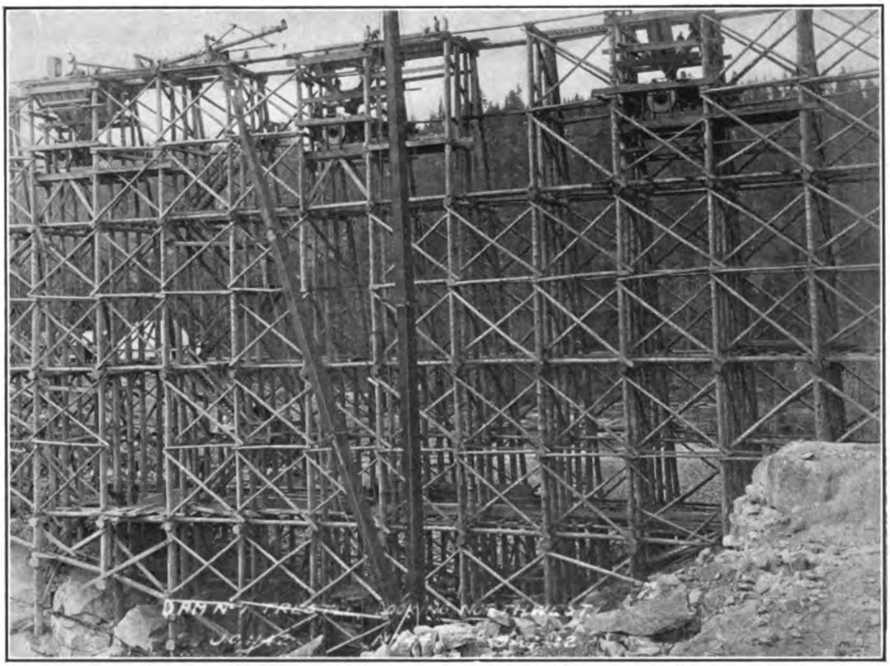

FIG. C.—Big Creek dam, showing trestle and mounting of mixers.

FIG. D.—Big Creek dam, showing delivery of materials to mixers and delivery of concrete from mixers.

CHAPTER XIV

ESTIMATES OF COST

Conception and Development of a Project.—A project passes through several distinct stages between the time of inception and the time of completion, and its progress involves several or possibly numerous estimates of cost. The various estimates, each with its function and value, may be based upon more and more accurate data gathered during the progress of the investigation, may result from the employment of different or more specialized engineering talent or may be due to different interests active in any of the various functions of promotion, financing or construction.

Thus, to illustrate briefly the nature of the estimates accompanying the various phases of a project. Assume that a promising site for a dam is known to or discovered by some person who forms a conception of a project for developing hydro-electric power, or for irrigation, or both. Then there immediately arises the question of cost of an adequate dam. Such an estimate, which we will call Estimate No. 1, is most informal and is based upon very little information that would be considered data from an engineering point of view. In fact it may be no more than a rough comparison with the reported cost of some similar project, ignoring many important elements of difference, and usually most optimistic in character.

The project is next taken up by some person or persons of the locality who have some standing and substance; not professional promoters but people who have some appreciation of the professional promoter's function, and who finally find it desirable to associate a professional with them in the enterprise. The head of the promotion syndicate thus formed has some though often an inadequate appreciation of the necessity for and value of the services of an engineer. Capital must be approached with a logical statement, and an estimate of cost that bears some stamp of engineering accuracy.

The engineer of the syndicate is usually a man in general practice in the locality, honest and of some or even considerable ability. However, his estimate, which we will call Estimate No. 2, is almost invariably subject to considerable revision for one or more of the following reasons:

First.—The amount of money available may not have provided for thorough or exhaustive surveys, borings, test pits, etc., nor for well-considered plans and accurate determination of quantities.

Second.—Unless the engineer has had rather more than average experience, the scope of the entire project as well as the size, location and character of the various features may be in error. Even if quantities are approximately correct the estimate of cost may be little more than a guess, with no sign of appreciation of the various elements which enter into it.

Third.—His interest in the project and association with the promoters may often cause him to be unduly optimistic himself, and may unconsciously affect his opinions as to quality of the materials to be encountered or the cost of various operations.

However, Estimate No. 2 is usually based upon a survey or at least a profile of the site, some information as to depth to rock, and the character of the rock. It also contains some appreciation of the manner and cost of procuring the principal materials, of the source of power, the cost of labor, amount of water to handle, etc.; in fact, the information upon which an experienced man can base a very fair estimate.

With Estimate No. 2 in hand the promoter seeks to interest capital. It is unnecessary here to trace his adventures. If the project appears to be a good one it finally secures consideration at the hands of some individual or organization in position to supply the money necessary to finance it. A very critical stage for the project is its examination at the hands of the financier's engineer; and its progress may be outlined about as follows: The prospectus and Estimate No. 2 are handed to an engineer for an office examination, a scrutiny which in a few days or possibly a few hours develops whether or not it is worth while to go into the matter further. The general scheme is first examined to see if it has been outlined in the best way, and with a proper conception of the size, location, design and function of the various main features. The data collected by the promoters' engineer, such as surveys, plans, borings, etc., must (and usually may) be taken at their face value. Plans and specifications of proposed structures are generally rather sketchy, but they should show that structures adequate in design and of suitable materials may be accomplished at about certain mentioned total quantities. Quantities are roughly checked and proper unit prices applied, thus arriving at Estimate No. 3. Before being financed the project may of course be submitted to the consideration of several parties. In each case some process such as outlined is or may be gone through, resulting in an equal number of estimates on the same information and basis, and corresponding to No. 3.

Estimate No. 3 is accompanied by the engineer's analysis of the prospectus. He corrects and supplements its usually meager and inexact information, investigates more or less thoroughly the most important collateral questions, such as rain-fall, run-off, land and crop values, markets for produce or power, transportation facilities, tributary population and industries, labor supply and prices, cost of fuel and power, natural resources, climate, etc. The fate of the project is much more often determined at this stage than at any other. It is highly desirable, therefore, that Estimate No. 3 be conducted by an engineer of experience and good judgment; all the more so as there is a distinct tendency to ascribe to the estimate an importance and weight in excess of its pretensions or desert. The reason for such exaggeration is not far to seek. In the circle, narrower than supposed, where such projects are examined and financed, the progress of the project is usually pretty well known; and if submitted to a second or third party it often labors under the handicap of being suspected to contain some "nigger in the wood-pile." Indeed, after one or two rejections, whether warranted or not, it is often difficult for the project to obtain further consideration.

In considering Estimate No. 3 several things should be borne in mind. The following may usually be fairly said regarding the engineer responsible for it:

First.—He is more able and experienced than the author of Estimate No. 2.

Second.—He is unbiased, whereas the other may be affected by some contingent or prospective interest in the success of the project.

Third.—He is acquainted with many projects, hence can assign to the one under consideration a fairly accurate relative value, which is fully as important as an absolute value. Indeed, projects within the engineer's knowledge may often be mutually exclusive. His broader view enables him to say which one should be built, which one should economically be built first, or even that it might be wise to have a project wait on the natural course of developments five or ten years hence.

Fourth.—If he is a member of a large engineering firm or organization, he should have at his command within such organization highly specialized engineering talent to which various particular features of the project may be advantageously referred.

Such an organization, however, has its corresponding disadvantage, which in some cases has been known to be considerable. The work of several specialists may be very imperfectly correlated and assembled. Unless the estimate in its course through the organization is governed continuously by one well-balanced engineer, each engineer or each gradation in authority may boost the prices estimated by each previous department, "just to be on the safe side." The result is that the estimate, when it finally emerges, proves to be such a load that the project is pronounced inadvisable.

Assuming that Estimate No. 3 or one of them, shows the project to be so favorable that the capital is pledged; then, without tracing all the steps in the formation of the company, the negotiations with the promoters, the examination of titles, franchises, charters, etc., there is an interval between decision and construction which is utilized in making Estimate No. 4.

In making Estimate No. 4 all previously considered data are supplemented by additional and more extensive information. Surveys are extended, checked and made to show more detail; borings and test pits are made in sufficient number to assure that no surprise will be encountered during construction. All questions which have a bearing upon the proper design and the cost of the works are thoroughly investigated and final plans and specifications drawn up. Both the quantities involved and the prices should have been subjected to such an investigation that any element of uncertainity is very largely eliminated. If Estimate No. 4 shows a larger total than Estimate No. 3, it should be because the scope of the project has been extended or altered in such a way as to show an additional return commensurate with the additional outlay.

The purpose of Estimate No. 4 is to form the basis for the construction contracts, and to determine the issues of securities. Very many of the best companies now engaged in examining, financing and constructing such projects undertake the construction only upon a cost plus percentage basis. Where such is the arrangement no further estimate than No. 4 is required, as cost accounts, being bills for labor and material, are not considered here as estimates.

However, when the construction contract is awarded to some contractor after receiving bids on a lump sum or unit price basis, the several bidders must each make an estimate, which will be here referred to as No. 5.

The lump sum basis is deservedly falling into desuetude on many kinds of work. Its only reason or excuse is where an experienced and financially strong contractor is able and willing to undertake the function of insurance, of insuring that a certain thing will not cost more than a certain amount. This feature sometimes appeals to certain timid capital, but it is an undesirable mingling of functions which should properly be kept separate. Usually it means that the financiers have not spent enough money on their own investigation (for Estimate No. 4) or, which is the same thing, that they have not been properly advised by an engineer in whom they have confidence. In such a case the contractor's estimate practically corresponds to an estimate No. 4. It must embrace a determinaton of the quantities involved, as well as a fixing of the price at which he is satisfied he can do the work.

The lump sum basis more than doubles the elements of risk and uncertainty which the contractor must be paid for assuming. Let it be emphasized that the contractor is paid. The cost of the investigation, plus a percentage on it, as well as the insurance, is somewhere in the bill just like the drummer's overcoat in the expense account. If such an investigation is made and bid submitted by but one contractor, the absence of competition can have but one effect upon the price. If several contractors investigate and bid there is an economic loss in the multiplicity of investigations. This multiplication of work results in an injustice to other work or to other projects undertaken; for however disguised or stated as "overhead charges," each contractor must saddle his expense for all investigations upon those projects which he does construct. Even the project under consideration pays the successful bidder. It must bear a proportion of the cost of that bidder's fruitless investigations of other projects, or else the contractor is in business for his health.

When, however, the contract is to be let on a unit price basis the attention of the contractor in making up Estimate No. 5 is devoted principally to one phase of the estimate, that of fixing the unit prices. It is usually assumed that the other phase, that of determining the quantities, has been studied with sufficient accuracy by the engineer in making up Estimate No. 4. The contractor's procedure in making up a schedule of unit prices is, to a greater or less degree, as follows: All operations, incidental as well as principal, which must be performed in order to finally deliver the completed work are each analyzed and resolved into elements such that their cost can be stated with sufficient accuracy. When all items of cost have been estimated and assembled, and certain allowances made for contingencies and profit, a total is arrived at which represents the total which should appear in the bid. The total, however, is the only point of resemblance between such an assemblage of items and the bid. The form must be changed and the items combined and redistributed with due regard to a balanced bid. In fact, possibly twenty items or shares of items are combined to make a total which divided by a certain number of cu. yd. gives the price to be bid per cu. yd. for that particular item. The process is strictly analogous to that applied by the engineer in determining quantities. Thus a mass is resolved into its measurable elements, *i.e.*, linear dimensions, which multiplied together give the mass. The contractor (although his process is more complicated as it involves results and processes of the engineer, judgment and experience) resolves the problem into elements as measurable as possible, the sum of which items gives him his total. Further description is not here necessary. The value and weight of the bid depend upon the detail with which the process is applied, as well as the intelligence and judgment of the bidder.

One further estimate, or rather half estimate, which may be called No. 6 is the final and accurate statement of quantities built. This statement taken in connection with the final and accurate, or rather the pertinent and binding, estimate of cost (*i.e.*, unit prices of Estimate No. 5) forms the final estimate or basis of payment for the work.

In thus outlining the nature and functions of the various Estimates Nos. 1 to 6 it is not intended to convey that the sequence of events is more than approximately stated. In any actual case a greater or less number of estimates may be made, they may merge into one another or serve the purposes of each other more or less. It is intended to convey that during the history of the project various estimates are required for various purposes, that normally each successive estimate is a step toward accuracy and that its expense is justified by the result of a previous estimate.

Obviously it is desirable that each estimate should approximate (as nearly as the available time and money will permit) to the accuracy of Estimates Nos. 5 and 6, in order that no injustice may be done to any of the interests concerned. Special attention is also invited to the importance of the estimates here designated as Nos. 2 and 3. Estimate No. 2 is a presentation of the case and No. 3 is practically an answer as to whether or not the project shall be undertaken. Each of them may be divided into (a) an estimate of cost and (b) a presentation or discussion of all collateral pertinent features upon which a judgment or estimate of the possible returns may be made. A treatment of the collateral features is outside the scope of the present work; they will not be here discussed except to say that as presented with Estimate No. 2, while they are often unduly optimistic, yet in a surprising number of instances some favorable feature is entirely overlooked or presented in an entirely inadequate manner. The optimism will be properly discounted by Estimate No. 3 and accompanying report, but the overlooked favorable feature may not be discovered or appreciated.

Regarding estimates of cost: Other things being equal they will carry weight and conviction in proportion as they show evidence of having been formed after careful analysis; *i.e.*, a reasonable determination of quantities based upon some survey and plan, and a subsequent procedure as outlined for contractor's estimate (of cost) No. 5. Thus a mere statement of 100,000 cu. yd. of masonry at $4.50, $450,000, while possibly a very excellent guess is not nearly as valuable and convincing as a plan or profile from which the quantity can be derived, accompanied by a tabulation of all the items entering into the cost, with a sum of $450,000.

The following tables, diagrams and data may be of some assistance in making up any of the above described estimates although they should be used only with some caution and an appreciation of their limits as to accuracy and consequent applicability for the particular estimate. The partial list of existing dams, with dimensions, quantities of masonry, cost and some accompanying pertinent notes, may (aside from its interest) be taken as a very rough indication of what another dam may cost if due regard is given as to whether the particular circumstances of the cases are comparable. Such particular circumstances are location, size, accessibility, price and quality of labor, cost of cement, amount of excavation and refill involved, amount expended in beautifying the structure and surroundings, etc. Obviously the length and maximum height as given in the table is only a very crude indication of the amount of masonry involved. For that reason, therefore, it would be much preferable to construct a profile of the dam and from the diagrams on pages

194 to 196 arrive at some number of cu. yd. as a basis for comparison. However, such analysis of and comparison with the table can at best furnish only a rough guide toward intelligent guess.

For purposes of estimates similar to Nos. 2 and 3 previously described, it would be necessary to have a fairly accurate profile across the valley or canyon at the dam-site, together with a fair indication from borings, test pits or otherwise, of the location of the rock surface; also some opinion as to depth to which it will be necessary to go into the rock for a foundation. With such information it should be sufficiently accurate to obtain cu. yd. of excavation and

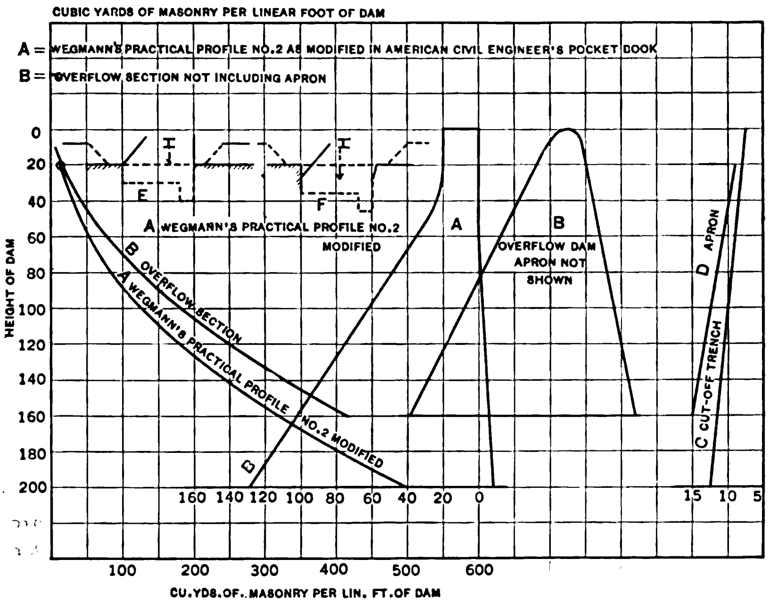

FIG. 40.—Curve showing amount of masonry per linear foot.

masonry from the diagrams; they are constructed from acceptable masonry sections, and the possible error should be much within that of the then available data. When, however, the project has reached a stage to warrant special studies and designs to meet all the particular conditions, much more accurate and detailed data in the way of surveys and borings will be at hand. Such diagrams will then be superseded by sections of the site and of the proposed structure.

Diagrams for Preliminary Estimates of Quantities.—For a preliminary estimate of the quantities involved, based upon profiles of earth and rock surfaces across the valley, use accompanying diagrams as follows:

For masonry in a dam without overflow, assume as acceptable, Wegmann's Practical Profile No. 2 as modified on page 616 of the American Civil Engineers Pocket Book. (See section "A" Fig. 40.) For cu. yd. of masonry per lin. ft. of dam read curve "A" for neat section to a horizontal base not including masonry in cut-off trench.

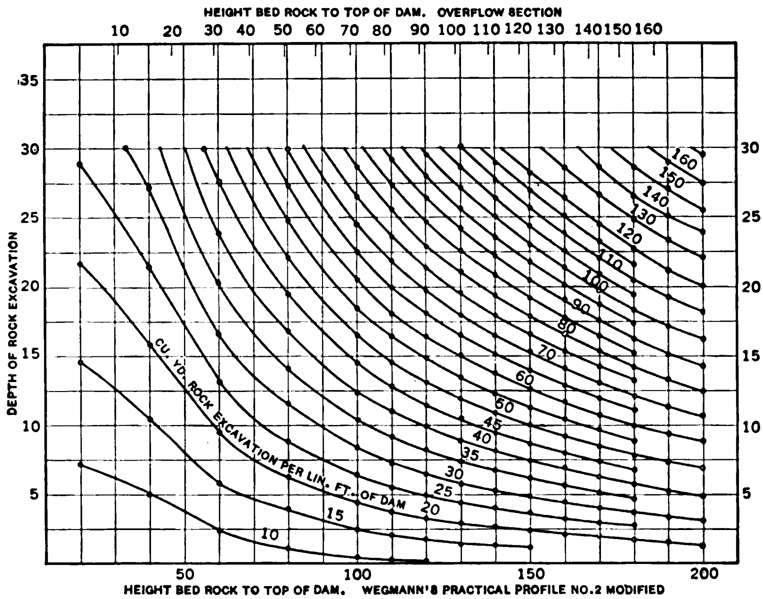

FIG. 41.—Diagram showing cu. yd. of rock excavation.

If the dam is built on surface of rock add for masonry in cut-off trench as per curve "C."

If rock is excavated and masonry slopes can start from the original rock surface, as at "E," read curve "A" for a height above rock surface, and add an amount equal to rock excavation as obtained from diagram Fig. 41.

If masonry slopes must be extended down to a certain elevation below original rock surface, as at "F," read curve "A" for a height of dam above that elevation, and add an amount equal to the rock excavation below that elevation.

For masonry in an overflow dam, proceed precisely as above, reading curve "B" Fig. 40. Then if on account of height of dam, or for another reason, an apron is necessary, add an amount obtained from curve "D."

For rock excavation, read diagram Fig. 41 in which ordinates equal depth of rock which it is assumed necessary to excavate; abscissa represent width of excavation in terms of height of dam, which height should be considered as starting from the elevation where the neat masonry slopes begin. Curves show cu. yd. per lin. ft. and include cut-off trench as per curve "C" Fig. 40. If

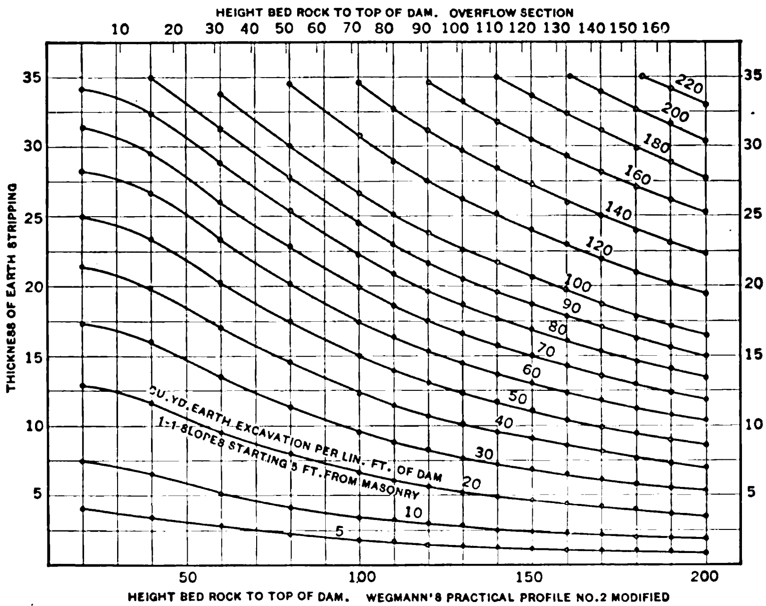

FIG. 42.—Diagram showing cu. yd. of earth excavation.

applied to an overflow dam with an apron, add to rock excavation as thus determined an amount at least equal to curve "D" Fig. 40, masonry in apron.

For Earth Excavation.—Read diagram Fig. 42 similar to rock excavation diagram, observing same rule for height of dam. Curves show cu. yd. per lin. ft. of dam for excavation to 1–1 slopes starting 5 ft. from neat lines of masonry. If applied to an overflow dam with an apron add for tentative estimate 1 cu. yd. per ft. depth of stripping. On both excavation diagrams are two scales for height of dam, according to which masonry section is being considered.

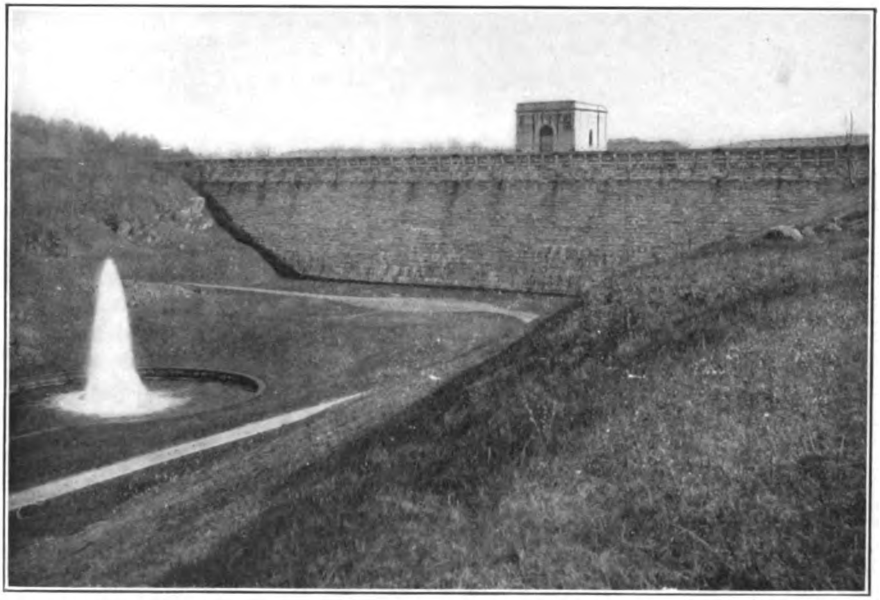

FIG. A.—Sodom dam.

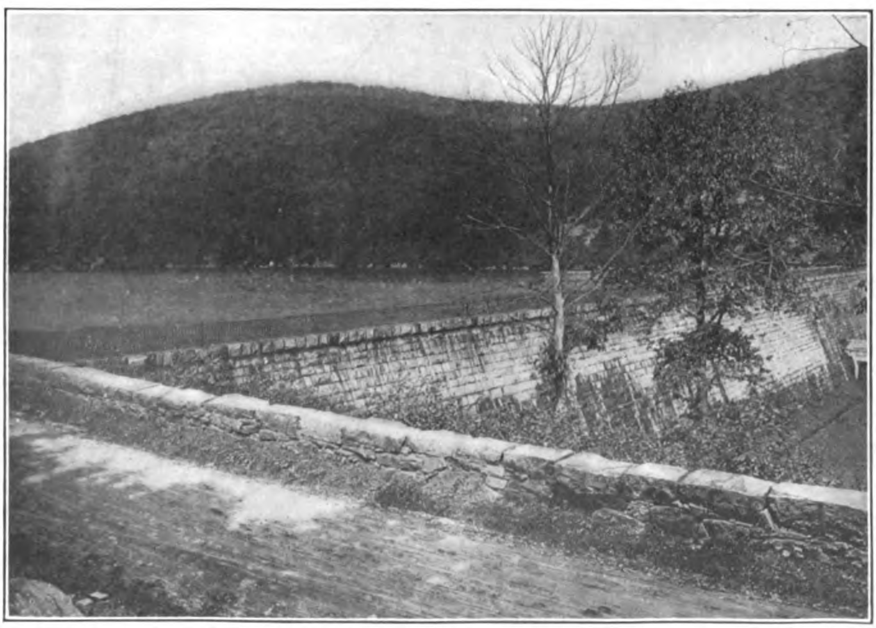

FIG. B.—Boyds Carners dam.

(Facing Page 196.)

