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Control and Inspection of Earthwork Construction

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Control and Inspection of Structural Earthwork Construction

George E. Thomas, PE

A. Principles of Construction Control

1. General. In many types of engineering work, structural materials are manufactured to obtain certain characteristics; their use is prescribed by building codes, handbooks, and codes of practice established by various engineering organizations. However, for earth construction, the common practice is to use material that is available locally rather than specifying that a particular type of material of specific properties be secured. A variety of procedures exists by which earth materials may be satisfactorily incorporated into a structure. When earth is the construction material, personnel in charge of construction control must become familiar with design requirements and must verify that the finished product meets the requirements.

Design of earth structures must allow for an inherent range of earth material properties. For maximum economy, tolerance ranges will vary according to available materials, conditions of use, and anticipated methods of construction. A closer relationship is required among the operations of inspection, design, and construction for earthwork than is needed in other engineering disciplines. Construction control of earth structures involves not only practices similar to those normally required for structures using manufactured materials, but also the supervision and inspection normally performed at the manufacturing plant. In earthwork construction, the processes by which an acceptable material is produced are performed in the field.

Large earth structures are usually built by a contractor. The basis of the contract is a set of specifications that has a schedule of items of work. Through information provided in specifications, the contractor proposes prices for performing the items of work that, when accepted, become a part of the agreement between the owner and the contractor. The primary functions of the construction control organization are to:

- Ensure the structure is built according to specifications.
- Report and document any changed conditions.
- Certify to what extent the items of work have been completed.
- Test for compliance with requirements.
- Determine payment due the contractor.

Although Engineers prepare plans and specifications defining work to be performed, once a contract is signed, the Engineers or there representatives have no right to change contract requirements, and the contractor has no right to change unit prices. A condition may exist in the field that is different from what was anticipated to exist during preparation of the specifications. If changes to designs are necessary, a contract modification is agreed upon by both the contractor and the owner.

Specifications requirements for earth structure construction may be grouped into two types: those requirements based on performance and those requirements based on procedures. This distinction must be clearly recognized. The requirement for earth fill in a dam embankment is usually based on both a minimum procedure and on performance. If the two requirements prove to be incompatible, the performance is adjusted to obtain compatibility. The desirability of a performance-type requirement is recognized; however, the present state of knowledge of soil behavior, the complexity of specifications required, and the extensive testing requirement make this type of specification very expensive for some types of noncritical earth structure construction.

When developing specifications, it is very important to include the assumptions, confidence, and uncertainties inherent in the design. This allows both the contractor and owner a clearer understanding of the expectations of the designer and the inherent risks involved in the contract. It has been common practice to make all existing data available to bidders either by incorporation of the data into specifications or by referencing reports and making them available for review. In the past, it was practice to develop a separate document called "design considerations" which included the designers' assumptions, uncertainties, and requirements for successful construction. Engineers should attempt to include some of these considerations directly in the specifications. An example would be to include the reasoning for excavations shown on certain levels in the foundation. Acceptable foundation conditions should be given along with the reasoning and the geologic interpretations providing the basis for the excavation limits. Also, it is recommended that the owner, with support for the engineer, provide a prebid briefing to prospective contractors. During these briefings, it is helpful to show preconstruction excavations or to actually perform excavations such as test pitting for the prospective bidders.

Often, engineers do not perform extensive design investigations; however, design investigations are performed to collect design data related to constructability. Given the nature of the earth and its complex geology, it is not economical to attempt to perform extensive investigations to alleviate any change of condition. Contractors are encouraged to perform prebid investigations and investigations as construction progresses, as necessary, to avoid surprises during construction.

Specifications requirements also may be divided into two groups: definite requirements and those qualified by the phrase "as directed." The undesirability of the latter requirement is recognized and is avoided whenever possible. However, the "as directed" type of requirement is used in establishing minor dimensions in areas where investigations sufficient to establish such dimensions are not justified and when new conditions are encountered for which requirements have not been established.

Where the "as directed" requirement refers to dimensioning, either maximum and minimum dimensions or an average dimension is noted on the design drawings. If maximum and minimum dimensions are stated, the usual practice is to excavate to the minimum dimension; and where visual examination indicates the excavation is still within inferior material, excavation up to the maximum dimension is continued, if required, to reach a satisfactory foundation.

Where the "as directed" requirement refers to a new condition, the contractor and construction engineer should exercise joint effort to establish practical performance limits or to establish procedures that will produce satisfactory results where performance cannot readily be defined. Some experimentation is desirable, but this aspect should be small compared to the total requirement.

2. Inspection. The adequacy of construction can only be determined by visual examination, by measurements, and by testing. Inspection determines whether the requirements of plans and specifications are being satisfied, it does not determine what the requirements should be. An inspector, commonly the design engineer or his representative, are familiar or should become familiar with the specified requirements for work to be inspected. In determining whether work satisfies the requirements, an inspector needs to be familiar with how the requirement is defined.

For earth structure construction, dimensional requirements and quality requirements are evaluated. A requirement may be defined as a condition to be achieved from which certain deviations (plus or minus) are tolerable, or it may be defined as a limiting condition from which deviation is allowable in only one specified direction. Both methods are in common use. Standard practice is to allow a 10% plus or minus variation before a change order is required

Although a procedure is specified for some types of work (notably, earth fill in dam embankments) inspection may be made on the basis of satisfying a performance requirement. Basically, inspection determines only whether work is acceptable or not acceptable, but it also is desirable to determine the extent at what level the work is acceptable or unacceptable.

An inspector should be familiar with the various safety and health regulations for construction activities. Inspectors should stay fully informed on progress of work and on future programmed work.

To facilitate construction control, relationships between engineering properties test results and index test results are established wherever possible. In addition, visual observations of soil characteristics are correlated with both index and engineering properties. When noticeable differences exist between acceptable and unacceptable work or materials, sufficient testing is required to confirm that differences persist. As these differences become less obvious, the amount of testing should be increased. Testing should be sufficient to provide adequate quality control and to furnish the necessary permanent records.

It is impractical to completely test all the work performed. The usual procedure is to select samples of work or materials for testing that are representative of some unit of work or material. Accuracy of such procedures depends on:

- the relationship of sample size to size of unit it represents,
- procedures used for sample selection, and
- frequency of sampling.

For most earthwork construction performed, the ratio of sample size to unit of work or material represented is small, so special sample selection procedures are used. Since the principal objective is to ensure adequate work, samples are selected at random with a minimum recommended frequency.

3. Field Laboratory Facilities. The primary purpose of a field laboratory is to perform testing of construction materials. Test data serve as bases for determining and ensuring compliance with specifications, for securing maximum benefit from materials being used, and for providing a record of materials placed. Physical properties testing of earth materials during investigation stages may be performed in a field laboratory.

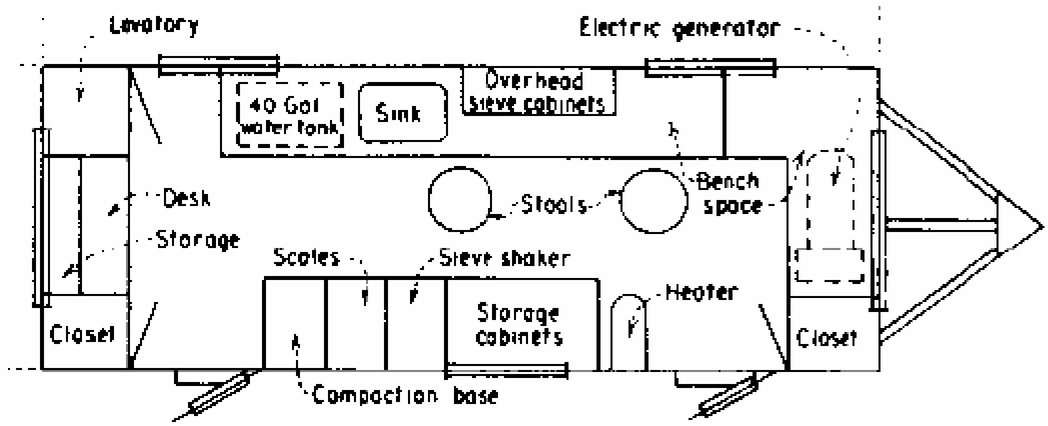
The size and type of field laboratory are dependent on magnitude of the job and on types of structures. When a laboratory building needs to be constructed, requirements usually will conform to one of the designs shown on figure 1 or a suitable modification. The Type C arrangement shown on figure 1 is appropriate for major earth structure projects. This design was used for dam construction in Idaho, which contains about 14 million yd^3 of embankment and 150,000 yd^3 of concrete. It was used in dam construction in California, which has about 30 million yd^3 of earth embankment and 100,000 yd^3 of concrete. The laboratory shown on figure 1 as Type B is applicable for projects where facilities are required for both earth and concrete testing of moderate scale. This type laboratory was used in a dam construction in North Dakota, which has about 1.4 million yd^3 of earthwork and 10,000 yd^3 of concrete. The small mobile or stationary laboratories shown as Type A on figure 1 are suitable for small projects, as a satellite laboratory on a large earth structure project, or on projects divided into several work divisions. They may be truck-mounted or trailer units. An example of a mobile laboratory is shown on figure 2.