PLATE XIV

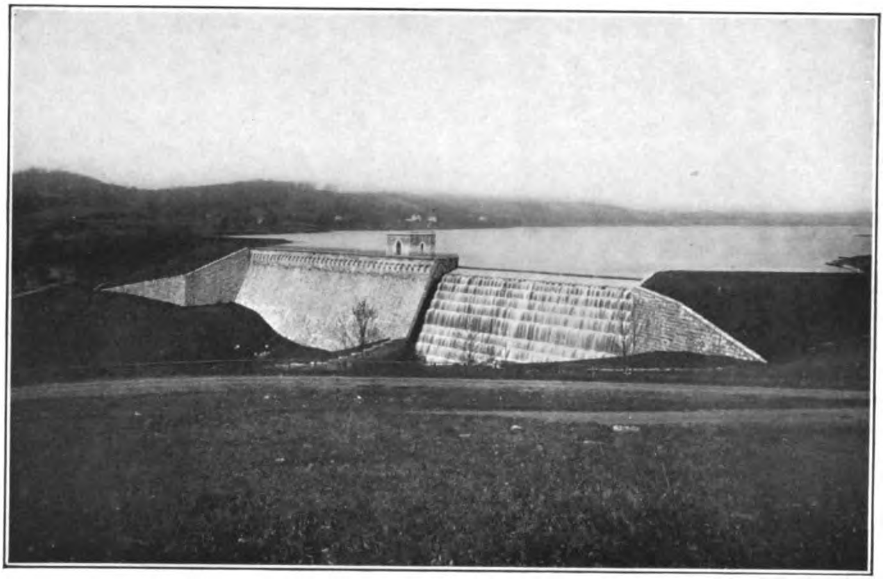

FIG. C.—Titicus dam.

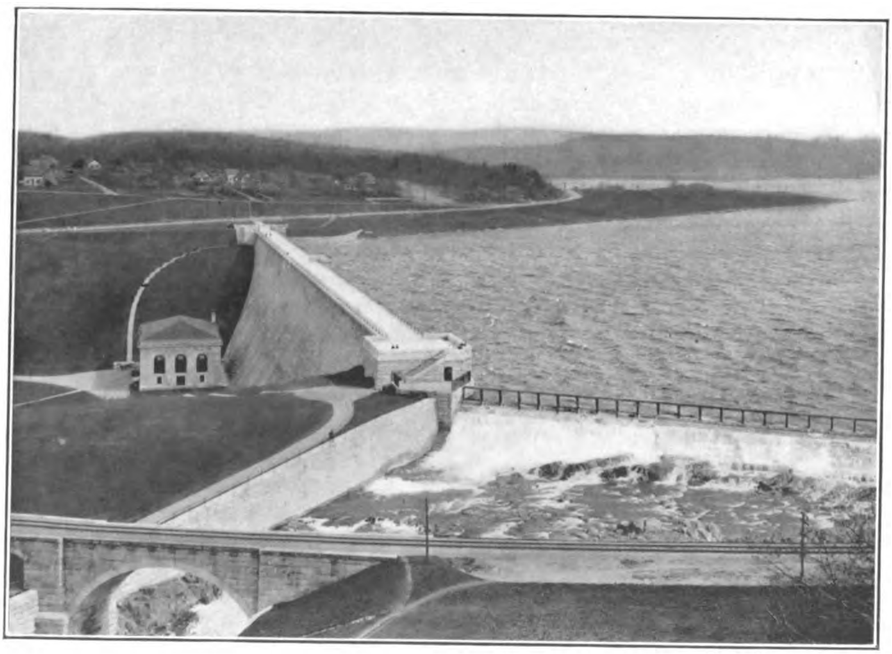

FIG. D.—Wachusett dam.

List of Items in an Estimate.—Although most of the various elements of cost have been discussed, as well as the various alternate modes of procedure for different conditions, the outline of an estimate will be here presented. Even at the expense of some repetition it will be as well to assemble the different items or possible items. Obviously no one estimate will include all of the items as many are mutually exclusive; some consideration, probably in most cases very brief, will eliminate many of them. Certain methods and items may be tentatively adopted subject to change on more complete or accurate information. According to the stage of the project, the purpose of the estimate, the accuracy of the surveys and information, the time and money available and the particular experience and judgment of the estimator; these items may be roughly estimated in total, or they may be analyzed, subdivided and figured to any required degree of accuracy.

A Plant;
B Expendible material;
C Material entering into work;
D Labor;
E General expenses;
F Contingencies;
G Overhead expenses;
H Profit.

A Plant.—Total plant charge may be considered under the following heads:

(1) First cost of plant at point of purchase.
(2) Freight on plant to railroad station nearest work; requiring that weights be approximately known and freight rates looked up.
(3) Haul from railroad station to dam site.
(4) Erection, including cost of materials (as cement and lumber) entering into the various housings and settings. Labor and materials of maintenance will be taken up elsewhere.
(5) Cost of haul back to railroad at end of work.
(6) Cost of freight to some point where plant can be sold or used on other work.
(7) Salvage value, a credit item; *i.e.*, the sum of items Nos. 1 to 6 minus No. 7 gives the total plant charge.

Item A–1. May be listed as

Power
Motors
Cableways
Tramways
Hoisting engines
Derricks
Pile drivers
Drills
Channelers
Stone-dressing machines
Pumps
Crushers
Grinding rolls
Mixers
Screens
Elevators
Bins
Locomotives
Cars
Track
Small tools
Conveyors
Dump buckets
Scows—boats
Machine, Carpenter, Blacksmith } Shops and equipment
Tanks
Steam shovels
Dredges
Rollers
Scrapers
Water carts
Carts
Wagons
Live stock
Camp and office buildings and equipment
Water supply
Sanitation
Stables
Lights.

Power should be further itemized depending on the source and manner of distribution:

For steam direct to engines, one or more boiler installations comprising
(1) Boiler house
(2) Chimney
(3) Water supply
(4) Boilers
(5) Feed pump
(6) Feed-water heater
(7) Fuel storage and handling equipment
(8) Distribution piping

For steam generation, compressed air distribution a central station comprising
Items 1 to 7, of proper capacity
(9) Steam piping to compressors
(10) Compressors including foundations
(11) House for compressors
(12) Cooling and condensing water supply
(13) Air receivers
(14) Air distribution piping

For steam generation, electric distribution
Items 1 to 7, house for engines or turbines
(15) Steam piping to engines or turbines
(16) Engines or turbines, including foundations
Item 12
(17) Generators
(18) Switchboard
If distance is large
(19) Step-up transformers
(20) Transmission line

For steam generation, electric distribution	(21) Step-down transformers or (22) Motor-generator (23) Sub-station building (24) Switchboard (25) Distribution wiring (26) Motor (27) Compressor and item 14
For hydro-electric generation, electric distribution	(28) Diversion dam (29) Head gates (30) Canal (31) Flume (32) Forebay (33) Screens (34) Penstock (35) Water-wheel including setting (36) Water-wheel governor Item 11 for water-wheels, generators, etc. Items 17 to 27 inclusive
For purchased electric power	Items 20 to 27 inclusive

In addition to the above it may be desirable to build a railroad to the work, for which estimate ties, track and fastenings, locomotives, cars, etc. The construction of such a road is here considered under Erection of Plant.

The main items of plant are merely suggested, others may be necessary on any particular piece of work. Selection of the proper kind and amount of plant can be definitely made only after a rather thorough study of the situation, including the amount and character of all the materials to be excavated, handled or disposed of; stream flow and measures for handling water; price and character of labor; climate; desired rate of progress, etc. For certain of the processes (as pumping) one cannot predict accurately the amount of plant required. It is well to provide largely, even an amount that will certainly meet any contingency, as carrying excess plant may be vastly cheaper than delay from insufficient plant. For the same reason duplicate or repair parts should be provided, especially for any items of plant whose breakdown would seriously affect the progress of the work.

Prices are practically never mentioned in any machinery or equipment catalogues. The estimator should aim to keep generally informed as to such prices, sufficiently so to be able to form an approximate estimate. Freight rates upon A, plant, B, expendible material, and C, material entering the work, should be a matter of inquiry at the time the site is being examined. In addition to the rates the total quantities and weights should be estimated as closely as may be. The haul from railroad to dam site may be of as much or more importance than the railroad freight. To settle properly the question involves estimates of cost of haul per ton-mile under the alternate or several possible methods. These estimates taken in connection with the total weights involved will indicate the justifiable expenditure for building or improving highways and bridges or for building and equipping a railroad. (See pages 127 and 128.) Broadly speaking any division between plant and erection of plant is largely fictitious, for an expenditure under head of erection is really a plant charge. The distinction is only made here in order that the items may be listed so that none will be overlooked, and to point the difference between plant which has a salvage value, and plant which has none.

Erection labor includes labor on highway or railroad to facilitate haul, building machinery foundations and housing, assembling the machinery and putting into running order, building cableway towers, grading for their foundations and laying track if they are traversing cableways, erecting camp and office buildings, stables, cement storage, sand and crushed stone bins, etc.

Erection materials would include cement for foundations, lumber, hardware, brick, etc., for the various buildings, or for culverts and bridges on the lines of transportation.

If, as is often the case, the contractor must construct the coffer-dams and other temporary work for the diversion and handling of water, and does not directly receive pay as an item in the contract, such works should be properly estimated as plant, or rather under the subdivision of erection of plant. Such estimate involves the determination of a scheme of diversion and a more or less definite plan for adequate works, unless both are specified. In any case it calls for a determination of quantities of materials required as piles, wood or metal sheet piling, lumber, hardware, cement, etc., and an estimate of the labor of building, including the necessary earth or rock excavation, embankment, etc. Whether such work and the expenditure on it is classified as plant or as an item of the contract is immaterial and only a matter of book-keeping; the item itself, it need not be pointed out, is of the utmost importance. It requires the ripest judgment as to design, plant and methods, and is usually after all the item principally in mind when estimating the amount which it is desirable to allow for contingencies.

Items A–5 and A–6 require no explanation; the information upon which they are based has been acquired and used for items A–2 and A–3. The destination and disposition of plant after a job is often an uncertainty, and in any case the transportation expense may be approximated closely enough.

Wherever and however disposed of item A–7, salvage, means the value of certain items of plant at some point of demand or place where they can be used on other work by the same or another contractor. The job under consideration must be charged with the cost of dismantling and transportation to some point where it becomes chargeable to another job. Where as is often the case the plant is to be used by the same contractor on other work the division of charges may be more or less arbitrary, in other words, a matter of book-keeping between two contracts and based upon a more or less casual valuation or estimate of depreciation. Where the plant is sold to another party the matter is not so simple. It will then depend, somewhat at least, upon cost to the purchaser of new or other second-hand plant at the point where he desires to use it. Entering into the problem then is the location of the second point of use as well as of the first point in their relation to centers of supply; also the amount and location of other second-hand plants which may be released at about the same time. It is quite possible if much work is in contemplation or is being prosecuted in a locality near the work under consideration and at the same time remote from a center of supply, that the salvage value will be a large per cent. of the original cost.

It may be good business to store the plant at some point (as at the nearest railroad station) that it must pass through to become available anywhere, and hold it for prospective adjacent work.

On the other hand, if surrounded by centers of supply or an abundance of other second-hand plant, if little new work is being let, or if it is desired to make a quick sale, a very much lower percentage of original cost will be realized.

On page 133 in discussing the use of hydro-electric power, a possibility was suggested which should not be lost sight of, *i.e.*, the continued use of portions of the plant, at the same location, as a permanent feature of the project or under the changed conditions created by the project. In such a case, as also in the case of any important highway or railroad built primarily to furnish access to the work, it is probable that this adaptation to permanent future conditions would have been contemplated by the promoters or builders of the project. If so, it would have influenced the financing of the project and the conditions of the construction contracts. In addition to features so broad and prominent as to have been considered in the original conception of the project, others might be found adaptable or necessary to the new or unforeseen conditions prevailing after completion of the work. Thus a small remaining community might make use of buildings, water supply, sewerage or lighting systems, refrigerating machinery, second-hand lumber, etc.

The foregoing considerations, however, can seldom be applied to any estimate of plant charge at the time (before the work is started) when such estimate must be made. Conditions two, three, or five years hence cannot be conjectured for such a purpose. The only course is to estimate the cost of getting it to a point of probable demand, and its sale value at that point. If circumstances happen to be such that the plant sells for more or less than the estimate then the difference is chargeable respectively to profit or loss upon the work. Obviously certain pieces of machinery have depreciated much less than certain others. Thus an engine, generator or air compressor which has been set on a good foundation, housed and cared for by intelligent operators will be worth a larger percentage of its original cost than other items as dump cars, concrete mixers, drills, rollers, scrapers, carts, etc., which have suffered the casualties incident to rougher contact with the work and the workmen. Certain portions of the plant as buildings are worth nothing for removal but may have some value if they can be used as buildings on the same location; or if they must be removed they may have a small value in the immediate locality as buildings or lumber. Other portions of the material or works charged originally as plant have no value at all. In fact, they are a liability to the extent of the cost of their removal in order to comply with the usual requirements for clearing up the grounds.

B—Expendible Material.—Material, supplies and minor equipment used up during the prosecution of the work may be listed as

Fuel
Blacksmith's coal
Explosives
Lubricants
Drill steel
Repair parts, or small shop supplies necessary to the maintenance of machinery in working condition; also small tools, as picks, shovels, rope, blocks, small hardware, lumber, feed for live stock, etc.

No hard and fast line can be drawn between these items and those larger and more permanent items considered under the head of plant.

Since one class of subsistence has been mentioned, brief consideration may be here given to subsistence of labor. While camp buildings were included under the head of plant, the actual operations of boarding the men and furnishing commissary supplies may usually be omitted from estimates. Such operations may be left to themselves, *i.e.*, considered as separate enterprises which will be self-supporting or even yield a small profit.

C—Material Entering into the Work:

Cement
Lumber
Steel (as reinforcing steel)
Pipe (steel, cast iron, wrought iron, vitrified, tile)
Valves
Gates
Screens
Guides
Operating machinery, etc.

Of the foregoing items cement is sometimes paid for as an item in the contract and sometimes not. In any case the cu. yd. of masonry and barrels of cement required per cu. yd. of various kinds of masonry are known, and cement prices can be readily obtained on inquiry. The simpler items of lumber, reinforcing steel or pipe can be readily estimated in quantity and prices obtained from current material price lists in the technical press.

When estimating specially designed pipe, valves, gates, operating machinery, etc., it becomes necessary to design the same and produce plans in sufficient detail so that they may be submitted to manufacturers and serve as a basis for bids. Such special features should be treated as a separate contract, the plans detailed by the owner's engineer and bids requested from manufacturers in those lines.

That the contractor for the main work should be required to burden himself with such a mass of unfamiliar detail in the short time usually allowed him in which to prepare his bid will result in an unsatisfactory treatment of the design or a very high price to cover contingencies. Such a requirement is altogether foolish and an economic waste. Preliminary or tentative plans furnish a sufficient basis for a bid per ton for setting metal work, leaving the details to be worked out later. In preparing such detail designs and obtaining bids from manufacturers, there are two extremes to the course open to the owner's engineer, either one about as much to be avoided as the other. He may accept manufacturers' designs and specifications without question, or he may ignore them entirely. If manufacturers' stock does not meet the conditions, if "made by the mile and sawed off to order stuff" does not always fit the case, it happens as often that some slight adaptations or immaterial changes in some other feature will render available the product of some manufacturer who has spent years in bringing that product to perfection. If entire surrender of his function may be called a sign of incompetency, an obstinate insistence upon it, to the extent of ignoring the manufacturer's experience and stock, may be equally such.

Labor.—After providing for plant and materials the next step is to estimate the cost of the labor required to perform the various items of work. To do this intelligently it is necessary that the main operations should be analyzed into their elementary parts and the labor estimated for each part. Thus laying masonry may be divided into producing a certain number of cu. yd. of large stone, number of cu. yd. smaller stone or waste for the crusher, crushing stone, producing sand, hauling, storing and handling cement, assembling certain materials at mixer, mixing, transporting to the point where the masonry is built, laying the masonry, etc. Certain of the above operations, as quarrying, may be further subdivided into drilling, loading and shooting, breaking or splitting to size, etc.

For certain operations the output or product will be almost directly proportional to the number of men employed, and prices per unit of output may be assigned for the particular circumstances and according to the experience and judgment of the estimator. Certain other operations or parts of operations, however, are practically an expense in proportion to time regardless of output, so that the cost per unit quantity varies inversely and as widely as the rate of progress. This being so, the various rates of progress between the beginning and end of the work, and the number of men required at each, should be estimated as closely as may be. Then a total duration of the work should be arrived at, and the cost of that operation should be estimated for the total time. That certain items, as for instance power-house operation, are distinctly of this nature would be evident to the most casual and inexperienced consideration. Certain other operations are less obvious, and all of them should be carefully scrutinized. The operation of a cableway costs as much for one derrick as for three or four.

A derrick requires an engineer for work at one-tenth its capacity as well as at full capacity. A given crushing and mixing plant will require just about the same number of attendants for an output of 100 cu. yd. per day as would the same plant for 1000 cu. yd. a pump at 100,000 gal. per day or 1,000,000 gal. The argument needs no further illustration. The question in any case is the extent to which labor may be released or transferred to other work whenever work must be prosecuted at a reduced rate. Thus it will be seen that in connection with nearly every operation there is an irreducible minimum expense no matter to what point the progress or output is reduced. Even in case of an interruption of such a nature and duration that all labor is laid off and operation entirely suspended for a time, there is still an expense for reconstructing the organization and bringing it to a state of efficiency on resumption of work, to say nothing of expense for protection of plant, watchman, insurance, interest, etc. Certain items of expense that are practically constant through both operation and shut-down are clearly to be classed as general expense, but it should be recognized that portions of so-called operating expenses partake of the same nature. The proportion in any case depends upon the plant, organization, nature of the operation, etc.

Whenever it becomes necessary to suspend work entirely the problem of unit cost is only to be met by a reduction of general expense, by discharging employees. The extent to which it is advisable to go in that direction is a matter of the known or estimated duration of the shut-down, probable dispersion of the force and the expense of reconstructing the organization when work is resumed. Whenever it is permissible or necessary to work, no matter at what rate of progress, the mere operation of plant for even the lowest rate entails at once a proportion (which may be considerable) of the total normal operating force. In such a case a satisfactory unit cost can only be attained, not by reducing the dividend, but by increasing the divisor; not in cheese paring or petty shaving of salaries but by boosting the output. To bring the force necessary for operation at half rate up to that necessary for progress at full rate is very largely a matter of adding to the number of laborers. At half rate many of the men (and those the very ones hardest to replace) are not working up to their capacity. While they thus have more or less opportunity for introspection it is unwise to furnish them with a grievance by cutting their pay.

LABOR COST OF MASONRY, ROOSEVELT DAM

	March 1908.			March 1909.	
	12,000 cu. yd.	1 cu. yd.		18,328 cu. yd.	1 cu. yd.
Cement tramway 1,700 ft. long. Took 8,958 buckets of sand and 5,626 buckets of cement to mixer. Average force:			Took 13,538 buckets of sand and 9,822 buckets of cement to mixer. Average force:		
1 Engineer at $3.50 4 Laborers at 3.00 } 27½ day shifts	$ 437.00	....	1 Engineer at $3.50 4 Laborers at 3.00 } 29½ day shifts	$451.44	....
2 Engineers at 3.50 4 Laborers at 3.00 } 25 night shifts	472.00	....	1 Engineer at 3.50 5½ Laborers at 3.00 } 28 night shifts	581.00	....
Two Lidgerwood cableways, 1,200 ft. long. Took from mixer to dam, 737 skips mortar, 1,380 skips concrete. Quarries to dam, 630 skips spalls			Took from mixer to dam, 1,056 skips mortar, 2,571 skips concrete. Quarries to dam, 1,376 skips spalls		
2 Engineers at $4.00 2 Signalmen at 3.00 4 Tagmen at 3.00 } 26½ day shifts	688.00	....	2 Engineers at $4.00 2 Signalmen at 2.50 4 Tagmen at 3.00 1 Oiler at 3.50 } 28 day shifts	795.06	....
Night Shifts.—Collecting stone in quarries, transporting to dam, and distributing, also moved some derricks. Took to dam, 5,400 cu. yd. stone. Took to crusher, 234 cu. yd. stone. 2 Foremen at $5.00 1 Foreman at 4.00 1½ Electrician at 3.00 1 Watchman at 2.50 2 Engineers at 4.00 2 Signalmen at 3.00 5 or 6 Engineers at 3.50 8 Laborers at 3.00 8 Laborers at 2.50 } Average force for 29 shifts.	2,829.06	....	Took to dam, 7,900 cu. yd. stone 4 Foremen at $4.00 and $5.00 1 Watchman at 2.50 2 Engineers at 4.00 2 Signalmen at 2.50 7 Engineers at 3.50 18 or 19 Laborers at 2.50 } Average force for 29 shifts	3,097.64	....
Transporting materials cost	$4,426.06	0.369		$4,925.14	0.269

Quarries:			
3 Foremen at $5.00 6 Engineers at 3.50 4 Drillers at 3.00 4 Helpers at 2.50 7 Quarrymen at 3.00 24 Laborers at 2.00 1 Blacksmith at 4.00 1 Helper at 2.50 1 Powderman at 4.00 1 Helper at 2.50	Average force, north side, 29½ day shifts	$4,155.50	....
1 Foreman at $4.00 1 Engineer at 3.50 4 Drillers at 3.00 4 Helpers at 2.50 10 Laborers at 2.50 ½ Tagman at 3.00 ½ Powderman at 3.50	Average force, north side, 29 night shifts	1,676.81	....
1 Foreman at $5.00 1 Engineer at 3.50 1½ Driller at 3.00 1½ Helpers at 2.50 5 Quarrymen at 3.00 6 Laborers at 2.50 4 Laborers at 2.00 ½ Powderman at 3.50	Average force, south side, 28½ shifts	1,612.50	....
		$7,444.81	0.620

Quarries produced, solid cu. yd.
5,134 cu. yd. rubble stone
1,676 cu. yd. for crusher
1,134 cu. yd. spalls
2,170 cu. yd. waste
10,114 cu. yd. cost $0.736 per cu. yd.

2 Foremen at $5.00 1 Foreman at 4.00 5 Engineers at 3.50 7 Quarrymen at 3.00 20 Laborers at 2.50 4 Laborers at 2.25 3 Drillers at 3.00 3 Helpers at 2.50 1 Blacksmith at 4.00 1 Helper at 3.00 1 Powderman at 5.00 1 Helper at 2.50	Average force, north side, 29½ day shifts	$4,087.35	....
Foreman, 10 hr., $4.82 Engineer, 14 hr., 6.10 Driller and helper 24 8/10 hr., 16.91 Labor, 106 hr., 33.29	Average night force, both quarries, driller and helper, 30 shifts—all others 26 shifts	1,656.46	....
1 Foreman at $5.00 4 Engineers at 3.00 1 Laborer at 3.00 16 7/10 Laborers at 2.50 7 8/10 Laborers at 2.25 131 6/10 Hr. driller and helper at 68¾ cents 1 Powderman at 4.00	Average force, south side, day shift, drillers 25¾ shifts—all others 28½ shifts	2,621.68	....
		$8,365.49	0.450

7,900 cu. yd. rubble stone
2,790 cu. yd. for crusher
2,200 cu. yd. spalls
3,150 cu. yd. waste
15,820 cu. yd. cost $0.529 per cu. yd.

LABOR COST OF MASONRY, ROOSEVELT DAM.—(*Continued*)

	March 1908.			March 1909.	
	12,000 cu. yd.	1 cu. yd.		18,328 cu. yd.	1 cu. yd.
Used upstream face stone for 1,790 sq. ft. For cost of cutting, see page 89 1,790 sq. ft. at $0.235. 1½ Per cent. of total masonry was face stone.	420.65	0.035	Used upstream face stone for 7,500 sq. ft. at $0.235 4½ Per cent. of total masonry was face stone.	$1,762.50	0.096
Mixing mortar and concrete: Materials drawn from bin into car, transferred 10 ft. and dumped into a 40 cu. ft. capacity Smith mixer, dumped into a skip on a car and hauled by a stationary engine either 50 ft. to No. 1 cableway or 125 ft. to No. 2 cableway.					
Mixed 1,474 batches of mortar = 1,615 cu yd. Mixed 2,760 batches of concrete = 3,616 cu. yd.			Mixed 2,112 batches mortar = 2,323 cu. yd. Mixed 5,142 batches concrete = 6,736 cu. yd.		
1 Foreman at $3.50 1 Engineer at 3.50 3 Laborers at 3.00 1 Laborer at 2.50 } 26½ shifts...... Cost per cu. yd. mixed, $0.093.	490.25	0.041	28½ Shifts...... Cost per cu. yd. mixed, $0.057.	520.31	0.028
Laying masonry, three derricks. Average force: 3 Foremen at $6.00 3 Engineers at 3.50 6 Tagmen at 3.00 6 7/10 Masons at 5.00 16½ Laborers at 3.00 ½ Waterboy at 2.00 } Actual total cost, 26½ shifts For six shifts one gang was split and with extra engineer and tagman ran another derrick.	3,506.25	0.291	Five derricks. Average force: 5 Foremen at $6.00 5 Engineers at 4.00 10 Tagmen mostly at 2.50 7 Masons mostly at 5.00 23½ Laborers at 2.50 1 Dogger at 3.50 } Actual total cost, 28½ shifts	4,840.50	0.264
676 Hours derrick = 17.7 cu. yd. per derrick-hour.			1073 Derrick-hours = 17.1 cu. yd. per derrick-hour		

Moving derricks, cleaning up underneath, and some moving from and back to the wall on account of anticipated flood. 77 Men—days	$249.18	0.021	Included in work of night shifts.		
Repairs chargeable to masonry. 84¾ Men—days	$298.21	0.025	170½ Men—days	$616.25	0.034
Pointing upstream face. Not done this month, assumed as same per sq. ft. of face as for March, 1909.	38.30	0.003	46½ Men—days	160.25	0.009
General expense. Superintendent, master mechanic, time-keeper, electrician, clerk.	837.50	0.070		848.00	0.046
Power. Purchased from Government. 144,236 h.p-hr. at $0.005 ... $721.18		0.060	165,013 h.p-hr. at $0.005 ... $825.06		0.045
Attendance, 2 men ... 175.00	896.18	0.075	Attendance, 2 men ... 175.00	1,000.06	0.055
Summary:					
Transportation materials	4,426.06	0.369		4,925.14	0.269
Quarrying	7,444.81	0.620		8,365.49	0.456
Cutting upstream face stone	420.65	0.035		1,762.50	0.096
Mixing mortar and concrete	490.25	0.041		520.31	0.028
Laying masonry	3,506.25	0.291		4,840.50	0.264
Moving derricks	249.18	0.021			
Repairs	298.21	0.025		616.25	0.034
Pointing upstream face	38.30	0.003		160.25	0.009
General expense	837.50	0.070		848.00	0.046
Power	896.18	0.075		1,000.06	0.055
Total	$18,607.39	1.550		$23,038.50	1.257

Above includes all labor, ordinary repairs for the month (except repair materials) and power; does not include plant charges, interest, insurance nor any material or supplies.

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Relation of Output to Unit Cost at the Roosevelt Dam.—As an illustration of the principles above set forth, attention is invited to figures (pages 206 to 209 inclusive) for the labor cost of masonry for two months upon the Roosevelt dam. They are revised slightly from some that were published at the time. Comparing two such periods it is to be expected that certain items of expense would be constant and the unit cost vary inversely as the output; also that certain other items would vary nearly as the output and show nearly constant unit costs. Thus for the two months under consideration, March, 1909, with an output of 18,328 cu. yd. should show for constant items as power-house attendance or general expense, unit costs 65.5 per cent. of those for March, 1908, with an output of 12,000 cu. yd. The other extreme of 100 per cent. would be reached in the case of an item where the total expense varied exactly as the output. However, no such item occurred in this case and might indeed rarely be possible. Arranging the unit costs in order between the two above extremes, we have:

	March, 1908	March, 1909	Per cent. 1909 of 1908
Power-house attendance, constant	$0.0146	$0.0095	65.5
General expense	0.070	0.046	65.7
Mixing	0.041	0.028	68.3
Transporting materials	0.369	0.269	72.9
Quarrying	0.620	0.456	73.5
Power, not including attendance	0.060	0.045	75.0
Laying masonry	0.291	0.264	90.7

The first two items show the expected percentages. Mixing was very largely an operation where certain attendants were required to run the equipment irrespective of output. Transporting materials show somewhat more flexibility in the force engaged. In the matter of quarrying, it must be said that quarry conditions are very difficult to compare; unit costs were based upon quantities used in the masonry which may not strictly represent relative quantities produced. If accurate figures could have been kept for production as distinguished from use the operation might have shown up more in proportion to output. However, there could not have been a wide difference from the foregoing figures, and we may see that the quarry force expanded or contracted more as one large unit than as several small ones. In the item of laying masonry a derrick and a certain force constituted a much more definite unit than in the

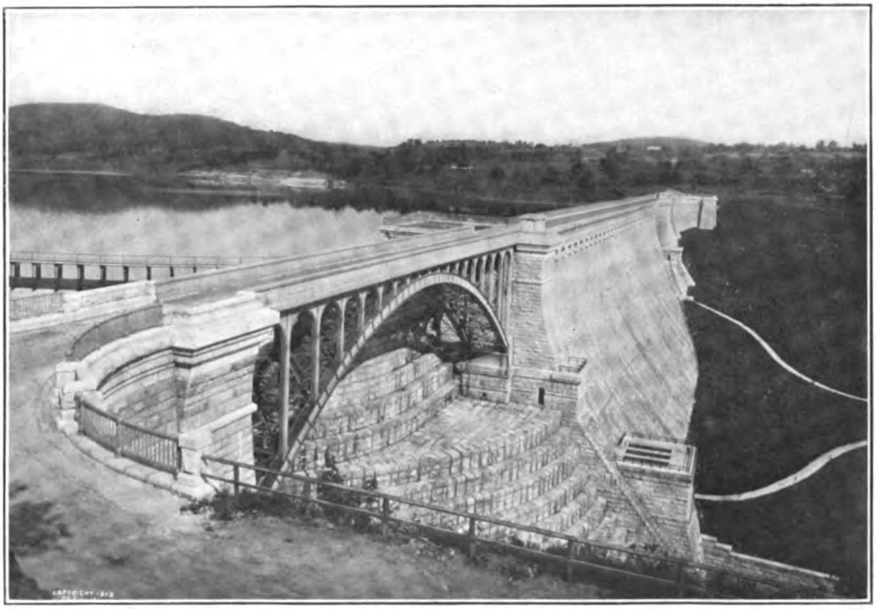


FIG. A.—New Croton dam.

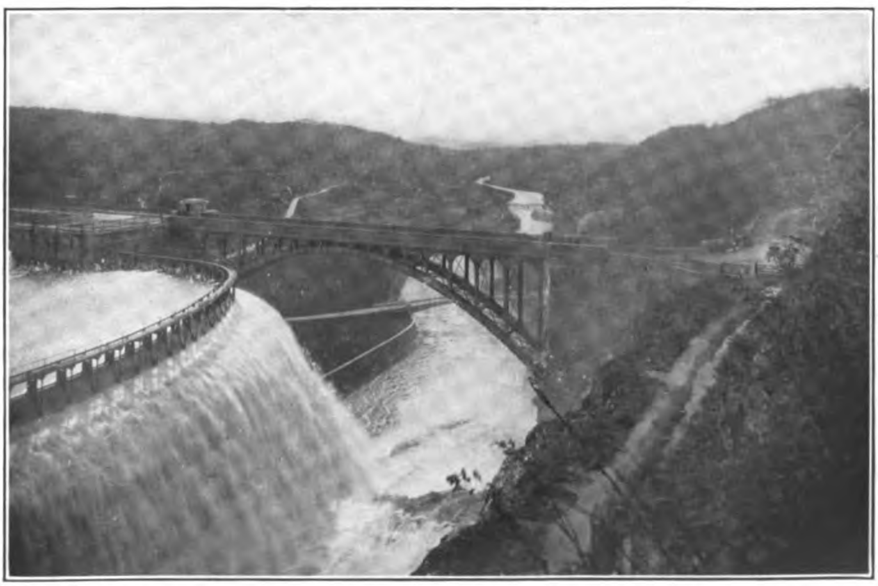


FIG. B.—Spillway and bridge, New Croton dam.

(Facing Page 210.)

Plate XV

Fig. C.—Cross River dam.

Fig. D.—Shoshone dam.

quarry, and increased production was accomplished by increasing the number of units from three to five. That there was a slight reduction of unit cost was due to the evolution of more efficient and economical forces during the year, instead of to increased output.

Attention has been directed to the cost of items which are not paid for directly as items of the contract. Many indeed are not mentioned at all. It is only by analyzing and itemizing all the operations, and carefully considering what plant and process is involved in each case, that a complete list will be arrived at. Such items as stripping of quarries and sand pits are of course naturally included in the estimate. Others are far less obvious and an attempt will be made to suggest them under the head of contingencies.

The relative size and importance of the total for an item has a bearing on the process of making up an estimate. Thus if accompanying an item of 300,000 cu. yd. of rubble is an item of 300 cu. yd. of dimension stone, it is useless to expend gray matter in figuring whether the latter will cost $35 or $40 per cu. yd. An equal accuracy would require that the cost of the rubble be determined to ½ cent per cu. yd. Again if certain temporary works be estimated at $100,000 and one-half of it is to be charged off to the excavation of 20,000 cu. yd. of material below water level, it is immaterial whether the cost of such excavation be figured at 50 cents or 60 cents.

E. General Expense.—Whatever the system of accounting may be, or however accounts may be kept to show the cost of various features of the work, it is necessary or convenient to lump under general expense many items that cannot at the time be readily assigned to another account. At the end of the work this account may be distributed among any number of feature cost accounts in proportion to their other costs, their credit amounts or in any other manner desired. Obviously no definite line may or need be drawn between the items charged to general expense and those charged to the particular features. Certain of the plant as office buildings may be carried either as plant or as general expense. Certain salaries as for book-keeper and time-keeper, may obviously be distributed at the end of the work; others such as for general superintendent, master mechanic, power house and repair shop employees merge gradually into those chargeable immediately to their proper accounts.

Material accounts may be charged with the materials when acquired and credited when the materials are used and charged to the feature upon which they are used. Some materials may be chargeable to general expense, as also would be any discrepancy between material accounts and stock actually on hand upon taking inventory or at end of work. However, details of accounting systems need not be entered upon here. It will only be pointed out that an accurate and intelligent final analysis requires that as far as possible expenses should be distributed when incurred.

The suggested principal items of general expense are:

Expense of investigating, estimating and securing the work (see also overhead expense).
Maintenance of the field office.
Certain salaries as for book-keeper, time-keeper, general superintendent, postage, telephone, telegraph.
(Freight bills go primarily to the plant or material accounts.)
Traveling expenses.
Unaccounted for discrepancies in plant or material accounts at end of work.
Legal expenses.
Entertainment expense.
Police; enforcing sanitary regulations, etc.
Interest on capital (a) Invested in plant.
(b) Advanced on materials entering into the work.
(c) To carry the pay-roll until such time as the monthly estimates will do it.
(d) Retained percentage.
Cost of the bond usually required for faithful performance of the work.

F. Contingencies.—Items of expense which may possibly be encountered, but which still are not anticipated to a degree justifying their inclusion in the estimate should be considered and allowed for under the head of contingencies. Contingencies should also include some insurance against absolutely unforeseeable accidents; in fact, the entire matter is insurance. Certain prices, which are considered fair and altogether probable of realization, have been fixed for the various items. The contingent fund is a portion of the difference between such probable figures and the figures under the worst possible conditions. It is only a portion of the difference for, according to probabilities, all items cannot become adverse. Further, any one item is seldom adverse to the limit. It is idle to discuss what proportion the contingent fund might properly be. It might vary in each case depending on the duration of the work, the completeness or information as to conditions, the experience of the bidder and for many other reasons.

In case the owner is able and willing to carry such cost of insurance a contract may be entered into whereby the contractor does the work on a basis of cost plus either a percentage or a fixed sum. Such percentage or fixed sum secures the contractor's experience in such work, the benefit of his already assembled and available plant and perfected organization. Under such an arrangement it might be said that the contractor is less liable to put forth strenuous efforts to keep down the cost of the work. Whether such carrying of insurance and guarantee of a reasonable profit would be a good arrangement for the owner depends in any case upon the nature of the work and the character and experience both of the contractor and the owner's engineer. At least it is an increasingly common way of conducting work.

Contingent Items.—Some of the principal points which should be considered in connection with the estimate of a contingent item are here suggested. No claim is made that the almost impossible task of including them all has been accomplished.

Labor.—Whether or not labor will be obtainable throughout the duration of the work at the prices estimated.

Possibility of strikes.

Whether labor will have to be imported at an expense for labor supply agents, railroad fares, etc.; if so, the percentage of desertions.

Laws governing conditions and hours of labor should have been considered in connection with making up the estimate proper but there may be features more or less uncertain.

Materials and Supplies.—Prices may vary greatly during the life of the contract.

For important items such as cement and fuel it is the common and safe practice to contract with mills or dealers for the entire quantities to be required. If a bid is made without contracting for materials and supplies, the bidder has simply put himself on the short side of the market, and the contingent item is to cover his possible error in judgment.

For most items other than cement and fuel the requirements are so small that any ordinary fluctuations in price would be immaterial.

Freight rates may be considered fixed, but the conditions of haul from railroad to work should be scrutinized for possible changes.

Climate and Weather Conditions.—The number of interruptions and amount of lost time on account of rain or snow or temperature so low that work cannot be prosecuted.

Frequency, size, and effect of possible floods. Judgment as to frequency and size of floods must be based upon rainfall and run-off figures for that watershed or upon such figures for an adjacent and similar watershed of known relative size. Adequacy of proposed temporary works for diverting the water and possible damage by a flood exceeding their capacity. Such damage includes cost of repairing or rebuilding the works on the same or another plan, cost of resulting delay, cost of re-excavating loose material which may have washed into the pit, possible loss of plant, etc.

Quarries and Sand Pits.—An uncertainty regarding a quarry is seldom as to the amount of stone but as to the shape in which it will come out; whether it will furnish at the estimated cost such large and well-shaped stone as may be required. This matter was discussed in the chapter on Quarrying. A contingent item may in some cases include the estimated cost of importing a proportion of the stone from a proved quarry. In the case of a deposit of sand of satisfactory quality, the uncertainty is as to whether the deposit has been prospected sufficiently to show that the required quantity is available. There should be borne in mind the possibility of having to go further and open another pit or even of resorting to manufactured sand. The character of the earth to be excavated at the dam site has usually been determined with ample accuracy from the borings put down to locate the rock surface.

The experience, ability and above all the character of the engineer as affecting the progress of the work and the cost to the contractor.

The possibility of expense for right of way in addition to what had been estimated or secured.

Expensive or burdensome regulations by local legislative or licensing bodies.

Solvency of the owner; expense, delay or legal procedure to secure pay for work done. If, as is sometimes the case, it is stipulated in the contract that the contractor shall accept his pay wholly or partly in bonds of the enterprise, then the value of such securities is partly discounted in the estimate and is partly an uncertainty.

Payments to placate political parties, to secure contracts or for prompt action on final estimates and claims. In the light of recent events this item might in New York State properly be classed as an overhead expense.

Accidents resulting in damage or breakdown to portions of the plant; also injuries to workmen resulting in claims for damages or compensation. If the latter risk is carried by the builder it is properly a contingent item; if the builder takes insurance in a casualty company the usual basis for paying for it is a percentage (often 2 per cent.) of the pay roll, and it should then be classed as a portion of the labor cost.

Losses on account of disputed or disallowed claims, or arising from ambiguities in or misunderstanding of the contract.

Omissions.—After the estimate or bid has been completed, there always remains the chance that some item has been entirely overlooked. This brings to mind a certain picture entitled "The Successful Bidder" which represents more grim truth than it does poetic license. In it is shown a disheveled contractor on the verge of nervous prostration, clutching a notice of award of contract and wildly questioning "What did I forget?" There is no remedy, nothing to say. The matter is simply one of the experience of the estimator in that kind of work under similar conditions.

G. Overhead Expenses.—A more or less arbitrary proportion of certain expenses which are or may be practically constant whatever the volume of business. For example, expense of main office and organization as distinguished from those pertaining and chargeable to a particular project.

Advertising.—The previously mentioned expenses for investigations which do not result in work.

Losses on other jobs.

Interest on capital invested or required to carry on the work.

Insurance and depreciation on idle or reserve equipment.

N. B.—The expense of investigating, estimating and securing a particular piece of work may be carried for a time under the name of the particular project; later, depending upon whether or not the work is secured, it may be called general expense of that project or transferred to overhead expense.

H. Profit.—Various items suggested under the head of Contingencies may affect the percentage to be added as profit. Thus if payment is to be received in bonds they must naturally be discounted somewhere either in the specifications or in the bid or in both. If, say it be a provision of the contract that they shall be accepted at 90, and in the estimation of the contractor they be sold only at 87, then the difference of 3 per cent. must be considered in fixing the item of profit. Should there be further a possibility that the contractor could under certain circumstances realize only 85 for them, the chance of the 2 per cent. loss should be included in contingencies.

The identity and experience (if known) of other bidders for the same work, the amount of other prospective work, whether contractors generally are busy or not, general business conditions actual and prospective, the percentage which is to be preferred rather than the alternative of not securing the work, the bidder's reputation for experience, ability and resources for certainly and satisfactorily performing the work, as compared with the reputation of other bidders, are all items to be considered in fixing the percentage of profits. Such considerations may merge gradually into a class which should be included under other heads. Thus a strategical position in respect of location, amount, and immediate availability of plant, or a monopolistic position as in exclusive possession of certain appliances or processes.

It is a not uncommon belief that work done under a cost plus percentage basis is done for a smaller percentage of profit than is figured in work done under the system of unit prices. This comes from a confusion of the items of contingenciės and profits, or an illogical assignment of the elements entering into each. Assume that under the cost plus percentage system 15 per cent. profit is a fair percentage, one that the contractor is willing to do business upon. Then if on adjacent work under unit price bid he adds 25 per cent. to his estimate of cost for no other reason than that it is under the different system, it is simply an index to his mental process in attempting vaguely to include under one item something that he thinks he may not have included under another.

Preparation of a Bid.—After all the foregoing items have been assembled to form a total for which the bidder is prepared to do the work, they must, as indicated on page 192, be combined and redistributed to conform to the items appearing in the proposal.

Thus price per cu. yd. for masonry =

Sum of all labor costs necessary to build that number of cu. yd. of masonry

Opening quarry labor quarrying,	Furnishing sand,
Transporting materials,	Mixing,
Crushing,	Laying, etc.

Materials entering masonry and not otherwise paid for.—Portion of cost of temporary diversion works and pumping (the other portion is borne by such excavation, earth or rock as is below water level). —Portion of expendible material; for accuracy this may be itemized.

Thus, may include portion of	Fuel, Blacksmith's coal, Lubricants, Repair parts, etc.
And entire items of	Explosives, Drill steel, etc.

Portion of plant—likewise itemized for accuracy.

May include all of { Drills, Crushers, Mixers, Screens, Elevators, Bins, Dump buckets, etc.

And portion of { Power plant, Derricks, Cableways, Locomotives, Shops, etc.

And none of certain items not connected with masonry.

A proportion of the sum of items { General expense, Contingencies, Overhead expense, Profit.

It is immaterial how such proportion is determined in the case of masonry or any other item in the proposal. The bidder naturally prefers to get as large a proportion as possible onto those items which will be encountered or completed at an early stage of the work. This desire must be modified by the consideration of a well-proportioned or "Balanced Bid"; any other will be scrutinized and regarded with disfavor. Thus excavation below water may be completed before masonry begins. It would be very pleasant to get back the cost of diversion works at that time, or even at the time the diversion works are built. However, the bid must be a compromise between the expenditures and desires of the contractor and the possible situation of the owner should the works be destroyed after he had paid for them and before they had served their purpose. When all the items entering into the cost (as above outlined for masonry) have been assembled, the total divided by the number of cu. yd. represents the price to be entered as the bid for the item.

Freak figures are occasionally seen. For example, when it is obvious that the bidder did not wish to have it appear what he thought the item worth, and puts in something ridiculous for that item, balancing the discrepancy by altering other items; or as when on a hunch that a competitor may bid a certain figure, he puts in a fraction of a cent less.

FIGURES AS TO COST OF PLANT, PLANT ERECTION REPAIRS, EXPENDIBLE MATERIAL, ETC.

After the foregoing discussion of estimates, their purposes, the manner of making and assembling them and the listing of the items, it will be pertinent and perhaps interesting, if some actual figures are given. Extreme accuracy cannot be expected of such figures, and where the time is available they should be checked by independent inquiry. They should, however, be of assistance to any one who must in a short time make or check an estimate.

Industrial Railroad.—After grading has been done or estimated, wooden trestles as follows: 10 to 12 ft. B. M. per sq. ft. of opening spanned, *i.e.*, between ground surface and base of rail, estimate $15 to $18 per M. ft. B. M. for erection. Wooden Howe truss bridges —say 32 M. ft. B. M. for an 80-ft. span, to 60 M. ft. B. M. for a 140-ft. span, and allow $20 to $25 per M. ft. B. M. for erection. Abutments extra.

Steel bridges, deck girder	50-ft. span	35,000 lb.
E45 loading, deck girder	75-ft. span	74,000 lb.
Deck girder	100-ft. span	125,000 lb.
Through riveted truss	130-ft. span	204,000 lb.
Through riveted truss	175-ft. span	342,000 lb.

Any of the above erected at 4½ cents to 5 cents per lb.; abutments extra.

For prices of lumber, ties, rails, fastenings, bolts, see current material prices in technical papers.

Equipment, freight locomotive,	60 ton weight about	$11,000
	100 ton weight about	16,000
	140 ton weight about	22,000
Box cars		$400 to 600
Flat cars		200 to 400

For actual cost of construction, equipment, and two years operation of an industrial railroad see page 128.

Cableway 2¼-in. lock strand including a 75-h.p. engine, and all lines, sheaves, and accessories, not including towers, may be assumed for a preliminary estimate as ranging from $10,000 for 700-ft. span to $15,000 for 2000-ft. span. In the case of fixed towers, the cost of the anchorage may vary widely according to the material anchored in, say from $100 to $400 per anchorage.

Towers may be estimated about on the following basis:

	A frame 75 ft. high	4 Post 75 ft. high	4 Post extra heavy 75 ft. high	4 Post 100 ft. high	Traveling 57 ft. high	Traveling 90 ft. high	Intermediate 230 ft. high for two cables[1]
Timber (so. pine)	12 in. × 12 in. posts 3 ft. × 12 in. bracing 4,000 ft. B.M.	10 in. × 10 in. posts 3 in. × 12 in. bracing 10,000 ft. B.M.	12 in. × 12 in. posts 4 in. × 12 in. bracing 13,000 ft. B.M.	12 in. × 12 in. posts 4 in. × 12 in. bracing 19,500 ft. B.M.	.... 25,000 ft. B.M.	12 in. × 12 in. posts 4 in. × 12 in. bracing 50,000 ft. B.M.	12 in. × 12 in. posts 4 in. × 12 in. bracing 101,000 ft. B.M.
Iron work	Including truss rods, $100	$75	$75	$100	$250	$450	Largest tower ever built
Labor, framing, and erecting.	$50 to $100	About $20 per M. ft. of lumber	About $20 per M. ft. of lumber	$15 to $20 per M. ft. of lumber	$15 to $18 per M. ft. of lumber	$15 or $16 per M. ft. of lumber	
Second-hand railroad axles.	....	....	....	....	14 at $15 to $20	20 at $15 to $20	
Boxes	....	....	....	....	28 at $7	40 at $7	

For traveling towers add for grading, ties, and rails. Estimate 75-lb. rail, say five times the length of travel for 57-ft. tower, and seven times the length of travel for 90-ft. tower.

[1] Later raised 60 ft., making a total height of 290 ft.

At the Kensico dam the 1861-ft. span, $2\frac{1}{2}$-in. lock strand, 10-ton cableways, with 400-h.p. electric engines giving a hoisting speed of 300 ft. and a conveying speed of 1200 ft. per minute, on 125-ft. high traveling towers, base of tower 50 ft. × 100 ft., with fifty-eight wheels under each tower. Cost above the rails, in place about $25,000 each.

Cableway engine:	
75 h.p., steam (not including boiler) or compressed air, with drum to move the towers, about	$3,300
Same with electric motors	$3,700 to 4,000
Water wheels:	
3800 h.p. at 250 ft. head, at 500 r.p.m., weight 83,000 lb	7,500
Lombard governor for above	1,600
1800 h.p. at 46-ft. head, at 200 r.p.m., vertical type, single runner, including oil governor, thrust bearing, penstock, and draft tube,	12,000
180 h.p. at 46-ft. head, at 425 r.p.m., including mechanical governor about	3,200
500 h.p. at 20-ft. head, including governor, 26,000 lb. to 32,000 lb. according to speed	$3,400 to 3,800
500 h.p. at 30-ft. head, including governor, 16,000 lb. to 17,000 lb. according to speed	$2,300 to $2,500
Generators:	
D.c., 500 kw., 150 r.p.m., weight 39,000 lb	4,500
A.c., 400 kw., 225 r.p.m., three-phase, sixty-cycle, 6,600 volts, 36,000 lb	4,800
A.c., 320 kw., 257 r.p.m., three-phase, sixty-cycle, 2,300 volts, 23,300 lb	4,000
A.c., 320 kw., 450 r.p.m., three-phase, sixty-cycle, 2,300 volts, 17,600 lb	3,100
A.c., vertical type, 300 kw., 164 r.p.m., three-phase, sixty-cycle, 2,300 volts, 28,000 lb	4,600
Exciter for either of the four above 14 k.w., 125 volts, including sliding base, pulley and field rheostat, 1,600 lb	400
A.c., vertical type, revolving field, 1400 kv-a., three-phase, 60-cycle, 200 r.p.m., 2300 volt	$9,500 to $10,000
Parsons type steam turbine, electric generating unit 500 kw., three-phase, sixty-cycle, 2300 volts about	14,000
Boilers:	
Per h.p	$10 to $12
250 h.p. weighs 20 tons.	
Transmission line:	
For 1000 kw. at 45,000 volts, wooden poles, single three-phase circuit erected per mile	$1,800 to $2,000

Power plant at Elephant Butte dam—

Three 500 kw. turbo-generator sets, furnishing 2200-volt sixty-cycle current.

Two small turbine driven exciters each of sufficient capacity to handle the three main units.

Four Erie vertical water-tube type boilers.

Steel storage bins, automatic chain grate stokers, coal and ash handling appliances, water purifying and cooling systems.

Cost erected, with house, including overhead charges	$136,500
See *Engineering Record*, Oct. 4, 1913.	
Distribution system (see Eleventh Annual Report U. S. Reclamation Service), about	$22,000

Motors:	
D.c., 35 h.p., 750 r.p.m., 3,200 lb	575
50 h.p., 750 r.p.m., 3,700 lb	750
80 h.p., 750 r.p.m., 6,600 lb	1,050
Induction, 15-h.p., three-phase, sixty-cycle, 440 volts, about	250
Hoisting engines:	
Double drum, electric, 75-h.p., a.-c. motor with another 11-h.p. motor for swinging gear, two speeds by shifting gears, rope speeds on drum 250 ft. and 500 ft. per minute. Weight about 37,000 lb. As installed at Kensico, cost	3,200
Double drum, electric, 35-h.p., d.-c. motor with swinging attachment; prices obtained from four makers in 1908	$1,400 to $1,600
Pile driver, weight 8,000 lb, steam hammer	$700
hoist	1,200
leads	300
Steam shovel, 70 tons	$10,000 to 11,000
Compressors:	
537 cu. ft. free air per minute, weight 17,000 lb., requires 100 h.p., price	1,700
957 cu. ft. free air per minute, weight 22,000 lb., requires 175 h.p. to 200 h.p., price	2,500
Temple electric-pneumatic (see page 156)	1,200
Drills:	
Tripod, 3¼ in., with tripod, 25 ft. of hose, and one set of steel to 16 ft	310
Hammer, largest size, with hose and one set of steel to 6 ft.	115
Electric locomotive, 10 ton, 36-in. gage, 500 volt, 10 miles per hour on level track, 50-ton load, about	2,200
Electric mine locomotive, 24-in. gage, 250 volt, drawbar pull 900 lb., at 10 miles per hour, about	1,800
Crushers:	
Gates No. 7½, weight 63,000 lb., 350 r.p.m., h.p. required 50 to 75, price	$3,100 to 3,200

Gates No. 5, 33,000 lb., 375 r.p.m., h.p. required 25 to 30, price........ $1,500 to 1,600
Blake jaw crusher, 9 in. × 15 in., 250 r.p.m., crushes 15 tons per hour, weight 16,000 lb., about........ 750
Pulverator (for sand) No. 2, 7 tons per hr., h.p. required 40, weight 5,000 lb........ 825
Pulverator No. 4, 20 tons per hour, h.p. required 75, weight 12,000 lb........ 1,400
Davis Standard crushing rolls, 36-in. diameter by 16-in. face, weight 27,000 lb........ $1,800 to 1,900
Bucket elevator, 30 ft. centers, belt or chain, for capacity of 35 tons crushed rock per hour, complete, about........ 400
For each additional foot between centers add about........ 6
Tromel, 40-in. diameter by 12 ft. long, five spiders, punches steel screen, countershaft and drive pulleys, weight 4,500 lb, about........ 400

Concrete mixer:
1 cu. yd. capacity, batch mixer, at 6,000 lb., about........ 600
Dump buckets, 2 cu. yd........ $150 to 200
Skips, 4 cu. yd........ $125 to 150
Hopper bottom dump cars, 4½ cu. yd. of $\frac{3}{16}$-in. steel plate, 36-in. gage, with brakes........ 250

Centrifugal pumps:
10-in, 3,000 gal. per min. at 50-ft. head, h.p. required 65, 865 r.p.m., weight 2,500 lb........ 260
6-in., 1,000 gal. per min. at 50-ft. head, h.p. required 25, 1,300 r.p.m., weight 1,000 lb........ 150

The erection and maintenance of plant, expenditure for supplies, etc., are very variable items. Experience upon one piece of work could only with caution be taken as even an approximate indication of what those items might amount to on other work. It is believed however that even qualified as above the following notes regarding plant and expendible material may be of some interest.

At the Roosevelt dam the main items of plant were as follows:

Air compressor, 700 cu. ft. of free air per minute.
One receiver.
Seven tripod drills.
Twelve plug drills.
Cableways—Three 1,200 ft. long 2¼-in. lock strand.
One 1,000 ft. long 2-in. lock strand.
Two 100 h.p. electric engines.
All lines, sheaves, and accessories.
Anchorages in rock, towers quite small.
Crusher, one No. 7½ o gates.
Fifteen derricks.
Sixteen electric engines, Three air engines } 35 h.p. and 40 h.p.

One motor-generator set, three motors, wiring, transformers, arc lights.
One 1½-cu. yd. and one ¾-cu. yd. mixer.
Two pile drivers, 30 ft. leads, steam hammers.
Two 11-in. Campbell hydraulic elevators.
Four 4-in. auxiliary hydraulic ejectors (pumps).
Three giant nozzles.
Four 4-in., one 6-in., one 8-in., centrifugal pumps.
Two small triplex pumps.
Leschen tramway 1,700 ft. long, two terminals, two short intermediate towers, one intermediate loading station, seventeen 10-cu. ft. buckets, one extra traveling line.

Miscellaneous:	
Thirty 4-cu. yd., four 2-cu. yd., four 1-cu. yd. iron skips, chain, rail, pipe, five flat cars, value about	$7,200
Hose, small tools, rope, jacks, blocks, etc., about	13,000
Blacksmiths' shop and tools, about	750
Eight mules, wagons, harness	4,500
The total cost of the plant, including freight at say an average of $1.50 per 100 lb., in Globe was about	$180,000
Wagon haul to dam site, 40 miles	7,650
Used in erection, lumber, $7,000, cement $700	7,700
	$195,350
Labor of erection, including moving and re-erection during five years, 104,580 hours labor	$34,745
Repairs on equipment during five years (could not always be distinctly separated from re-erection), 67,600 hours labor	28,575
Supplies, oil, iron, steel, repair parts, small hardware, including freight, and about $900 wagon haul	66,800
Blacksmith's coal about 140 tons, including freight and haul	3,570
Explosives, including freight and haul	20,000

Above does not include erection or maintenance of camp, office, or commissary. For magnitude and nature of work see pages 38, 81, 89, 139, 206 to 210, 243 to 246.

Material entering into the work.

The items have been suggested, and the procedure for estimating them has been sufficiently discussed on page 203, obviously prices cannot be given.

Labor cost.—For a general discussion of the subject see pages 203 to 210 and elsewhere throughout the book.

In the specific illustrative labor costs given (see index) it has been attempted so to describe conditions, rates of wages, etc., that they may be of some assistance in forming an estimate.

Cost of Steam Power.—The following is taken from tables prepared by the Canadian Hydro-Electric Power Commission (see their *Fifth Report*) from data published in technical journals and also from data collected by their own engineers regarding existing plants in the district.

STEAM POWER PLANTS, CAPITAL AND ANNUAL COSTS

Size of plant, h.p.	Capital cost of plant per horse-power installed	Annual cost of 10-hour power per brake horse-power	Annual cost of 24-hour power per brake horse-power
Engines: Compound, Corliss, condensing. Boilers: Return tubular with reserve capacity.			
100	$91.40	$33.18	$60.05
200	70.10	28.14	51.72
300	63.90	26.27	48.83
400	59.55	24.84	46.12
500	55.25	23.73	44.21
750	53.30	23.56	44.02
1000	51.00	23.26	43.71
Engines: Compound, Corliss, condensing. Boilers: Water tube with reserve capacity.			
300	73.20	25.77	46.32
400	67.50	24.18	43.61
500	63.40	23.19	42.03
750	59.70	22.88	41.56
1000	56.80	22.47	41.11

Capital cost includes engines, boilers, etc., installed and buildings. Annual costs include interest at 5 per cent., depreciation and repairs on plant, oil, waste, labor, and fuel (coal at $4 per ton). Brake horse-power = mechanical horse-power at engine shaft.

Effect on the cost of steam-power (in previous table) of a variation in the price of coal of $0.50 per ton.

Size of plant, h.p.		10 Hour	24 Hour
200	Compound condensing	$1.69	$3.71
300	Compound condensing	1.62	3.60
400	Compound condensing	1.56	3.44
500	Compound condensing, water-tube boilers	1.39	3.05
750	Compound condensing, water-tube boilers	1.39	3.05
1000	Compound condensing, water-tube boilers	1.39	3.05

It may be noted that while another authority agrees closely with the foregoing figures, two others set the annual cost of 10-hour power in a 300 h.p. plant 50 per cent. higher. It is probable that as usually installed upon construction work the capital cost might be somewhat less owing to the more temporary character of buildings, chimneys, etc.; also that the annual cost might be somewhat higher.

FIG. A.—Shoshone dam.

FIG. B.—Shoshone dam.

(Facing Page 224.)

PLATE XVI

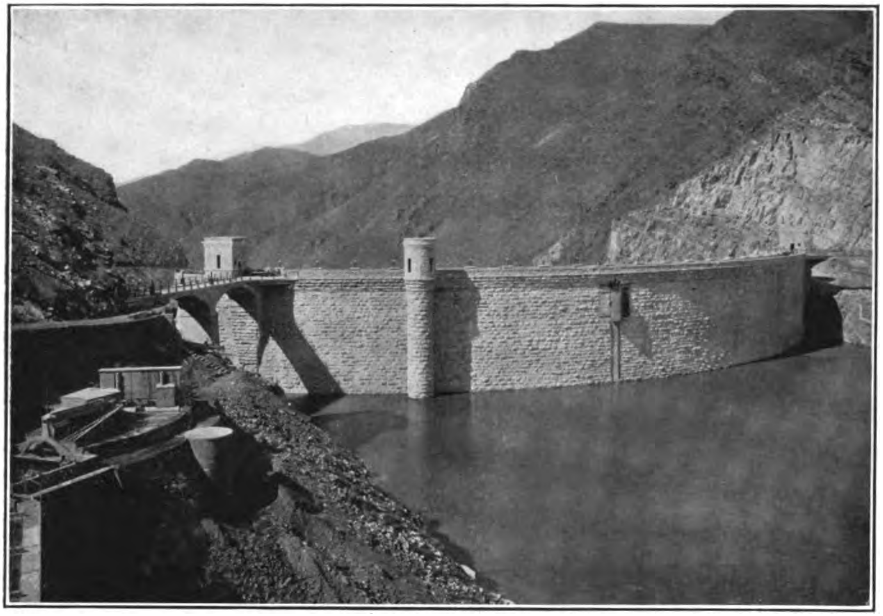


FIG. C.—Roosevelt dam.

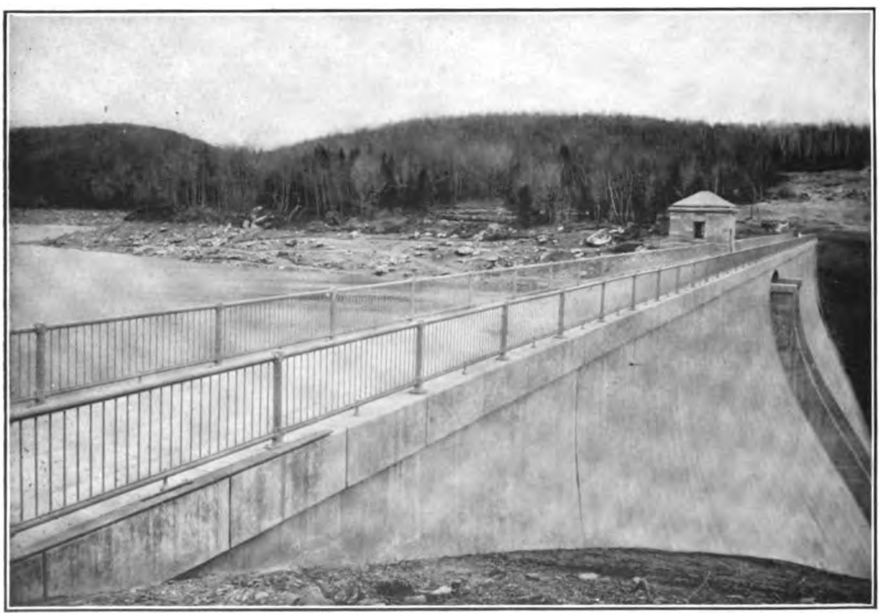


FIG. D.—Farnham dam.

CHAPTER XV

PARTIAL LIST OF EXISTING DAMS WITH DESCRIPTIONS AND COSTS

Sodom Dam (Fig. 43).

This is said to have been the first dam upon the construction of which a cableway was employed. A 2-in. cable weighing 7 lb. per

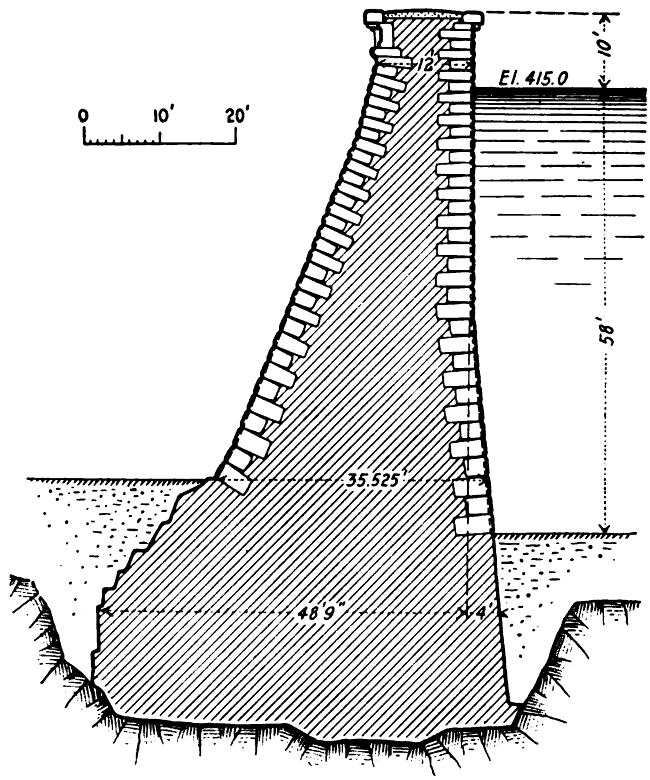


FIG. 43.—Section of Sodom dam, New York; built 1888 to 1893 of rubble masonry; length, 500 ft.; max. height, 98 ft.; contains 35,887 cu. yd.

ft. was stretched between towers 667 ft. apart. It was at first given a sag of 20 ft. which increased 3 ft. to 5 ft. under load. It parted under a load of 6 tons while in service and a new cable was installed; the towers being raised 10 ft. to permit a greater sag. (As at present installed cableways are given a sag equal to 5 per cent. of the span.) Cost of cableway installed was $3750.

For river diversion a crib dam was built across the river 80 ft. upstream from the masonry. A canal 26 ft. wide × about 15 ft. deep was built along one hillside returning to the river about 500 ft. below the dam. Maximum flood during the construction period 4166 cu. ft. sec.

Quantities of principal items, and the contract prices:

Earth excavation	5,986 cu. yd.	$0.35
Rock excavation	12,260 cu. yd.	1.50
Deep rock excavation	3,600 cu. yd.	2.00
Rubble masonry in 1–2 American cement mortar	300 cu. yd.	3.75
Rubble masonry in 1–2 Portland cement mortar	23,280 cu. yd.	4.50
Rubble masonry in 1–3 Portland cement mortar	6,260 cu. yd.	4.25
Brick masonry	530 cu. yd.	10.25
Granite dimension, stone masonry	776 cu. yd.	35.73
Facing stone masonry	4,287 cu. yd.	10.75

Sand pit 1 mile from dam, quarry 1½ miles from dam.

Maximum progress on masonry 3000 cu. yd for a month, with twelve masons and three derricks. Average progress 1700 cu. yd.

Total cost of dam, $366,499.

Costs to contractor:

- Rubble stone $1.97 per cu. yd. including 5 cents royalty.
- Facing stone $9.75 per cu. yd. including 15 cents royalty.
- Facing stone set per cu. yd., $10.97.
- Dimension masonry (including dressing) $30.08 per cu. yd.
- Cement in shed at dam, American $1.00½, Portland $2.31½ and $2.51½.
- Laborers paid $1.25 and stone masons $3.50 per day.
- Double teams hauled stone, 1 to 1½ cu. yd. per trip, six to eight trips per day.

Titicus Dam (Fig. 44).

The quantities of the principal items of work and the contract prices were as follows:

Earth excavation and disposal in embankments, 500,000 cu. yd at	$0.25
Other items of earth excavation, 6000 cu. yd. at	$0.25 to 0.40
Rock excavation, 33,000 cu. yd. at	2.25
Permanent timber work, 175 M ft. B.M. at	$30 and 32
Concrete, 1500 cu. yd. at	$4 and 4.50
Brick masonry 500 cu. yd. at	12.00
Rubble masonry in 1–2 American cement mortar, 140,000 cu. yd. at	3.40

Additional price for substituting in above:	
1–1 American cement mortar	0.70
1–1 Portland cement mortar	2.50
1–2 Portland cement mortar	1.75
1–3 Portland cement mortar	1.00
Granite dimension stone masonry, 1100 cu. yd. at	20.00
Facing stone masonry, 5000 cu. yd. at	12.00
About 200 tons of pipe and special castings at	$38.00 and 100.00
Other items as face work of masonry, paving, rip-rap, valves, etc. brought total to	$933,065

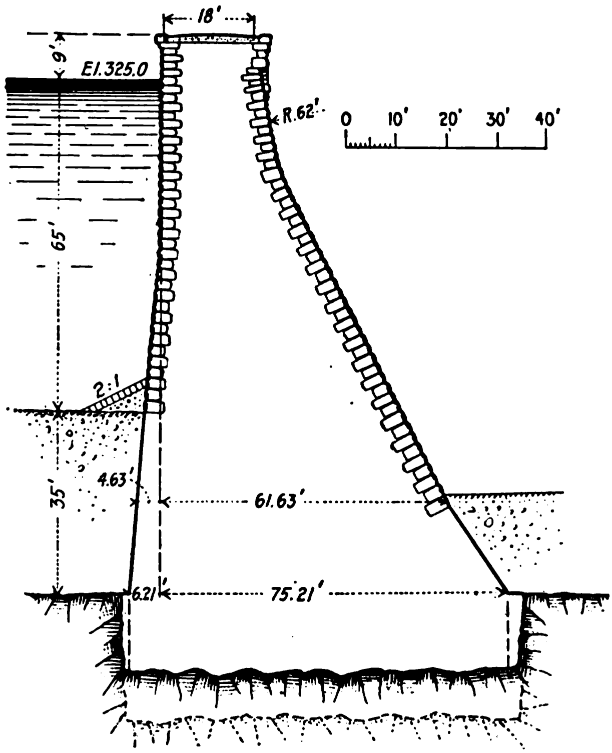


FIG. 44.—Section of Titicus dam, New York; built 1890 to 1895; length, 534 ft.; max. height, 109 ft.; contains about 149,000 cu. yd.; contract price, $933,065.

The face masonry was laid with beds perpendicular to face. Six derricks and thirty-six masons averaged 3240 cu. yd. per month. Maximum progress 5700 cu. yd. per month.

New Croton Dam (Fig. 45, also see Fig. 17).

As first conceived and designed this dam was known as the Quaker

Bridge dam. Opposition to the first proposed location caused it to be moved upstream $1\frac{1}{8}$ miles where it was first known as the Cornell dam, changed later to the New Croton. In both design and construction this dam was a notable advance upon anything previously undertaken and there was very little precedent for many of the problems involved. At the time this was undertaken cableways, self-swinging derricks, as well as other features of plant had

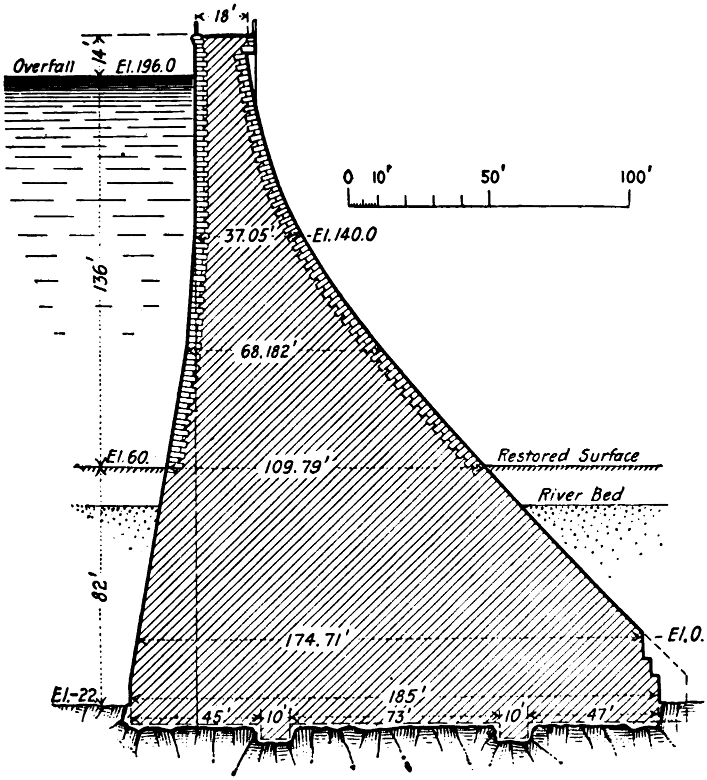


FIG. 45.—Section of New Croton dam, New York; built 1892 to 1906; part rubble masonry; part cyclopean masonry; length, 1200 ft.; max. height, 238 ft.; contains 855,000 cu. yd.

not been developed and were not appreciated as at present. During the early stages of the work several assignments of contract and reorganizations of forces operated to load the job up with much ill-considered or unnecessary plant. The dam as originally designed and largely built consisted of a masonry wall about 500 ft. long in the center of the valley, to be continued on the north side by a masonry waste weir, and on the south side by an earth dam having a masonry core wall founded on rock. The maximum height of the earth dam was to be 120 ft. above the original surface. When these plans were well along toward completion they were reconsidered, and finally 110 ft. of the earth and core wall section was replaced by extending the original masonry section, thus reducing the maximum height of the earth dam to 50 ft. The delay caused by the reconsideration and change amounted to two or three years.

The principal items of the original contract and the prices of the accepted bid were as follows:

Earth excavation, 550,000 cu. yd. at	$0.61
Earth excavation, 35,000 cu. yd. at	0.95
Rock excavation, 300,000 cu. yd. at	1.95
Refill and embankment, 900,000 cu. yd. at	0.25
Permanent timber work, 320 M. ft. B.M	$40 and $45
Crib work, 8000 cu. yd. at	3.00
Rubble masonry (American cement 2–1), 470,000 cu. yd. at	4.05
Rubble masonry (Portland cement 2–1), 30,000 cu. yd. at	5.346
Rubble masonry (Portland cement 3–1), 50,000 cu. yd. at	4.941
Dry rubble, 12,000 cu. yd. at	2.50
Rip-rap, 5000 cu. yd. at	1.75
Broken stone, 10,000 cu. yd. at	1.50
Block stone, 2500 cu. yd. at	15.00
Dimensions stone masonry, 2500 cu. yd. at	35.00
Facing stone masonry, 24,000 cu. yd. at	13.50
Brick masonry, 2000 cu. yd. at	10.00
Concrete Portland cement, 3000 cu. yd. at	6.006
Concrete American cement, 10,000 cu. yd. at	4.55
Miscellaneous items, face dressing, sodding, etc., makes total	$4,150,573

Owing to the many changes made in the plans the actual cost of the dam, not including engineering, land, and legal expenses, was $6,886,872.

The quantities of the main items involved in the dam as finally built were

Earth excavation	1,821,400 cu. yd.
Rock excavation	400,250 cu. yd.
Refill	900,000 cu. yd.
Masonry, all kinds	855,000 cu. yd.

The principal items of plant, and their value, have been given as follows, without, however, any statement as to whether value included freight and erection.

56	Hoisting engines, 15 to 25 h.p.	$33,600
3	10-ton, 1400 ft. cableways, 75-h.p. engines	30,000
10	10–20 ton locomotives	30,000
75	5–7 ton derricks	75,000
8	Pumps	8,000
5	Rock drills	4,500
2	Pile drivers, 3000-lb. hammers	1,000
8	Concrete mixers, 3½ and 5 cu. yd.	8,000
3	Stone crusher	7,000
3	Electric light dynamos	2,000
80	Flat cars	12,000
100	Dump cars	10,000
750	Tons steel rails	30,000
36	Boilers, 10 to 100 h.p.	14,000
3	Steam shovels	15,000
1	Dredge	5,000
75	Teams, with wagons, carts, rollers, scrapers	40,000
27	Steam engines, 10 to 60 h.p.	5,000
	Machine shop	7,000
	Saw mill	1,000
	Sand and gravel screening plant	1,000
8	Derrick swing attachments	1,200
6	Steam derrick swings	1,800
50	Dump buckets	3,000
6	3-yd. skips	600
200	2-yd. skips	4,000
	Duplicate parts for repairs	5,000
	Miscellaneous tools and equipment	20,000
		$374,700

The cost of laying rubble masonry during 1898 and 1899 when average conditions prevailed, and ten hours constituted a day's work, was about as follows:

Quarrying	$0.841	Based on labor prices	Quarrymen	$1.50–$1.75
Sand pits	0.073		Masons	3.00
Cleaning stones	0.079		Hoisting engineers	2.00
Mortar[1]	0.646		Stone cutters	3.00
Masonry	1.118		Labor	1.50
Rigging	0.046			
Pumping	0.103			
Plant	0.200			
Insurance	0.021			
Office	0.081			
Total	$3.208			

[1] For 1–2 American cement mortar; for 1–2 Portland cement mortar substitute $1.71 and for 1–3 Portland cement mortar substitute $1.357. For further cost figures see pages 34–36, 80, 92, 142.

The maximum force employed was 475 men on the dam, and 376 men in the quarry. The largest month's work on rubble masonry was during June, 1898, when 17,186 cu. yd. were laid. On cyclopean masonry the largest month's work was during August, 1904, 17,000 cu. yd.

Acknowledgment is made of the principal sources of information

"The Design and Construction of Dams," by Edward Wegmann.

"The Construction of the New Croton Dam," by Edward Wegmann and J. B. Goldsborough in the *Journal* of the American Society of Engineering Contractors, November, 1910.

The Foundations of the New Croton Dam, by Chas. S. Gowen, *Transactions* American Society Civil Engineers, June, 1900.

The Changes at the New Croton Dam, by Chas. S. Gowen, *Transactions* American Society Civil Engineers, June, 1906.

Various issues of *Engineering News*, *Engineering Record*, *Engineering Contracting*.

Cross River Dam (Fig. 46).

In addition to the dam, the contract included a spur track connection with a railroad and 5½ miles of highways. The prices bid for the principal items were as follows:

Earth excavation, 65,000 cu. yd., class A at	$0.80
Earth excavation, 3000 cu. yd., class B at	1.50
Earth excavation, 120,000 cu. yd., class C at	0.45
Rock excavation, 25,000 cu. yd., class A at	1.75
Rock excavation, 5000 cu. yd., class B at	5.00
Rock excavation, 25,000 cu. yd., class C at	1.75
Rock excavation, 4000 cu. yd., class D at	1.75
Refill and embankment, two classes at	$0.20 and $0.40
Portland cement, 135,000 barrels at	1.30
Concrete face blocks, 17,500 cu. yd. at	12.00
Monolithic concrete, 6000 cu. yd. at	6.50
Cyclopean concrete, 132,000 cu. yd. at	2.90
River control works (lump sum)	30,000.00
Superstructure of gate house (lump sum)	8,000.00
Macadam highway surfacing (12 ft. wide), 8000 lin. ft at	1.40
Gravel highway surfacing, 20,000 lin. ft. at	0.25
Fence and guard rail, 47,000 lin. ft. at	0.25
Stone boundary wall, 90,000 lin. ft. at	0.80

Miscellaneous smaller items such as granolithic, face dressing, paving, rip-rap, special castings, valves, sluice gates, screen and stop-plank guides, railing, drain pipe, grassing, clearing reservoir, etc., made a total for the bid of $1,246,212.

The actual total cost, owing to various departures from the estimated quantities, was $1,389,210.94. Cost of engineering was $81,787.49. Cost of inspection was $45,031.25.

Maximum quantity of masonry laid, 18,430 cu. yd. per month = 864 cu. yd. per day.

Maximum quantity of cyclopean masonry per day 823 cu. yd. by six gangs of eleven men each.

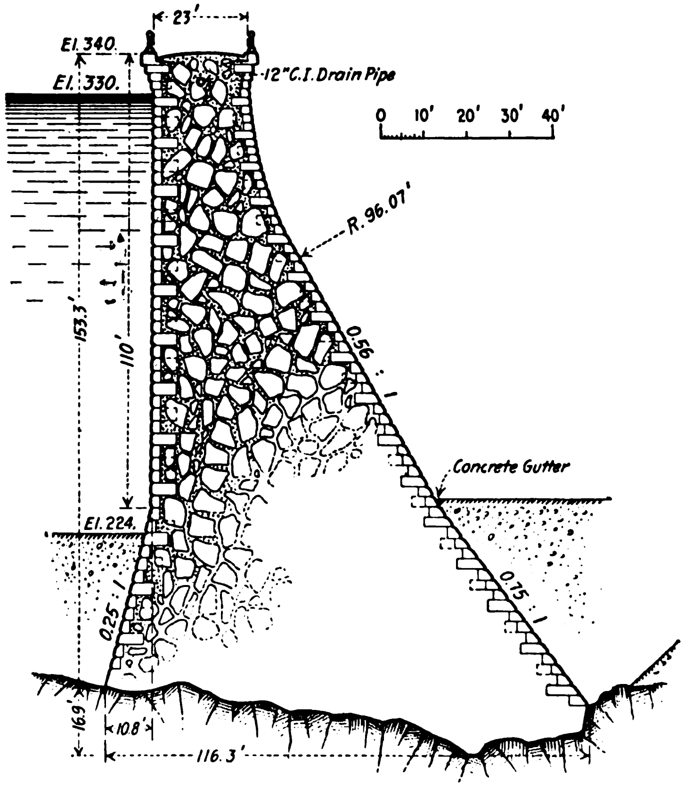


FIG. 46.—Section of Cross River dam, New York; built 1905 to 1907; cyclopean masonry; concrete block face; length of top 986.5 ft.; max. height above foundation 170 ft.; contains 158,467 cu. yd.; masonry all kinds.

Average quantity of cyclopean masonry for entire work laid by gang of eleven men = 105 cu. yd. per ten-hour day.

The distribution of the cost between the features of dam, spur track, and highways is indicated by the following division of principal items.

Dam	Spur Track	Highways
	7480 lin. ft. standard gage, length of trestle 642 lin. ft., average height of trestle 21 ft., max. cut 40 ft., max. fill 40 ft.	1.43 miles of 12-ft. macadam cost $40,225 per mile, including one 40-ft. arch bridge, one 8-ft. arch, small culverts, catch basins, paved slopes, etc. The 40-ft. arch cost $8985.83. Dirt roads (4.06 miles) cost $28,031 per mile, including one 18-ft. and one 8-ft. arch, small culverts, catch basins, paved slopes, etc.
Earth excavation, 89,403 cu. yd.	69,052 cu. yd.	126,795 cu. yd.
Rock excavation, 40,016 cu. yd.	954 cu. yd.	23,287 cu. yd.
Refill and embankment, 71,138 cu. yd.	....	715 cu. yd.
Cyclopean masonry, 132,254 cu. yd.	....	....
Concrete blocks, 17,655 cu. yd.	....	....
Monolithic concrete, 7294 cu. yd	....	2,354 cu. yd.
Portland cement, barrels, 134,630	....	2,840
Fence and guard rail	....	40,145 lin. ft.
Stone boundary wall	....	88,268 lin. ft.

Croton Falls Dam (Fig. 47).

The entire contract included the construction of the main dam; a diverting dam on another stream; an open channel 3522 ft. long connecting the reservoirs formed by the two dams; 2½ miles of standard gage railroad, 14½ miles of new highways; fourteen large concrete highway bridges; 21 miles of stone wall boundary fencing; 21½ miles of wooden guard rail fencing; clearing 1600 acres of reservoir; 2 miles of 15-in. twin-tile pipe sewer.

Following are the principal items of the contract, and the prices at the accepted bid. Total amount of bid $3,028,853.

River control works, lump sum	$90,000.00
Earth excavation, class A, 490,180 cu. yd. at	0.75
Earth excavation, class B, 11,675 cu. yd. at	1.65
Earth excavation, class C, 130,000 cu. yd. at	0.35
Rock excavation, class A, 202,025 cu. yd. at	1.50
Rock excavation, class B, 13,800 cu. yd. at	4.00

Rock excavation, class C, 35,850 cu. yd. at	1.25
Refill and embankment, A, 203,500 cu. yd. at	0.40
Refill and embankment, B, 87,620 cu. yd. at	0.20
Portland cement, 270,000 barrels, at	1.75
Cyclopean masonry, 262,000 cu. yd. at	2.65
Concrete face blocks, 28,200 cu. yd. at	8.50
Monolithic concrete masonry, A, 3000 cu yd. at	7.00

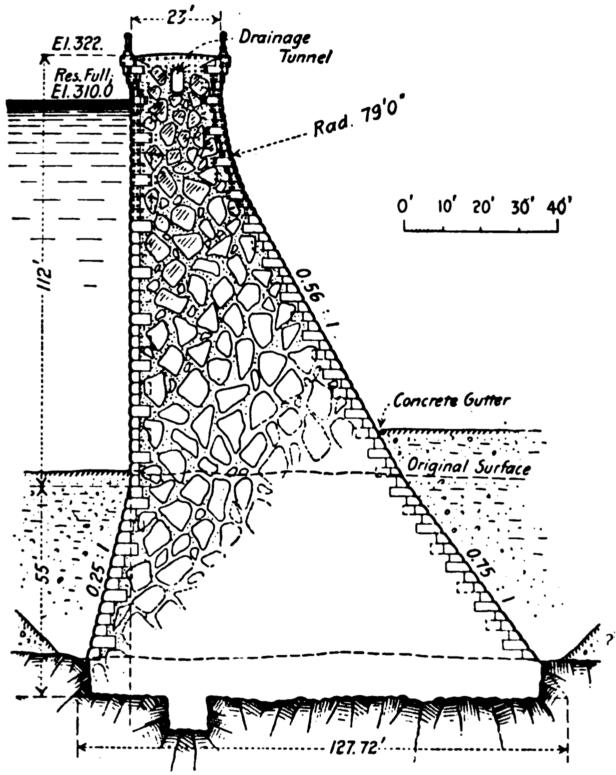


FIG. 47.—Section of Croton Falls dam, New York; built 1906 to 1911; cyclopean masonry; concrete block faces; 1100 ft. long; max. height 173 ft.; contains 290,540 cu. yd. masonry all kinds.

Monolithic concrete masonry, B, 13,600 cu. yd. at	5.50
Block stone, masonry, 3000 cu. yd. at	30.00
Dimension stone masonry, 1500 cu. yd. at	55.00
Various items: Granolithic, grouting, face dressing, paving, rip-rap, total value	86,875.00
Various items of metal work: Valves, gates, cast iron-pipe, special castings, railings, guides, etc., total value	60,340.00
Gate house superstructures, stop planks, gratings, etc.	13,000.00
Highway surfacing, sewers, fencing, wall, etc	136,843.00

As constructed, the principal items were distributed among the various features as follows:

Main dam:	
Earth excavation	280,000 cu. yd.
Rock excavation	95,000 cu. yd.
Refill	180,000 cu. yd.
Cyclopean masonry	245,000 cu. yd.
Concrete face blocks	30,000 cu. yd.
Monolithic concrete	15,000 cu. yd.
Dimension stone	540 cu. yd.
Reinforcing steel	575 tons
Portland cement	230,000 barrels

The temporary channel for handling the water took out of the stream 600 ft. above the dam and returned to it 800 ft. below. Of this 1400 ft. total length, 800 ft. was timber flume 24 ft. wide × 8 ft. 2 in. deep.

Diverting dam, 1185 ft. long	Earth with core wall
Earth excavation	108,090 cu. yd.
Rock excavation	56,234 cu. yd.
Refill and embankment	252,000 cu. yd.
Cyclopean masonry	37,053 cu. yd.
Portland cement	48,384 barrels
Dimension stone	776 cu. yd.
Paving	21,904 cu. yd.
Connecting channel, 3522 ft. long { In rock, 24 ft. wide, ½ to 1 slope. In earth 15 ft. wide, 1½ to 1 slope.	
Earth excavation	306,392 cu. yd.
Rock excavation	43,093 cu. yd.
Refill	23,246 cu. yd.
Concrete masonry	4,164 cu. yd.
Dry paving	27,717 cu. yd.
Portland cement	4,403 barrels
Highways 14½ miles { In cut, 22 ft. roadway On fill 28 ft. roadway	
Earth excavation	798,000 cu. yd.
Rock excavation	69,000 cu. yd.
Concrete and rubble masonry	20,000 cu. yd.
Dry paving	35,600 cu. yd.
Portland cement	22,000 barrels
Surfacing	80,000 lin. ft.
Post and rail fencing	110,000 lin. ft.
Total plant cost including installation and renewals, minus salvage, about	$425,000
Total value of work done under the contract, about	4,250,000

Maximum progress on masonry in October, 1908.

27 eight-hour days, eight gangs of twelve men (less four gangs off one-half day), laid 24,266 cu. yd. cyclopean masonry.

30 eight-hour days, two gangs of six men, laid 720 face blocks = 1666 cu. yd.

Common labor was paid, \$1.50, \$1.75 and \$2.
Masons and carpenters were paid, \$2.50 to \$4.
Foremen and engineers were paid, \$3 and \$3.50.
Eight-hour day rigidly enforced.
From paper by Frederick S. Cook in the *Proceedings* of the Municipal Engineers of the City of New York.

Olive Bridge Dam (Fig. 48).

The Olive Bridge dam is a portion of a system of dams and dikes retaining the water in Ashokan Reservoir. (See page 143 for brief

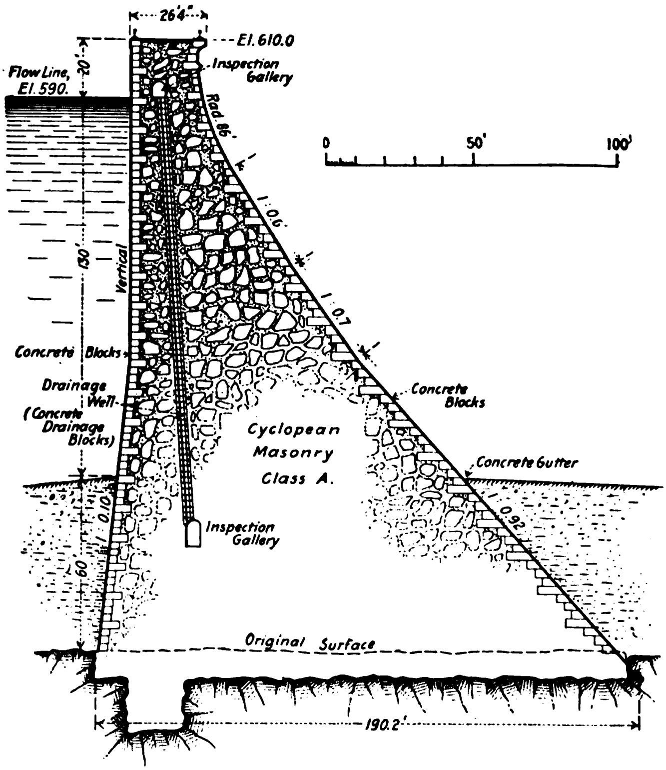


FIG. 48.—Section of Olive Bridge dam, New York; built 1908 to 1913; cyclopean masonry; concrete block faces; length 1,000 ft.; max. height 251 ft.; contains 488,200 cu. yd.

description.) Olive Bridge dam itself consists of three sections. The central section of masonry dam proper, is 1000 ft. long. At the south end the masonry section merges, in a distance of 170 ft., into a corewall section, and continues as a corewall and embankment dike for a total length of 1700 ft. Similarly at the north end, the masonry second merges in a distance of 166 ft. into a corewall and embankment section of a total length of 2100 ft. Total length of the three sections 4800 ft. The quantities of the main items for each of the three sections, also the contract prices are given below.

	South wing	Masonry dam	North wing
Earth excavation, class A at $1.40 per cu. yd...	$4,000	....	$20,000
Earth excavation, class B at $2.50 per cu. yd...	4,000	....	16,000
Earth excavation, class C at $0.68 per cu. yd...	....	150,000	....
Earth excavation, class D at $0.50 per cu. yd...	20,000	....	40,000
Rock excavation, class A at $3.00 per cu. yd....	5,000	95,000	....
Embankment and refilling various classes at $0.50 and $0.60.	630,000	110,000	1,270,000
Concrete masonry at $4.90 per cu. yd..........	25,000	....	35,000
Cyclopean masonry, class A at $3.40 per cu. yd.	....	432,000[1]	....
Concrete blocks at $11.50 per cu. yd..........	....	56,200	....
Paving at $2.50 per cu. yd....................	8,000	....	12,000
Surface dressing, soil at $0.50 per cu. yd........	20,000	....	40,00.
River control, lump sum.....................	....	10,000	0

The above prices did not include cement; the contract price of Portland cement was $1.50 per barrel.
Concrete blocks required 1.4 barrels of cement per cu. yd.
The 432,000 cu. yd. of cyclopean masonry required 449,000 barrels.
Cyclopean masonry = 25.3 per cent. stone, balance concrete.
Other items pertaining to the main dam were

Masonry filling of stream flow opening, 8000 cu. yd. at......	$1.50
Cast-iron pipes and special castings, 75 tons at.............	101.00
Steel castings, 80 tons at................................	150.00
Reinforcing steel, 25,000 lb. at...........................	0.07
Wrought iron, cast iron and steel, 590,000 lb. at............	0.08
Bronze work, 4,000 lb. at.................................	0.50
Small pipe, 2500 lin. ft. at...............................	0.50
Setting all metal work, 900,000 lb. at......................	0.02

The plant employed on the work is described on pages 143 and 144.

Kensico Dam (see Figs. 22, 23, 32).

Of cyclopean masonry, containing expansion joints and drainage system, with upstream face of concrete blocks and downstream face of stone.

[1] Includes some concrete.

The contract includes some work on the reservoir, adjacent highways, bridges, etc.

The quantities of the principal items of work, and the contract prices are as follows:

Stream control (lump sum)	$35,000.00
Excavation, class A, 230,000 cu. yd. at	0.49
Excavation, class B, 650,000 cu. yd. at	0.50
Excavation, class C, 380,000 cu. yd. at	1.40
Excavation, class D, 80,000 cu. yd. at	2.00
Excavation, rock, blasting not permitted, 40,000 cu. yd. at	1.50
(Except for the last item the excavation is classified by location, and not as earth and rock.)	
Special preparation of rock surface, 40,000 sq. yd. at	0.50
Refill:	
4-in. layers, 30,000 cu. yd. at	0.50
6-in. layers, 130,000 cu. yd. at	0.65
12-in. layers, 230,000 cu. yd. at	0.20
Covering reservoir bottom, 300,000 cu. yd. at	0.55
Deposited in bulk, 650,000 cu. yd. at	0.25
Surface dressing, 100,000 cu. yd. at	0.50
Portland cement, 900,000 bbl. at	1.50
Mass concrete, A, 2500 cu. yd. at	5.75
Mass concrete, B, 35,000 cu. yd. at	5.25
Cyclopean masonry, 900,000 cu. yd. at	2.65
Facing above, 40,000 sq. yd. at	0.15
Concrete blocks, 60,000 cu. yd. at	7.50
Reinforced concrete, A, 2500 cu. yd. at	12.50
Reinforced concrete, B, 8500 cu. yd. at	9.00
Grout, 3000 cu. yd. at	5.00
Tool dressing concrete, 75,000 sq. ft. at	0.20
Dimension stone masonry, 14,700 cu. yd. at	$23.05 to $24.54
Face dressing, stone:	
Quarry face, 15,000 sq. ft. at	0.13
Bull point, 80,000 sq. ft. at	0.25
Rough point, 4000 sq. ft. at	0.25
Fine point, 160,000 sq. ft. at	0.30
4 cut, 150,000 sq. ft. at	0.55
6 or 8 cut, 1000 sq. ft. at	1.00
Straight cast-iron pipe, 350 tons at	60.00
Special cast-iron pipe, 70 tons at	100.00
Stone wall, 110,000 lin. ft. at	0.80
Wooden fence and guard rail 25,000 lin. ft. at	0.30
Wire fence, 55,000 lin. ft. at	0.50
Macadam, 15,000 cu. yd. at	3.00
Also numerous small items.	
Total amount of bid	$7,953,050.00

In the light of some revision of quantities, and separating those quantities that pertain to the reservoir and the highways, the dam proper will require

Excavation	1,200,000 cu. yd.
Embankment	2,000,000 cu. yd.
Cyclopean masonry	800,000 cu. yd.
Concrete blocks	60,000 cu. yd.
Dimension stone masonry	21,000 cu. yd.
Roughly squared stone masonry	8,000 cu. yd.
Mass concrete	42,000 cu. yd.
Reinforced concrete	11,000 cu. yd.
Portland cement	900,000 barrels
Pipe, valves and other metal work, value	$180,000

Numerous small unlisted items, make the estimated total cost of the dam, including waste weir, waste channel and conduit, drainage system, and grading of grounds, about $6,735,000.

For further information regarding the work and methods, see pages 37, 77, 91, 107, 108, 144, 163.

For illustrated description of crushing plant see *Engineering News*, February 22, 1912.

The maximum height of 300 ft. is from a point 130 ft. below stream bed and applies to a very short distance. General section is 250 ft. high.

At 300 ft. below crest the thickness is 230 ft.

Boonton Dam (Fig. 49).

A preliminary organization acquired right-of-way, land for quarry, built quarry railroad, trestles along the dam, cofferdams and office buildings, installed pumps and considerable plant, did about 75 per cent. of the excavation, quarried a large quantity of stone and cut about 80 per cent. of the Ashlar.

Estimated value of the work, about $498,600.

A contract was then let for

Earth excavation	45,400 cu. yd.
Rock excavation	15,300 cu. yd.
Embankment	108,000 cu. yd.
Slope paving	7,300 cu. yd.
Cyclopean masonry, at $1.98	229,850 cu. yd.
Concrete masonry	5,831 cu. yd.
Ashlar and dimension stone	19,646 cu. yd.

At contract price of $663,700, which about represented the cost to the contractor; various allowances increased the amount finally paid the contractor, to	$720,000
Portland cement (not included above) 176,000 barrels	281,400
Total not including engineering, about	$1,500,000

Maximum progress 21,000 cu. yd. per month.

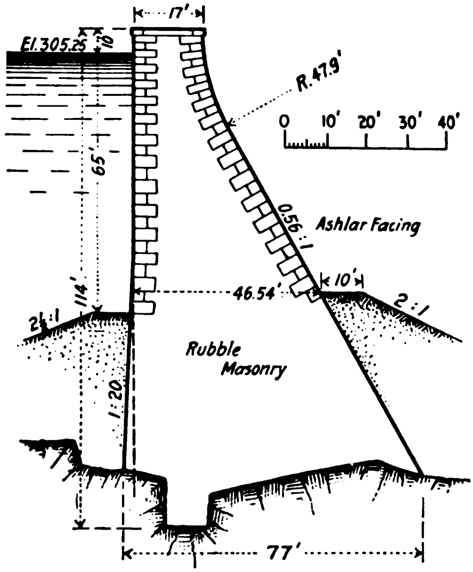


FIG. 49.—Section of Boonton dam, New Jersey; built 1900 to 1906; cyclopean upstream face rubble; downstream face ashlar; 2150 ft. long; max. height 103 ft.; contains 255,327 cu. yd.; masonry all kinds.

Wachusett Dam (Fig. 50).

Much preliminary work was done by forces employed directly by the Metropolitan Water Board. This included preliminary surveys and borings in 1893–94, supplemented by many more borings in the latter part of 1895 and during 1896.

In 1897 the Board constructed the upper temporary cofferdam, the small flume, and about 200 ft. in length of the large flume. During 1899 a contract was let for a small amount of earth and rock excavation, and the Water Board directly constructed the remainder of the large flume. During 1900 considerable earth excavation was done and the lower temporary dam was built by Metropolitan Water Board forces while plans and specifications were being drawn up for the main contract. The main contract was awarded early in October, 1900, and the illustration Plate IV, Fig. C shows the temporary works and the condition of the pit at that time.

Following are the principal quantities and prices of the main contract as per the final estimate.

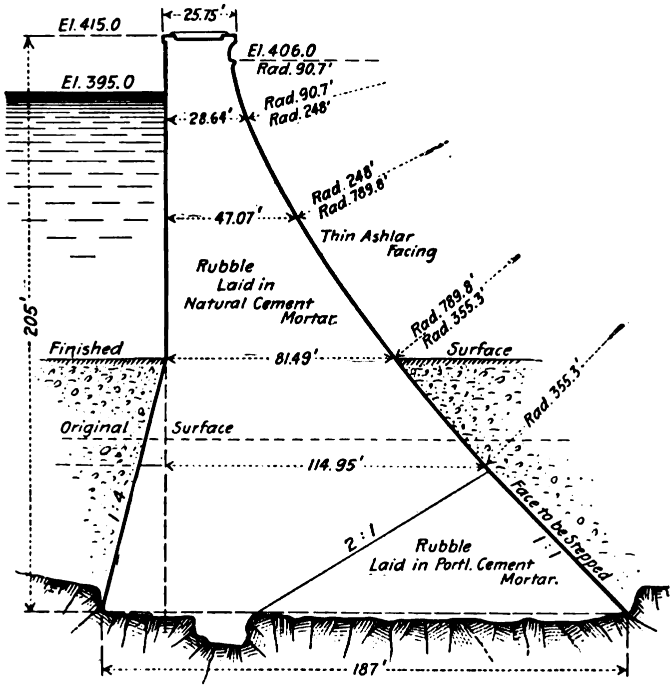


FIG. 50.—Section of Wachusett dam, Mass.; built 1900 to 1906; rubble masonry cut-stone faces; length: main dam, 971 ft., overflow Weir, 452 ft., core wall 53 ft., total 1476 ft.; max. height 207 ft.; from the lowest point of cut-off trench 227 ft., contains 266,663 cu. yd. of masonry all kinds.

Earth excavation, ten classes, aggregating 274,087 cu. yd. at prices ranging from 25 cents to \$1, total	\$122,369.30
Rock excavation, eight classes, aggregating 102,640 cu. yd. at prices ranging from 90 cents to \$4, total	116,709.05
Rubble masonry:	
1–2 Natural cement mortar, 179,755 cu. yd. at \$3.80	683,069.00
1–2 Portland cement mortar, 31,911 cu. yd. at \$4.80	153,172.80
1–2½ Portland cement mortar, 8927 cu. yd. at \$4.55	40,617.85
1–3 Portland cement mortar, 28,333 cu. yd., at \$4.20	118,998.60
1–1 Natural cement mortar, 2994 cu. yd. at \$4.00	11,976.00
Carried forward	\$1,246,912.60

Brought forward	$1,246,912.60
Ashlar masonry:	
3411.4 cu. yd. at $12.00	40,936.80
4556.8 cu. yd. at 12.50	55,710.00
1168.6 cu. yd. at 13.50	15,776.10
Dimension stone masonry:	
1,776.00 cu. yd. at $16.50	29,304.00
965.63 cu. yd. at 20.00	19,312.60
Brick masonry:	
794.0 cu. yd. at $15.00	11,910.00
270.5 cu. yd. at 14.00	3,787.00
Concrete masonry:	
1–2½–4½ Portland, 7208.2 cu. yd. at $6.50	46,853.30
1–3–6 Portland, 1313.6 cu. yd. at $5.50	7,224.80
1–2–5 Natural, 1134 cu. yd. at $4.35	4,932.90
1–3–6 Natural, 19 cu. yd. at $3.75	71.25
Slope paving:	
1 130 cu. yd. at $6.00	6,780.00
769 cu. yd. at 3.25	2,499.25
Hauling and setting metal work, 893.6 tons at $14.00	12,510.40
Various items—face dressing, granolithic surfacing, roadways and paths, furnishing and laying vitrified pipe drains, drilling holes on side lines of cut-off trench (see page 44).	
Extra and special items of work and allowances	193,491.27
Total	$1,698,012.27
Other contracts were:	
Preliminary excavation in 1899	19,120.00
Special castings	16,286.65
Valves and gates	29,917.67
Bronze grooves for stop planks and screens	17,363.00
Miscellaneous	23,899.66
Total of contracts	$1,804,599.25
Summary of cost of Wachusett dam:	
Engineering	$241,697.56
Preliminary (1)	24,518.93
Temporary works	120,173.53
Additional (2)	114,279.44
Contracts	1,804,599.25
Power and gate house	72,937.34
Total	$2,378,206.05

(1) Borings. (2) Excavation done by Metropolitan Water Board.

The masonry prices include the cost of cement; cement cost delivered, Portland about \$1.55 and American about \$0.70 per barrel.

Sand was obtained from a pit on land of the Commonwealth and hauled $\frac{3}{8}$ mile by teams to the dam.

For further description of various features of the work see page 23, Borings; page 33, Temporary works; pages 140 and 160, Plant and power.

Farnham Dam (see Plate XVI, Fig. D).

Built for the water supply of Pittsfield, Mass. $2\frac{1}{2}$ miles from railroad.

Cyclopean masonry with concrete block faces. The gravity section is about 600 ft. long; this is extended to 900 ft. total length, by corewall and embankment section at each end.

The quantities and prices of the principal items were as follows:

Earth excavation, 15,245 cu. yd. at 50¢ and 90¢ according to elevation.

Rock excavation, 8481 cu. yd. { at \$2.00 where blasting was permitted. / at \$4.00 where blasting was prohibited.

Mass concrete, 2254 cu. yd. at \$7.00, \$7.50, and \$8.00 according to mixture.
Cyclopean masonry, 38,008 cu. yd. at \$5.00, \$5.50, and \$6.00.
Concrete blocks, 4844 cu. yd. at \$11.50, \$12,00, and \$12.50.
Gate house superstructure, \$1500.
Total cost of dam including engineering, approximately, \$355,000.

Regarding expansion joints and leakage see page 117. Plums in the cyclopean masonry amounted to about 30 per cent.

Roosevelt Dam (Fig. 51, see also Frontispiece).

The dam is situated on the Salt river 75 miles E. N. E. of Phoenix, 40 miles from the railroad at Globe and 60 miles from the railroad at Mesa. A highway was built down the valley of the Salt river to the latter city and most of the freight was hauled over that route. The lowest bid received from cement manufacturers for cement delivered at the dam site was \$4.89 per barrel. The Reclamation Service found cement materials at the dam site and there built and operated a cement mill with a capacity of about 400 barrels per twenty-four hours. This mill furnished all the cement for the dam, and a large quantity for use on other parts of the project.

Cost of manufacturing 107,589 barrels of cement in 1908

	Cost per bbl.
Office salaries, superintendent, chemist, etc.	$0.049
Operating labor	0.292
Maintenance labor	0.055
Maintenance material	0.085
Clay digging	0.054
Clay hauling	0.078
Limestone quarry	0.121
Supplies	0.051
Fuel wood for drying clay	0.067
Fuel oil	0.922
Power	0.122
Depreciation about	0.80
Total	$2.696

Owing to the many interruptions to the work, and to limited storage capacity, the average operating costs for the entire period were higher. The cement mill, machinery, installation, complete cost $249,447.92. Owing to the remote location, length of service, cost of hauling and handling the salvage was very small, not over 5 per cent. The total cost of operation, for the entire period of about five years and three months, including depreciation of plant, was $1,063,542.36. The total output of 338,452 barrels cost $3.14 per barrel.

On January 10, 1905, a contract was let for 50,000 barrels (42 gal.) of fuel oil at $3.48 per barrel delivered at the mill, of which about 20 cents represented the price of oil in Southern California, about $1 freight to Mesa, and the rest for 60-mile haul in tank wagons. In October, 1908, a contract was made for an additional 33,000 barrels at about $4.34 per barrel, about 50 cents of the increase being in the price of oil at the fields. One barrel of oil burned a trifle over four barrels of cement. The saving to the Reclamation Service from building and operating the mill was $1.75 per barrel upon very nearly the entire output of the mill, although a small quantity of cement was hauled toward the railroad and used in territory where outside cement might have competed.

In addition to furnishing the cement the Reclamation Service also furnished the sand. Natural sand from the river would have involved a very expensive haul, and the deposits would have been inaccessible as soon as any water was stored or in case of a freshet. A plant was erected for crushing sand from rock, which was done throughout the work with entirely satisfactory results. The total plant charge was a trifle more than $31,000, and the total cost of producing sand, including quarrying the rock, crushing, plant maintenance and depreciation, and overhead charges, was about $1.50 per cu. yd.

To furnish power for construction purposes there had been constructed a 225 sec. ft. capacity canal 19 miles long taking water from the Salt river and delivering it to a 1000-kw. wheel in a temporary power house at the dam site. This canal is a feature of the completed project. Power was sold to the contractor at ½ cent per h.p.-hour.

With cement, sand and power furnished as above the following prices were bid for the construction of the dam:

Excavation, class 1, sand and gravel in bottom, cu. yd. at	$1.75
Excavation, class 2, rock below river level, cu. yd. at	5.00
Excavation, class 3, rock above river level, including all waste material from spillways at	1.50
Rubble masonry in dam at	3.15
Rubble masonry in Wing walls at	4.75
Coping masonry, concrete in bridge piers, concrete bridges (three 60-ft. spans each), 2 at	$7,500.00

(For accounts of the progress of the work and some of the difficulties encountered see *Engineering News*, Sept. 10, 1908, *Engineering Record*, Dec. 31, 1910.) Some of the foregoing quantities were exceeded during construction, excavation Class 3 very largely, owing to the amount of waste in the spillways; and the masonry in the dam somewhat, owing to increasing the height of the dam 10 ft. after the contract was let. Around one end of the dam a tunnel was driven, lined and provided with gates. This tunnel, known as the sluicing tunnel was with floor at river level, and up to its capacity diverted the river during construction. Experience with these gates indicated the desirability of relieving them of the high operating head. For this purpose a second outlet tunnel was driven on the other side of the river at elevation about 125. Three lines of 5-ft. cast-iron pipe are placed through the dam, converging to a 9-ft. diameter tunnel and continuing some 100 ft. through the rock abutment to two outlets. On the upstream ends of the 5-ft. pipes are three 58-in. balanced valves. The frontispiece shows water discharging from this tunnel.

The cost of the dam and appurtenances to Dec. 31, 1911, nearly a year after completion, was as follows:

Roosevelt dam (as given in detail below)	$3,126,430.96
Sluicing tunnel, 108 sq. ft. section, 450 ft. long, excavation lining, reinforced concrete screen tower 78 ft. high, etc.	125,410.04
Hydraulic gates, including contract with manufacturers	128,832.18
Gate chamber, setting, accessories and design (for a description of these gates see *Engineering News*, May 30, 1907)	249,841.27
Reservoir	23,356.06
Second outlet tunnel, including three 58-in. balanced valves ($13,890), 270 ft. 5-ft. cast-iron pipe, 100 ft. of tunnel driven and lined, apparatus for operating, and for measuring discharge	55,713.49
Roadway approach to east end of dam	13,530.55
Total	$3,594,282.37

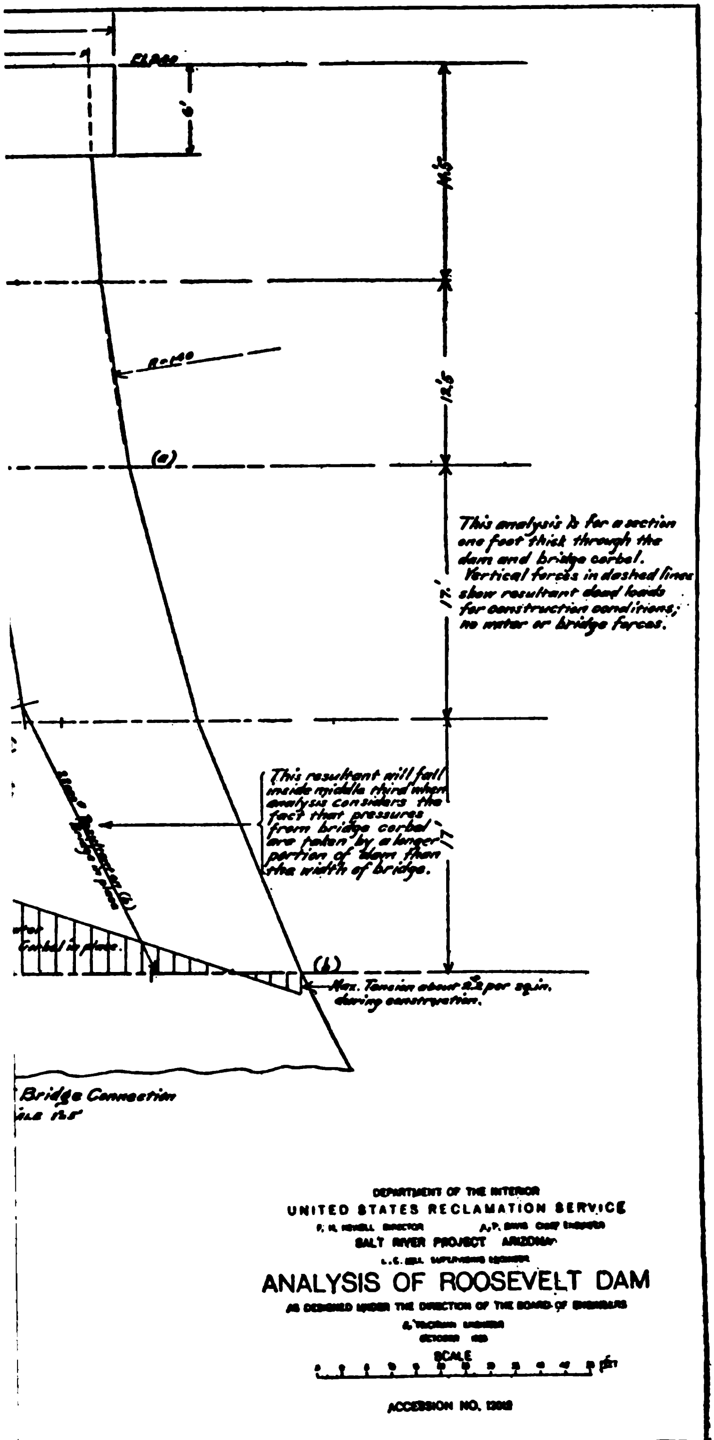
Details of Item above, Cost of Roosevelt Dam

Main contract, not final as some claims had not been adjusted	$1,691,103.87
Cement	787,336.43
Engineering	31,501.56
Inspection	43,426.77
Miscellaneous labor and material	26,200.23
Laboratory-tests of cement and concrete	1,921.53
Shop charges	629.54
Sand crushing plant and operation	65,313.81
Sand-quarry	71,298.28
Loss on power	58,319.78
Pilasters and capstone, architectural treatment of top of dam	7,478.60
Sprinkling system—labor $554.33, material $509.64	1,063.97
Lights for dam, labor $1166.25, material $3491.62	4,658.18
Bridge railings, etc.	1,841.02
Gate and tool houses on top of dam	14,340.01
Repairs at toe of dam, damage from flood over dam at elevation 90	15,579.64
Indemnity claims	2,045.25
Superintendence	17,709.16
Proportion of camp maintenance	64,891.31
Proportion of Washington office and other overhead charges	219,772.02
	$3,126,430.96

For further information regarding Roosevelt dam see pages 38, 69, 81, 88, 89, 104, 114, 139, 147, 160, 172, 206-210, 222, 223.

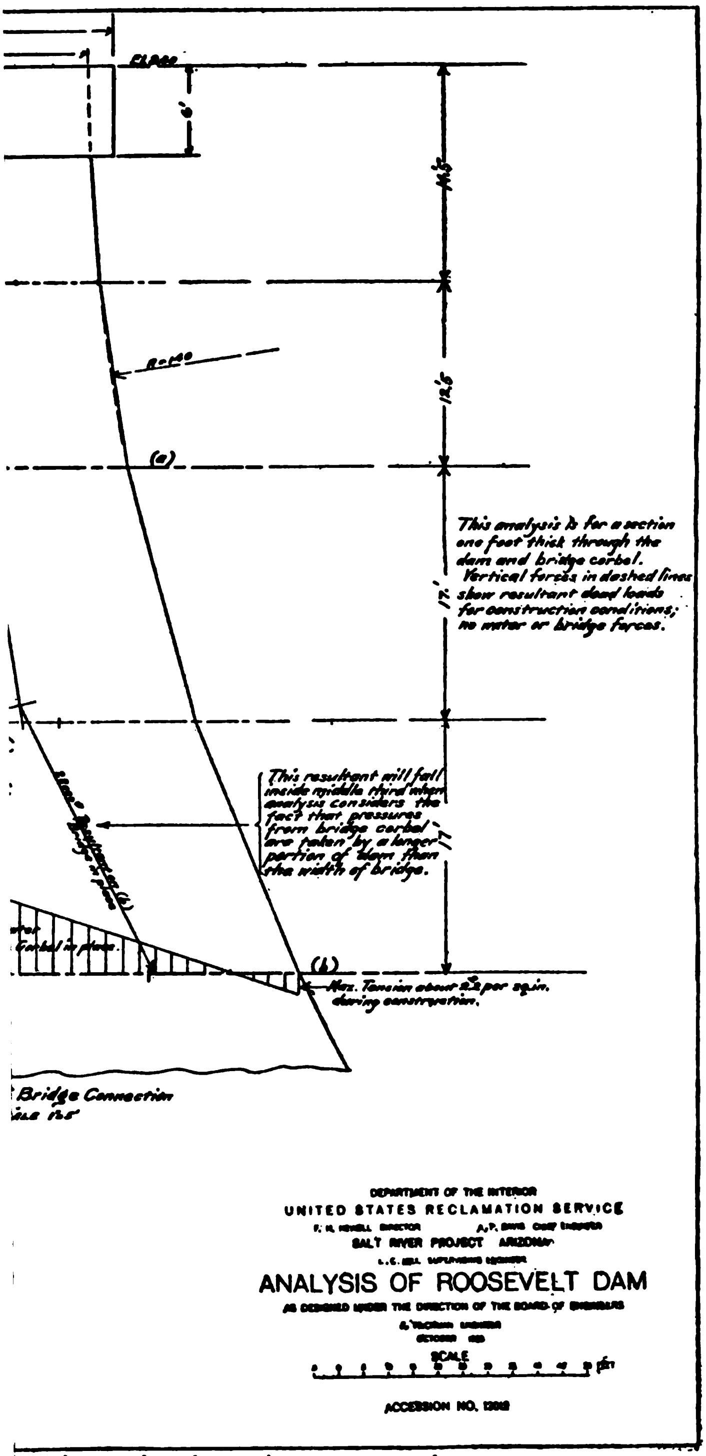
Pathfinder Dam (see Figs. 52 and 53).

Following are the quantities of the principal items of work as given in the specifications also the prices at which the contract was let.



w weir at each end; contains 342,000 cu. yd.

(*Facing page* 246.)



w weir at each end; contains 342,000 cu. yd.

(*Facing page 246.*)

Masonry above elevation 5660, 50,000 cu. yd. at $7.60	$380,000
Masonry below elevation 5660, 3000 cu. yd. at $9.95	29,850
Excavation, class 1, 2500 cu. yd. at $2.50	6,250
Excavation, class 2, 500 cu. yd. at $6.70	3,350
Excavation, class 3, 2500 cu. yd. at $4.30	10,750
Concrete, 1000 cu. yd. at $7.60	7,600
Hauling cement, 40,000 barrels at $3.23	129,200

The Reclamation Service furnished the cement at Caspar, Wyoming, 45 miles from the dam.

Before the work was started a tunnel 10 ft. wide × 13 ft. high, 480 ft. long was driven around one end of the dam. (For a description of the gates installed in this tunnel see *Engineering News* for January 2, 1908.) A rock-fill temporary dam diverted the flow of the river through the tunnel. Leakage through the rock-fill dam was intercepted by sand bag dams and conducted across the excavation in a small wooden flume. The deepest point in the excavation was 21 ft. below mean low water or 14 ft. below the river bed. The total masonry in the dam as per the final estimate was 60,210 cu. yd., an increase of about 7000 cu. yd. over the preliminary estimate. Undoubtedly this involved an increase in some of the excavation items, and an increase in the item of cement to about 5600 barrels.

The *Eleventh Annual Report* of the U. S. Reclamation Service gives the following as the cost of the works as follows:

Pathfinder dam, including main contract, cost of cement, etc.	$892,514.98
Buildings, gate-keepers house, emergency gate house, and store house	19,252.57
Pathfinder tunnels (contract price for 480 lin. ft. of tunnel and of shaft excavation was $33,295.75)	199,669.21
High-pressure gates	$191,098.97

Much of the stone for the dam was quarried from the spillway channel distant 300 ft. from the north end of the dam. Ten guy derricks were used on the work, six in the quarry and four on the dam. One 15-ton and one 10-ton cableway of 350-ft. span delivered materials. Four boilers aggregating 200 h.p. furnished power for all of the work. The maximum progress on masonry was during August, 1908 when 5040 cu. yd. were laid, or an average of 168 cu. yd. per day. This quantity would have been much larger but for the small working area.

Shoshone Dam (Fig. 53).

Includes also:

Spillway tunnel 20 ft. square in section × 500 ft. long.

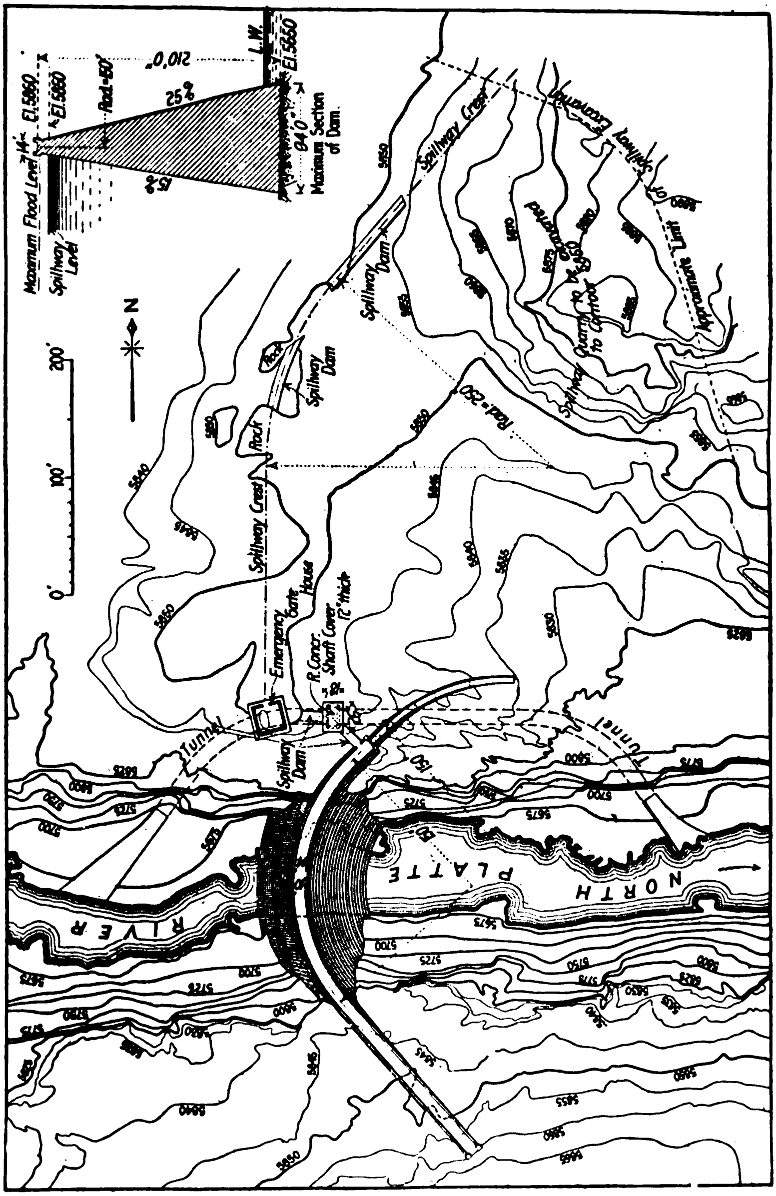


FIG. 52.—Pathfinder dam, Wyoming; built 1905 to 1909; granite rubble, masonry length 342 ft.; max. height 218 ft.; contains 60,210 cu. yd.

Outlet tunnel at stream level 10 ft. square in section × 500 ft. long. Three gates each 3 ft. × 7 ft. clear opening operated by pumping oil into cylinders.

Second outlet tunnel, not contemplated in original contract, 110 ft. above stream, 80 sq. ft. in section × 300 ft. long.

Let by contract at the following prices

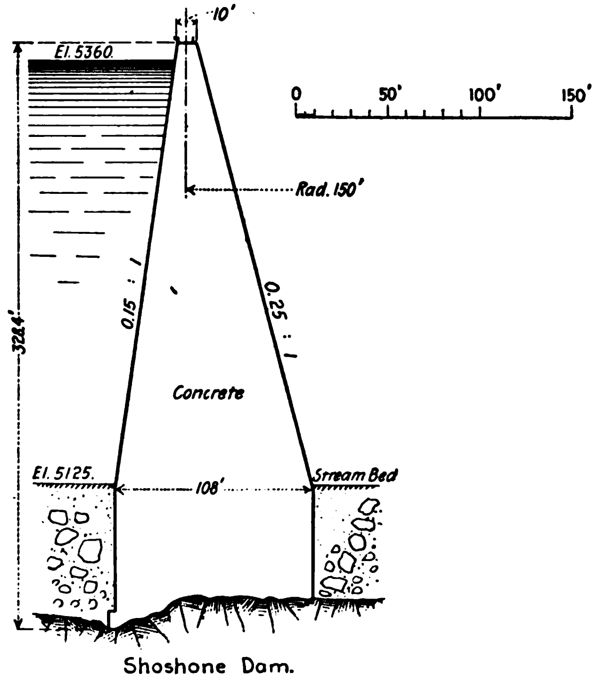


FIG. 53.—Section of Shoshone dam, Arch Wyoming; built 1903 to 1910; rubble concrete masonry; length 200 ft.; max. height 328 ft.; contains 75,534 cu. yd.

Temporary works, crib dam 12 ft. × 200 ft., flume 9 ft. × 13 ft. × 1300 ft,	$37,000
Excavation, class 1, 32,000 cu. yd. at $1.60	51,200
Excavation, class 2, 3000 cu. yd. at $2.50	7,500
Excavation, class 3, 1000 cu. yd. at $3.50	3,500
Excavation, class 4, 6000 cu. yd. at $2.00	12,000
Excavation in spillway (open cut), 28,000 cu. yd. at $1.95	54,600
Excavation in spillway (tunnel), 7000 cu. yd. at $5.50	38,500
Excavation in road tunnel, 150 lin. ft. at $26.000	3,900
Excavation outlet tunnel 10 ft. × 10 ft., 480 lin. ft. at $36.00	17,280
Concrete in base of dam, 19,000 cu. yd. at $4.75	90,250
Concrete above base of dam, 50,000 cu. yd. at $4.00	200,000
Total	$515,730

The United States furnished the cement at Cody, 8 miles from dam.

The original contractors resigned and the work was sub-let by the bonding company to another contracting firm who completed it.

The *Tenth Annual Report* of the U. S. Reclamation Service gives the following feature costs to June 30, 1911:

Dam and appurtenances	$869,018.93
Lower outlet tunnel	20,975.96
Sluice gates	70,435.19
Upper outlet tunnel	19,715.12

These feature costs include in addition to payments under the contract, sluice gates, cement, engineering, inspection, etc.

The same report gives the earnings on the bonding company's contract to June 30, 1911 as $502,027.19.

The *Eleventh Annual Report* stated that the contract was completed with earnings of $505,631.76. There was at that time, however, and still (January, 1914) is, pending in the Court of Claims a suit brought by the final contractors against the Government for the recovery of a considerable sum for alleged changes in the contract.

It is not known what the loss (if any) was to the original contractors and the bonding company. For additional information see pages 39 and 40.

Arrowrock Dam (Figs. 54–54A).

Being built by force directly employed by U. S. Reclamation Service to store water for irrigation and incidental power; the highest dam yet built. Of concrete (built between forms) containing as many plums as can be introduced with economy (about 20 per cent.) 250,000 cu. yd. of excavation (sand and gravel) in the bottom, and 350,000 cu. yd. rock excavation in the spillway. A large part of these two items will be used in the masonry. Equipped with two 15-ton 1500-ft. span cableways with electric motors. These, with 4-cu. yd. skips, orange-peel and clamshell buckets handled the excavation in the bottom.

For description and cost of temporary works see page 31.

For account of construction and operation of railway see page 128.

For electric power load curves see page 141.

For an excellent article on sand and gravel excavation methods and costs see *Engineering News*, July 17, 1913.

Elephant Butte Dam (Figs. 55–55A).

Being built, by force employed directly by U. S. Reclamation Service.

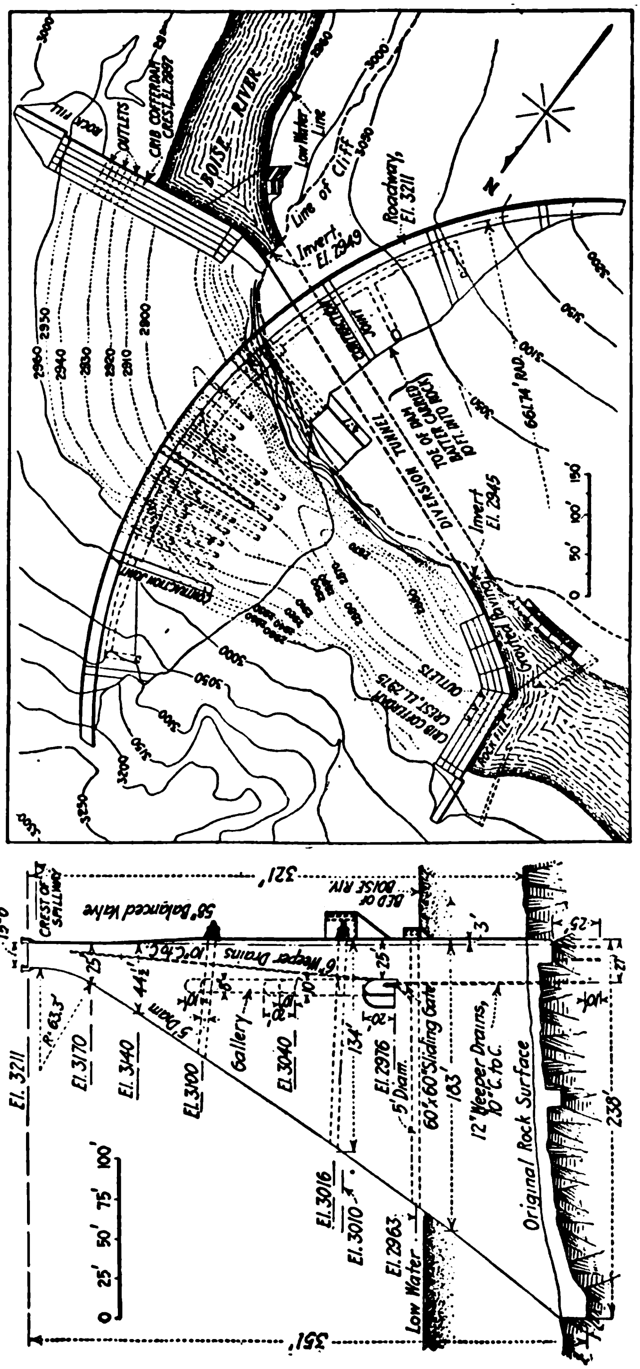


FIG. 54.—Arrowrock dam, Idaho; started 1912; length 1060 ft.; curved to R = 662 ft.; max. height 351 ft.; will contain about 530,000 cu. yd.

Reservoir capacity 2,627,000 acre ft., making it the largest artificial reservoir in the world. A railroad $10\frac{1}{2}$ miles in length furnishing access to the work cost—

Surveys	$3,348
Grading for railroad, terminal yards, and for buildings, 80,000 cu. yd., also labor laying ties, rails and ballast; one mile near the dam is double tracked	168,635
Trestle work Ash and Spring canyons, and culverts, 350 M. ft. B.M. timber	20,528
Track scales	2,685
Total	$195,196

Three $2\frac{1}{4}$-in., 8-ton, 1400-ft. span, Lidgerwood cables; each with 300-h.p., 2200-volt motor, hoisting speed 200 ft. and conveying speed 800 ft. per minute, cost

Head towers, 80 ft. high	$8,646
Tail towers, 125 ft. high	10,687
Cableways, engines, complete, including freight and installation	61,847
Total	$81,180

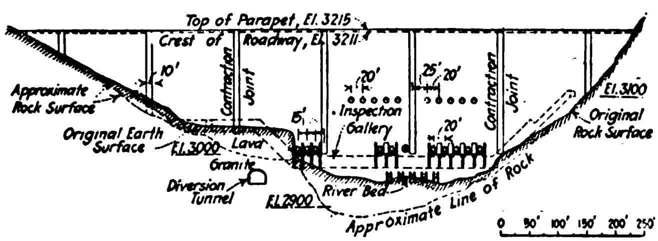


FIG. 54A.—Developed front elevation of Arrowrock dam.

For cost of power plant, see page 221.

For description of sand cement, crushing, and mixing plants, see pages 152–155.

For description and cost of diversion works see pages 36–37.

The sand required for the masonry comes from the excavation in the river bed. Grab buckets on the cableway take up the sand and dump it into hopper bins. Thence it is drawn into cars, and by the cars taken to and dumped into a storage pile. When required for use the sand is loaded into cars by derrick and grab bucket, brought back to the mixing plant, dumped into a hopper and elevated to the bins over the mixers.

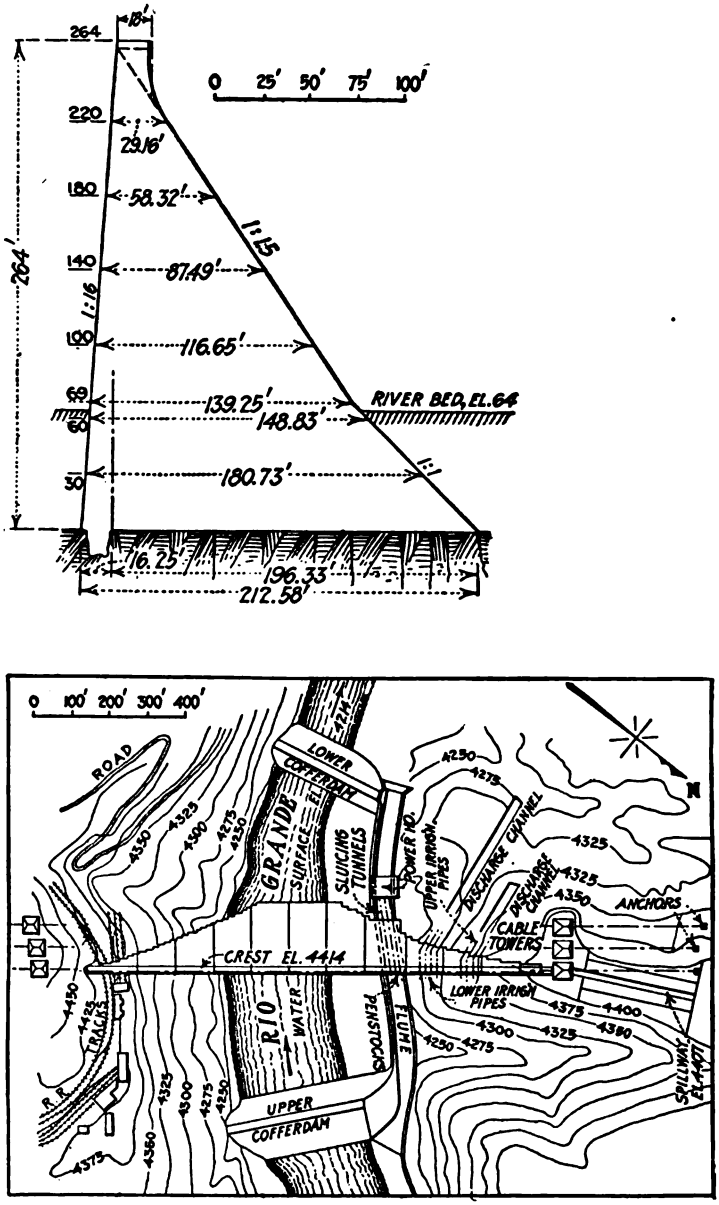
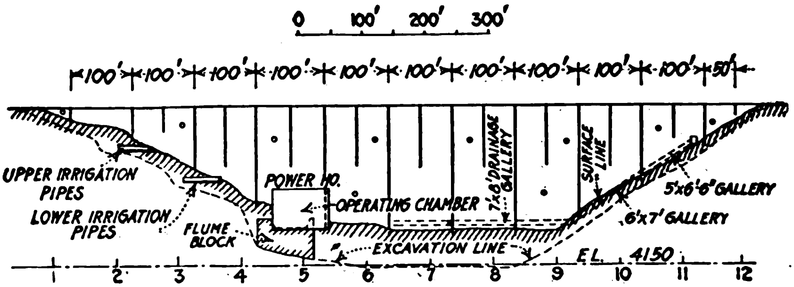
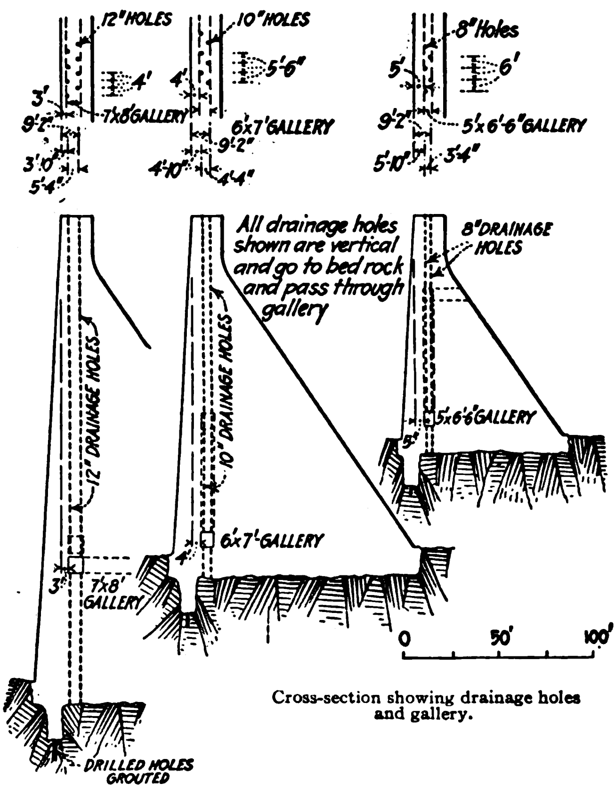


FIG. 55.—Elephant Butte dam, New Mexico; begun 1910; length 1200 ft.; max. height 275 ft.; will contain about 500,000 cu. yd.



Elevation showing expansion joints.



Cross-section showing drainage holes and gallery.

FIG. 55A.—Elephant Butte dam.

Cataract Dam (Fig. 56).

Plant, installed by the Government, consisted of

Two cableways, 1100-ft. span, on 57-ft. towers Six 6-ton stiff-edge derricks Four 1-cu yd. concrete mixers	Electrically driven
In quarry: Four 10-ton, One 5-ton, One 3-ton	Locomotive cranes steam-driven
Air compressor	
Two stone crushers	
Pumps, etc.	

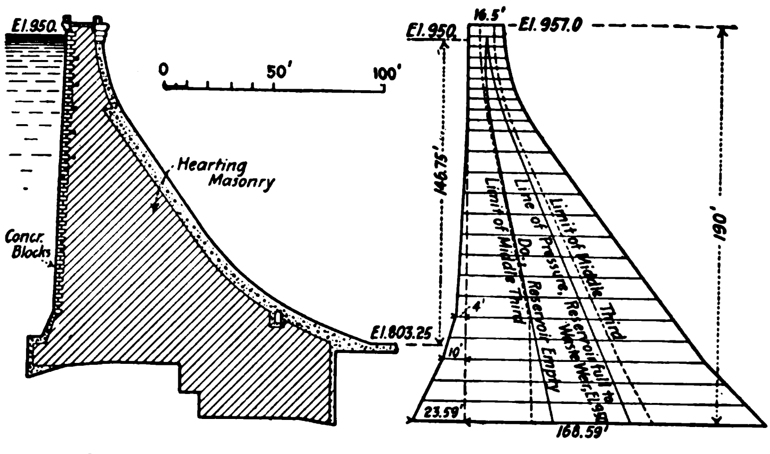


FIG. 56.—Cataract dam, New South Wales; built 1902 to 1908; cyclopean masonry; upstream face of concrete blocks; downstream face of concrete length, 811 ft.; max. height, 192 ft.; contains 146,242 cu. yd.

Above plant cost installed $161,000.
Principal items of work:

Excavation	215,000 cu. yd.
Cyclopean masonry	111,455 cu. yd.
Rubble masonry	1,975 cu. yd.
Concrete masonry	23,846 cu. yd.
Concrete face blocks	8,966 cu. yd.
Iron work, pipes, valves	319 long tons

All stone for concrete, and all fuel hauled $6\frac{1}{2}$ miles over 2 ft. gage railroad.

Cyclopean masonry, 65 per cent. sandstone, 35 per cent. mortar and concrete.

Constructed in 4 years, 11 months.

Cost:

Clearing	$115,000
Excavation, 215,000 cu. yd	237,000
Masonry	1,037,000
Outlet works	26,000
Roads	28,000
River diversion	19,000
Valve house	24,000
Sanitary, medical, insurance	25,000
Supervision and contingencies	91,000
Total	$1,602,000

Vyrny Dam (Fig. 57).

Cost as follows:

Borings and preliminary	$34,600
Excavation 220,820 cu. yd. and refill 79,500 cu. yd	287,000
Puddle wall, including excavation	16,800
Masonry and brick-work	2,532,000
Regulating works	46,000
Basin and other work below dam	40,000
Total	$2,957,000

Seven steam cranes employed on dam, each with engineer and eighteen men laid on an average 40 cu. yd. per day.

Double-track, 3-ft. gage railroad between dam and quarry, 1 mile.

System of drains in the foundation.

Granite Springs Dam.

Wyoming; built 1903 and 1904; rubble masonry; length 410 ft.; max. height 96 ft.; contains 14,422 cu. yd.

Total cost of dam including water rights, land, clearing, building, excavation, outlets, spillway, engineering, superintendence, general expense .... = $109,194.

Rubble stone = 64.8 per cent. Mortar = 35.2 per cent.

Rubble masonry cost the contractor:

		Per cu. yd. Masonry
Quarrying 9414 cu. yd. at $1.96	$18,452.60	$1.28
Mortar 5008 cu. yd. at $1.93	9,676.08	0.67
Laying 14422 cu. yd. at $1.11	16,017.90	1.11
Cement 8844 barrels at $3.58	31,665.38	2.13
Total	$75,811.96	

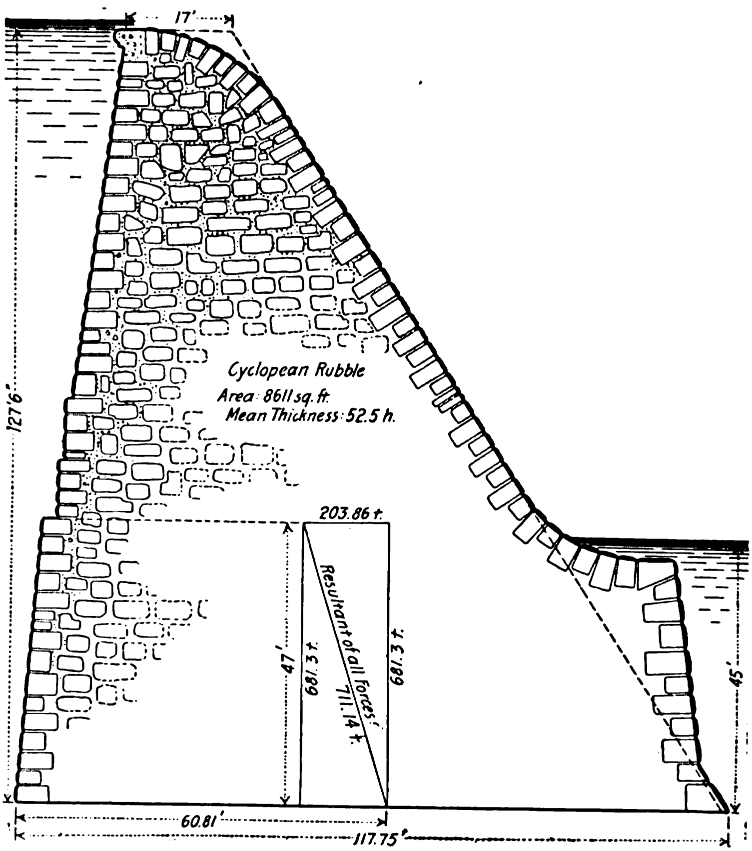


FIG. 57.—Vyrny dam, Wales; built 1882 to 1889; cyclopean rubble masonry; length 1350 ft.; max. height 136 ft.; contains 260,000 cu. yd.

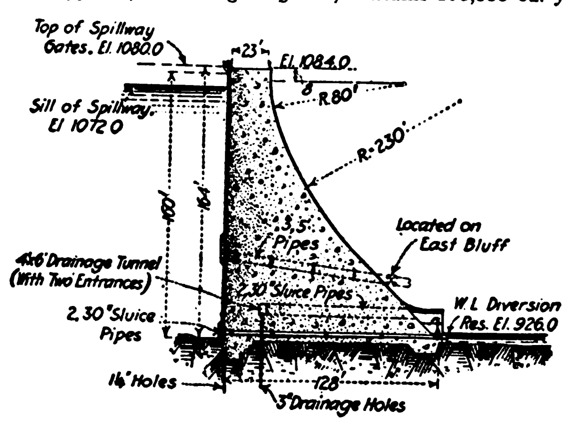


FIG. 57A.—Section of Medina dam, Texas; built 1911 to 1913; length on crest 1580 ft., at river level about 600 ft.; max. height 166 ft.; contains 205,000 cu. yd.

Medina Dam (Fig. 57 A).

The dam has a base thickness of 128 ft. A cut-off trench extends 14 ft. below the general level of the foundation, or to 180 ft. below the crest of the dam. In the dam there is a drainage tunnel, 13 ft. above the foundation and 23 ft. from the upstream face.

From the bottom of this tunnel 122 3-in. drainage wells, 4 ft. to 5 ft. apart extend to a depth of 30 ft. below the foundation. A branch railroad about 19½ miles long was built from Dunlay on the Southern Pacific railroad over which was transported plant, cement and oil used for fuel in generating steam power. The principal items of plant were two 10-ton cableways of 1180-ft. and 1250-ft. span, four derricks with 70-ft. booms, five 1-cu. yd. Smith mixers, dumping into hoppers, seven 150-h.p. locomotive type boilers, two No. 7½ D Gates crushers; sand crushing rolls compressor, 2-cu. yd. bottom dump buckets, cars and electric lights.

Limestone for concrete was quarried 1000 ft. to 2000 ft. north of the west end of the dam, and another quarry 500 ft. south of the east end furnished plums. (See page 37 for reference to stream diversion, and page 162 for masonry progress.)

The first masonry was laid in November, 1911 and the last in December, 1912.

Barker Dam.

Boulder, Colorado; built 1909; cyclopean concrete; length 250 ft. on bottom, 625 ft. on crest; max. height 185 ft.; max. thickness 123 ft.; contains 140,000 cu. yd.

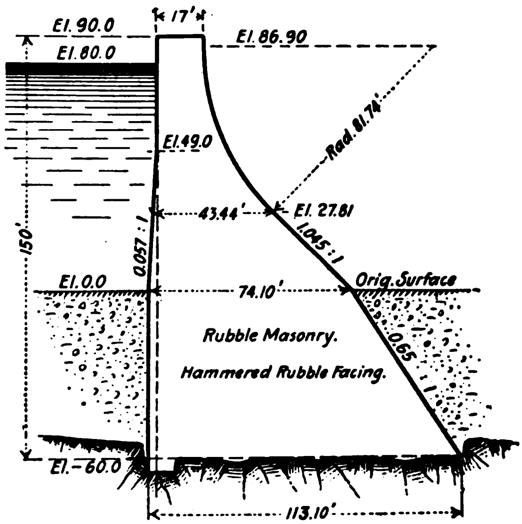
Above an elevation 145 ft. below the top of the dam transverse expansion joints were constructed each 48 ft. During construction the flow of the stream was passed through a 4 ft. × 6 ft. opening through the dam. Watershed 40 square miles, reservoir capacity 524,100,000, cu. ft.

Spier Falls Dam (Figs. 58–58A).

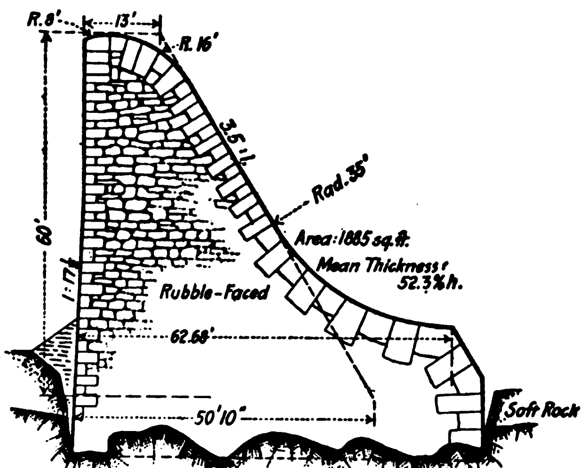
(See page 32 for an account of the stream diversion works.)

Plant included seven cableways, of which three were respectively 1000 ft., 1670 ft. and 2140 ft. long, and mounted parallel to the dam; the remaining four were each 670 ft. long, installed at right angles to the dam. At first coal was used for power but in 1902 a transmission line 23 miles long was built from Mechanicsville and electric power was used to run three Duplex two stage Rand air compressors.

Two of the compressors were 27 in. × 17 in. × 30 in. and one was 14 in. × 22 in. × 16 in. They furnished air at 80 lb. pressure to a



Spier's Falls Dam.



Spier Falls Dam, Hudson River, N.Y.

(Eng. Record, March 8, 1902.)

FIG. 58–58A.—Section of Spier's Falls dam, New York; built 1900 to 1905; length 552 ft.; max. height 150 ft.; contains 180,000 cu. yd.

system of 8-in. air mains covering the work. The excavation amounted to 270,000 cu. yd. The maximum masonry progress was 8000 cu. yd. per month, and the labor cost of laying, not including mixing was said to be 60 cents per cu. yd. The total cost of the work has never been published.

Colorado Dam (Fig. 59).

Overflow dam—straight.

Contains 70,000 cu. yd. limestone rubble, contract price, $3.60 cu. yd.

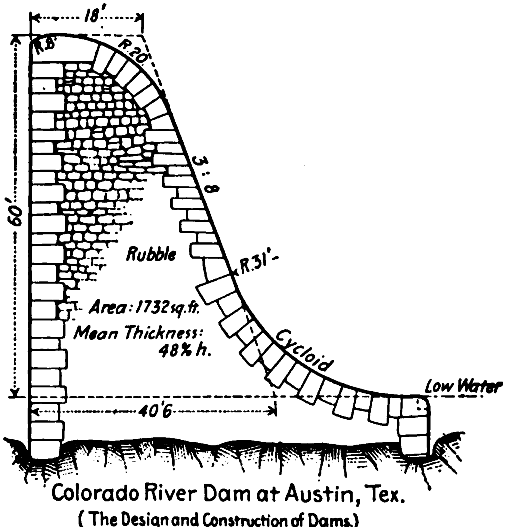


FIG. 59.—Section of Colorado dam, Texas; built 1891 to 1892; rubble masonry; length 1125 ft.; height 66 ft.; contains 88,000 cu. yd.

Contains 18,000 cu. yd. granite facing, contract price $11 to $15 cu. yd.

The granite was quarried 80 miles from the dam. Cost $608,000. (Including power house and distribution system $1,400,000.)

Failed April 7, 1900, with 11.07 ft. of water over crest.

See paper No. 40 of *Water Supply & Irrigation Papers* of U. S. Geological Survey.

La Grange Dam (Fig. 60).

Overflow dam—curved on radius of 300 ft.

Rubble in Portland cement mortar contracted at $10.39 per cu. yd. which included excavation but not cement. Used 31,500 barrels of cement, cost $4.50 per barrel. Has passed 12 ft. depth of water.

Cost $550,000.

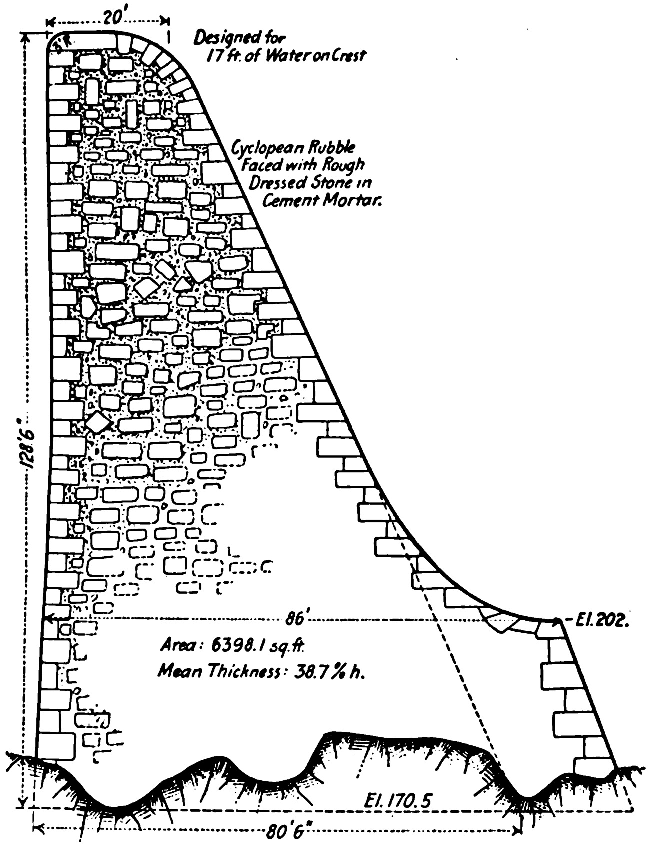


FIG. 60.—Section of La Grange dam, California; built 1891 to 1894; rubble masonry; length 320 ft.; height 127.5 ft.; contains 39,500 cu. yd.

Poona Dam (Fig. 61).

Built on several tangents, reinforced by heavy buttresses at the angles.

Height above river bed 98 ft.

Cost $630,000.

Waste weir 1453 ft. long, 11 ft. below remainder of dam.

Tansa Dam (Fig. 62).

Built heavy enough to permit being raised 17 ft. Small amount

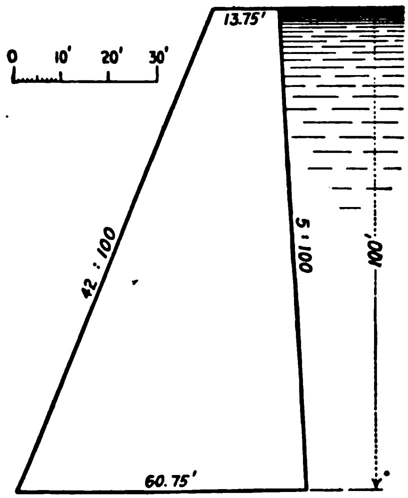


FIG. 61.—Section of Poona dam, India; rubble masonry; length 5136 ft.; height 108 ft.; contains 360,000 cu. yd.

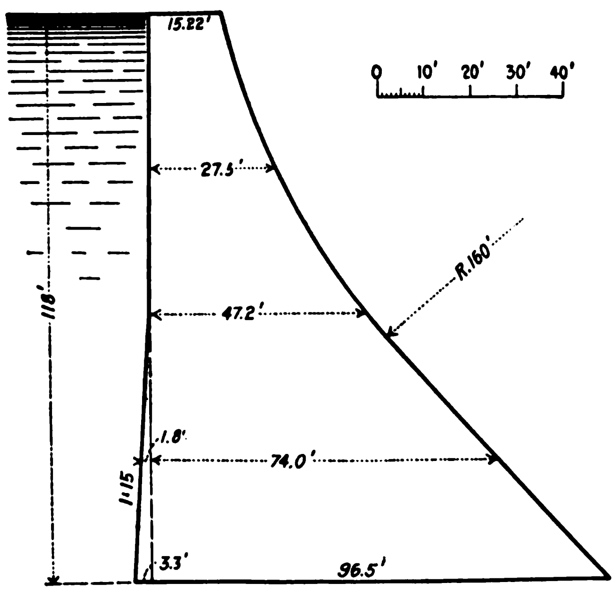


FIG. 62.—Section of Tansa dam, India; built 1886 to 1891; rubble masonry; length 8800 ft.; height 118 ft.; contains 408,520 cu. yd.

of Portland cement used, though chiefly built with hydraulic lime mortar burned at dam site.

Excavation in places 45 ft. deep.

Principal items of work were:

Excavation	251,127 cu. yd.
Loose rubble stone	544,700 cu. yd.
Lime	81,700 cu. yd.
Washed sand	122,555 cu. yd.
Rubble masonry	408,520 cu. yd.

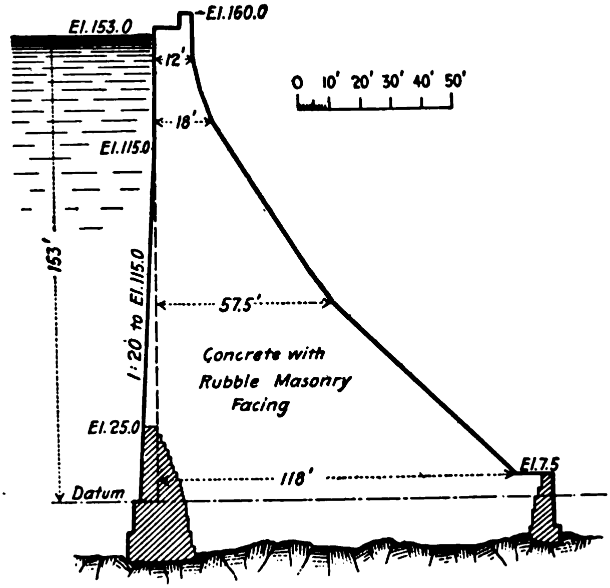


FIG. 63.—Section of Periyar dam, India; built 1888 to 1897; concrete masonry; length 1231 ft.; height 173 ft.; contains 185,000 cu. yd.

Proportion of mortar in rubble masonry = 36.7 per cent.

Maximum progress during month of January, 1891 when 700 masons laid 26,000 cu. yd. of rubble.

Built macadamized road, 8 miles from nearest railroad station.

Total cost $988,000.

Periyar Dam (Fig. 63).

Work on foundations limited to a low water season of only three months. Flood discharges up to 120,000 c.f.s.

Concrete with uncoursed rubble faces.

Concrete composed of 25 parts hydraulic lime, 30 parts of sand and

100 parts of broken stone. Native labor. Lime was obtained from a quarry 16 miles from dam. Has 920 ft. of spillway, 403 ft. being formed by a masonry wall, rest excavated from rock. Outlet channel involves rock excavation 21 ft. wide on bottom 5500 ft. long and running up to 50 ft. depth; also a rock tunnel 6650 ft. long with 80 sq. ft. section. Native labor, under difficult and expensive conditions. Estimated cost of entire work $3,220,000.

See also *Water Supply & Irrigation Paper* No. 87.

Assouan Dam (Fig. 64).

Of the total length, 1800 ft. is solid masonry and the remainder contains 180 sluices provided with gates for passing the high water

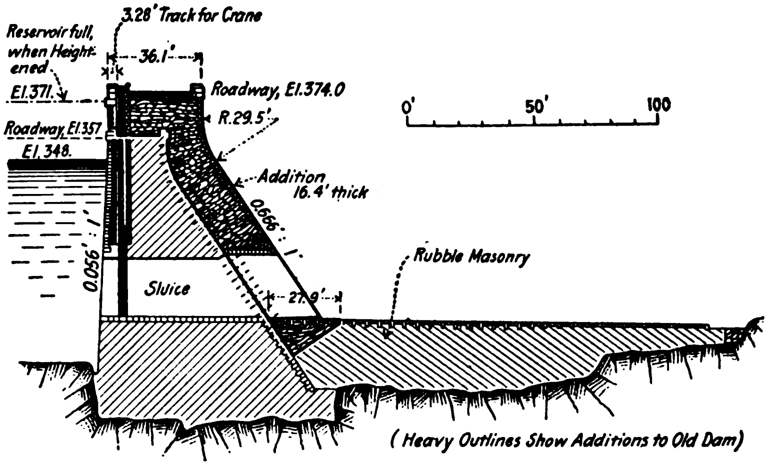


FIG. 64.—Section of Assouan dam, Egypt; built 1898 to 1903; granite rubble; length 6400 ft.; height 96 ft.; contains 704,000 cu. yd.; 1907 to 1910 raised 16.4 ft., involving additional thickness of 16.4 ft. normal to face; no addition to length, additional masonry about 475,000 cu. yd.

flow. Of the sluices 140 are 6.56 ft. wide × 22.96 ft. high, and forty are the same width by one-half the height. The original work includes 6540 ft. of canal 49 ft. wide, four locks each 263 ft. × 31 ft.; also the Assiout dam 350 miles downstream from the Assouan, built for the purpose of diverting the water into the irrigating canals.

The Assiout dam is of masonry 2769 ft. long, with a maximum height of 48 ft., and retains about 33.5 ft. of water. It contains 111 arched openings each 16 ft. 4 in. span, closed by steel sluice gates 16 ft. high. The original contract price was £1,500,000 for the Assouan and £500,000 for the Assiout; but the foundation for the Assouan was considerably deeper than anticipated, and required expensive treatment, so that the actual cost of the Assouan dam was £2,450,000.

In May, 1907, a contract was let at $5,110,000 for raising the dam 16.4 ft., including construction of an additional lock, etc. Iron and steel, incidental and additional work increased the cost to $7,410,000. The masonry added to the downstream face was not at once bonded

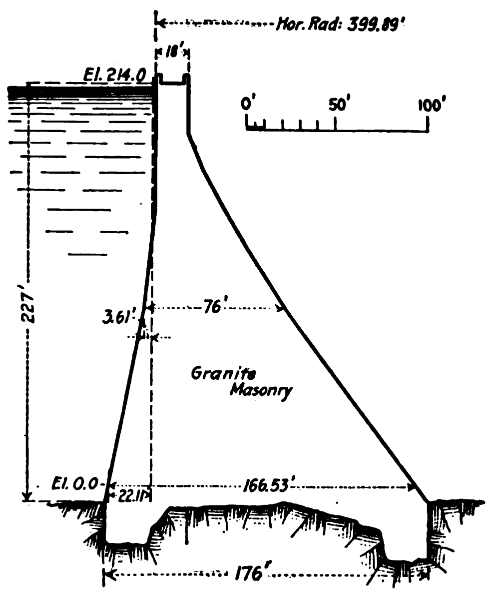


FIG. 65.—Section of Lake Cheesman dam, Colorado; built 1900 to 1904; granite rubble masonry; length 710 ft.; max. height 227 ft.; curved to radius 400 ft.; contains 103,000 cu. yd.

with the old work, but built with a narrow intervening space which was grouted later when the new masonry had settled and acquired the temperature of the old. The capacity of the original reservoir was about 900,000 acre ft., this was increased by raising the dam, by about 1,070,000 acre ft.

Lake Cheesman Dam (Fig. 65).

The rubble was composed of 74 per cent. stone and 26 per cent. mortar; 0.78 barrels of cement was required per cu. yd. The cement was hauled 30 miles by wagon at a cost of 1 cent per 100 lb. per mile. Total cost of cement at dam was about $4 per barrel. Watershed

1800 square miles, maximum flood 1945 c.f.s. Reservoir capacity 3½ billion cu. ft. The quarry from which the stone was obtained was about 2000 ft. from the dam.

The outlets from the reservoir consisted of three tunnels, aggregating 1300 lin. ft. The excavation at the dam-site amounted to 2600 cu. yd., mostly rock. It was only necessary to go down about 10 ft. below the river to reach a satisfactory foundation, and the handling of the stream seems not to have been an expensive matter. The total cost is stated to have been about $1,000,000.

La Boquilla Dam.

On Rio Conchos, Chihuahua, Mexico; started 1910; length 840 ft.; max. height 261 ft.; bottom thickness 200 ft.; curved to radius 866 ft.; will contain about 390,000 cu. yd.

The excavation amounts to 96,000 cu. yd. of earth and 360,000 cu. yd. of rock. Of the rock excavation 260,000 cu. yd. is from the spillway. Portland cement used. First masonry laid March, 1911, work suspended in 1913 on account of the Mexican revolution. Reservoir capacity will be 2,800,000,000 cubic meters and requires an auxiliary dam 2610 ft. long × 108 ft. maximum height.

Four cableways installed; used electric power generated nearby at a steam power plant, using principally oil for fuel.

Mercedes Dam.

Durango, Mexico; built 1901 to 1905; rubble masonry with cut stone facing; length 535 ft.; max. height 132.8 ft.; contains 28,000 cu. yd.

Crest length does not include 98 ft. of spillway 6 ft. below crest. Includes diversion tunnel 6 ft. × 6 ft. 5 in. in section and 77.6 ft. long.

Of the mass of masonry 37 per cent. equals mortar. Ten per cent. of the masonry was laid in Portland cement (using 1800 parcels) and 90 per cent. in hydraulic lime mortar manufactured nearby.

Mexican labor—very largely hand work. Total cost $200,000 Mexican.

La Jalpa Dam.

Mexico; built 1902; limestone rubble masonry; length 1800 ft.; max. height 87 ft.; contains 92,000 cu. yd.

Used native hydraulic lime mortar, cost $500,000. Gold.

Boyds Corners Dam (Fig. 66).

Rubble 21,000 cu. yd., cut stone 6000 cu. yd. Total cost $370,000.

San Mateo Dam (Fig. 67).

This dam was planned for a maximum height of 170 ft. (at which height it would be 680 ft. long), but was stopped at 146 ft. height.

Built of 1–2–6½ concrete deposited in interlocking blocks about 30 ft. × 30 ft. × 10 ft. in size, molded in place.

Rock was hauled in wagons about 1000 ft. from the quarry.

Cement cost in San Francisco $3 to $3.25 per barrel, thence hauled by rail 20 miles to San Mateo, and by wagon 5 miles to the dam. The sand came from North Beach, San Francisco where it was loaded on tram cars, dumped into barges, towed 20 miles up San Francisco Bay to San Mateo landing, thence hauled 8 miles by

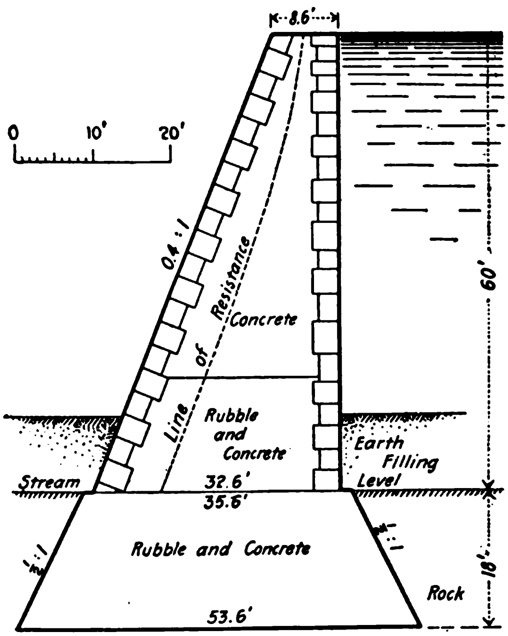


FIG. 66.—Section of Boyds Corners dam, New York; built 1867 to 1870; rubble masonry; length 670 ft.; max. height 78 ft.; contains 27,000 cu. yd.

wagon to the dam. Its cost delivered was $2 per cu. yd. The rock was crushed and the concrete mixed at one end of the dam, the concrete run out in cars on a high trestle, dumped through chutes, then shoveled into wheelbarrows and wheeled to place. The maximum rate of progress was about three of the above-mentioned blocks, or 1000 cu. yd. per day.

Estimated cost for the dam to the 170 ft. height $2,000,000.

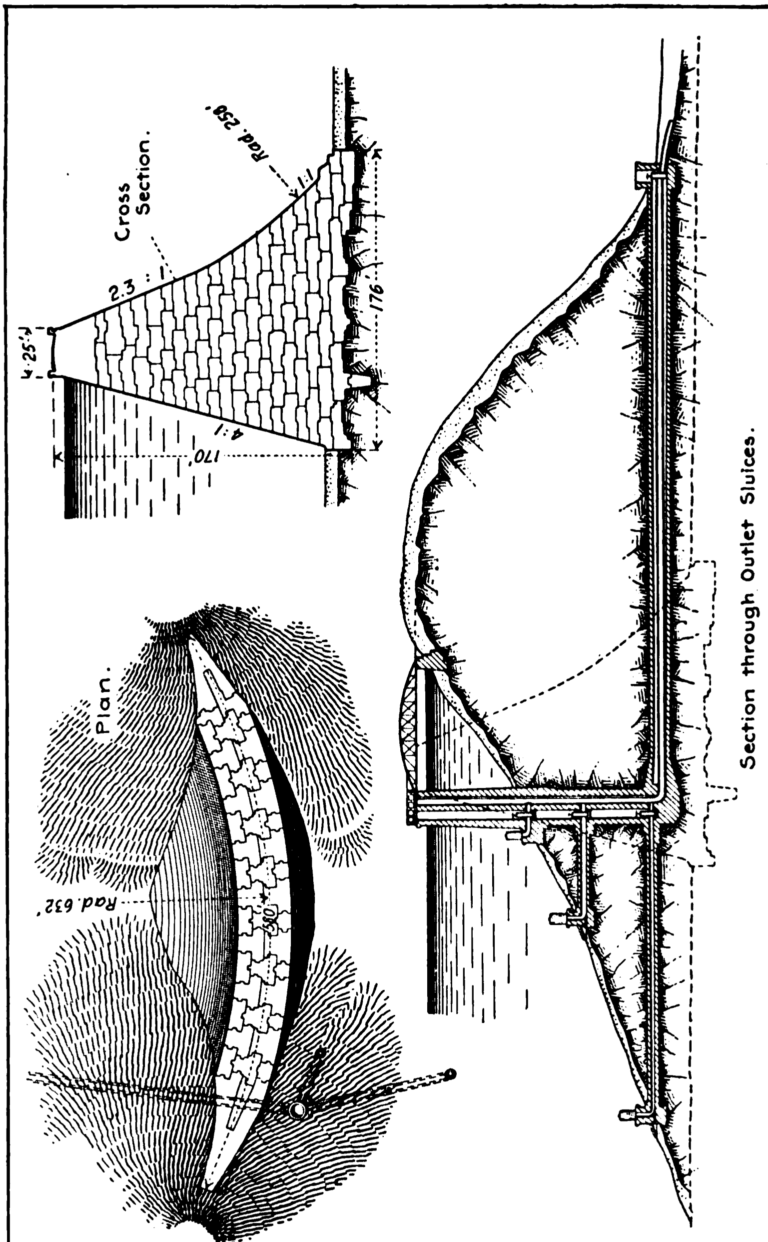


FIG. 67.—Section of San Mateo dam, California; built 1887 to 1889; length 580 ft.; max. height 146 ft.; curved to radius 835 ft.; base thickness 176 ft.; contains 139,000 cu. yd.

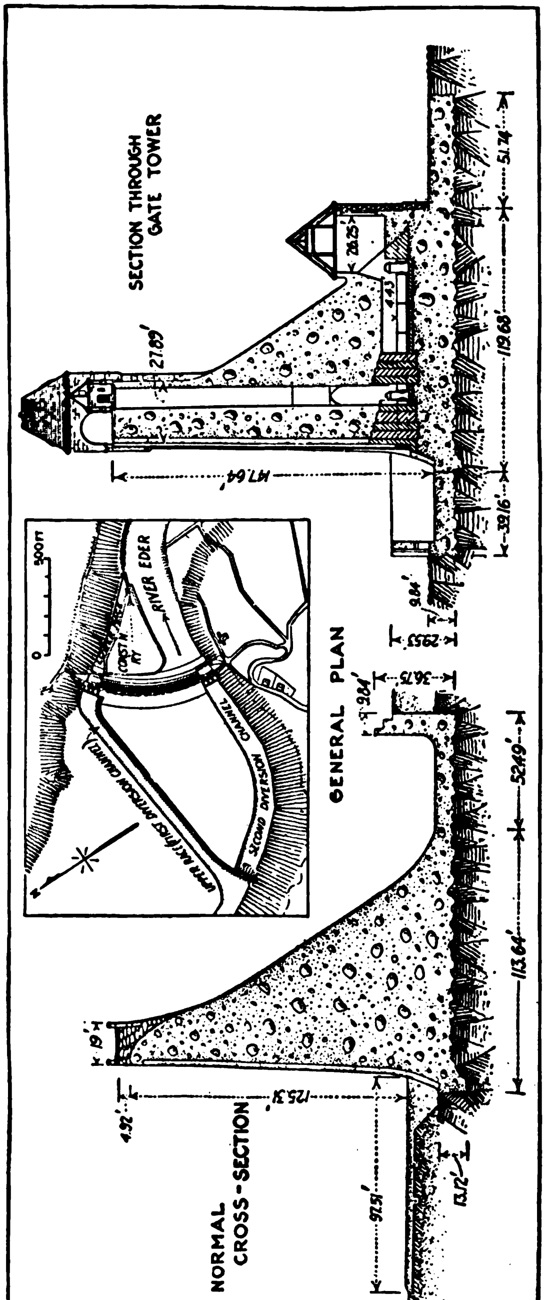


FIG. 68.—Plan and section of Waldeck dam, Germany; started 1911; length about 900 ft.; max. height about 160 ft.; will contain about 300,000 cu. meters; curved to radius of 1000 ft.

Mauer Dam.

Germany; built 1904 to 1912; rubble masonry; length 918 ft.; max. height 203 ft.; contains 332,000 cu. yd. masonry; required 222,000 cu. yd. of excavation.

During 1904–05 built a diversion tunnel 29.5 ft. wide × 23 ft. high × 1255 ft. long through rock, capacity 10,590 sec. ft.

Upper temporary dam 30 ft. high, 900 ft. above main site, a low dam 460 ft. below main site prevented the water from backing into the pit. Maximum discharge of the river 42,360 sec. ft.; minimum 53 sec. ft.

Stone was quarried in the valley 2½ miles from the dam, brought to the work on hand cars, and hauled by a hoisting engine up an incline to the top of the work. Sand was obtained from the valley directly below the dam. First stone laid June, 1908, completed November 16, 1912.

Spillway was 285 ft. long, 5.9 ft. below crest of dam. At maximum rate of progress laid 1000 cu. yd. per day, and employed 250 masons and 550 laborers.

In connection with the dam, built a power house in which are four 1500-h.p. units. In year of normal rainfall the power is reckoned as 3000 h.p. twenty-four-hour power, at 89 ft. head.

Cost of dam and appurtenances. . . $1,416,000.

Reservoir area 593 acres.

Capacity 1765 million cu. ft.

Watershed area 467 square miles.

Waldeck Dam (Fig. 68).

Built of rubble composed approximately of ⅔ stone and ⅓ mortar; the mortar consisting of 1 volume of lime, 1½ of trass (volcanic slag) and 2 of sand, a mortar in common use in Germany for hydraulic construction.

The excavation amounted to about 200,000 cubic meters (261,585 cu. yd.) of loose soil and rock. Two hundred masons laid 700 cubic meters (916 cu. yd.) of masonry per day, 200 men cleaned the stones and 300 were engaged in the quarries. The total force was about 900. The drainage area above the dam is 542 square miles, and the reservoir capacity is 202,400,000 cubic meters or 53.5 billion U. S. gallons. The estimated cost of the dam is $1,880,950.

Name	Location	Date	Length	Max. Height	Cu. Yd. Masonry	Cost	
Furens.....	France....	1862 1866	330	170.0	52,300	$318,000	
Lauchensee	Germany.	1892 1895	840	98.0	37,400	243,750	
Einsiedel...	Germany.	1890 1894	590	93.6	31,600	312,500	
Remscheid.	Germany.	1889 1892	...	82.0	22,886	91,154	
Gillepe.....	Belgium...	1870 1875	771	154.0	325,000	874,000	
Sweetwater	California	1887 1888	380	98.0	20,507	234,074	Rubble in 1–3 Portland cem. mortar. Stone quarried 800 ft. from dam.
Barren Jack	New South Wales	1906 1911	784	240.0	320,000	3,680,000 Estimated	Reservoir capacity 785,000 acre ft. Cost includes 26 miles of railroad, new highways, etc. Cu. yd. of masonry estimated from amount (48,000 tons) of cement said to be required.
Marklissa..	Germany.	1905	427	147.7	83,700	$595,000	Curved to R = 427 max. thickness 124.8 ft.

The following is a partial list of recent German dams, their magnitude in cu. yd. of masonry and total cost of the reservoirs. It is taken from a list which appeared in *Engineering Record*, July 19, 1913, in connection with an article on flood prevention and water conservation measures in Germany.

Name	Date	Cu. yd., Masonry	Total Cost	Reservoir Capacity, Cu. Ft.
Harzdorf	1902	20,918	$165,000	22,239,000
Greenwald	1906–08	56,218	540,000	95,310,000
Friedrichswald	1902–06	54,911	360,000	70,600,000
Voitsbach	1904–06	15,690	94,400	8,825,000
Muhlscheibe	1904–06	20,918	123,000	8,825,000
Gorsbach	....	41,837	206,000	17,650,000
Mohne	1912	344,500	5,000,000	4,590,000,000
Helienbecker	1894–96	11,700	67,300	15,900,000
Fuelbecke	1894–96	20,000	79,750	24,700,000
Hasperbach	1901–04	74,500	507,500	72,450,00
Ennepe	1902–04	21,300	716,500	363,500,000
Verse	1902–04	31,350	179,000	58,300,000
Glorbach	1903–04	45,700	216,000	74,200,000
Jubach	1904–05	36,600	161,600	37,100,000
Henne	1901–05	140,000	805,000	388,300,000
Oester	1904–07	68,000	429,000	109,600,000
Lister	1913	140,000	1,010,000	660,000,000
Neye	1907–08	?	408,000	211,800,000
Urft	1900–04	185,650	1,000,000	1,606,000,000
Eder	....	392,220	4,745,000	7,144,720,000
Setienberg	1905–08	88,200	68,300	40,650,000
Wolfesgrund	1905–07	26,200	125,000	27,930,000

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