At times, the type of laboratory suitable for a particular job may be difficult to determine. The main factors in determining the size and number of laboratories are size of contemplated work, concentration of work, and complexity of materials to be tested. In most cases, concrete and earth materials testing are combined into the same building. However, for small satellite control laboratories, separate facilities for earthwork control may be desirable. When earthwork is concentrated at one location, as for a dam, the project laboratory can be erected near the worksite so that laboratory facilities are immediately available for necessary control work. When earthwork is spread out over long distances, as in the cases of canal and road construction, testing facilities in addition to those at the main project laboratory must be provided near the work. Some projects employ a utility-type vehicle or an equivalent equipped with the necessary testing equipment as shown in figure 2. Other projects have used small skid-mounted buildings. As work progresses, these buildings are towed or hauled to new locations. Other projects have used large portable boxes where equipment can be stored. For testing work in very dry or rainy weather, sheltered facilities are advantageous so soil tests can be made without objectionable soil moisture changes.

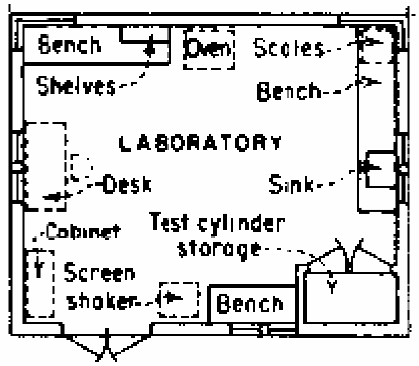
To ensure acceptable accuracy of test results, not only must equipment be maintained to a known accuracy, but the laboratory testing environment requires efficient control. Laboratory areas, where permeability and hydrometer analyses are performed, should be maintained at a comfortable, uniform temperature with a variation in temperature not to exceed ± 5 °F; otherwise, test results may be erratic and questionable. Usually, the most satisfactory method of temperature control is a central forced air system where air is heated or cooled (as required), filtered to remove dust, and returned through ducts to the various parts of the laboratory. Laboratory areas receiving heat from other sources such as concentrated sunlight, radiators, ovens, or hotplates should be avoided for tests where temperature change would adversely affect test results. For example, a hydrometer analysis would be adversely affected by a heating radiator if a constant temperature water bath is not used.

Some types of balances are sensitive to air currents. These balances must be either properly shielded or located in areas where air currents (from the heating or ventilating system) do not interfere with their proper operation. If possible, a high relative humidity should be maintained in areas where undisturbed soil samples are stored.

Operations that produce dust, such as sieving, processing, or pulverizing, should be conducted in an area separate from the main laboratory; an adequate system should be provided for removing dust from the atmosphere either by filtration or exhausting to the outside. Noisy operations such as sieving, compacting, and maximum index density testing should be conducted in rooms separate from the main laboratory because of their adverse effect on personnel and on other laboratory operations. When exposed to noise levels above 85 decibels for over 15 minutes per day, proper hearing protection must be worn. This noise level is commonly attained during the sieving of soils containing gravel.



(TYPE A)
(MOBILE)



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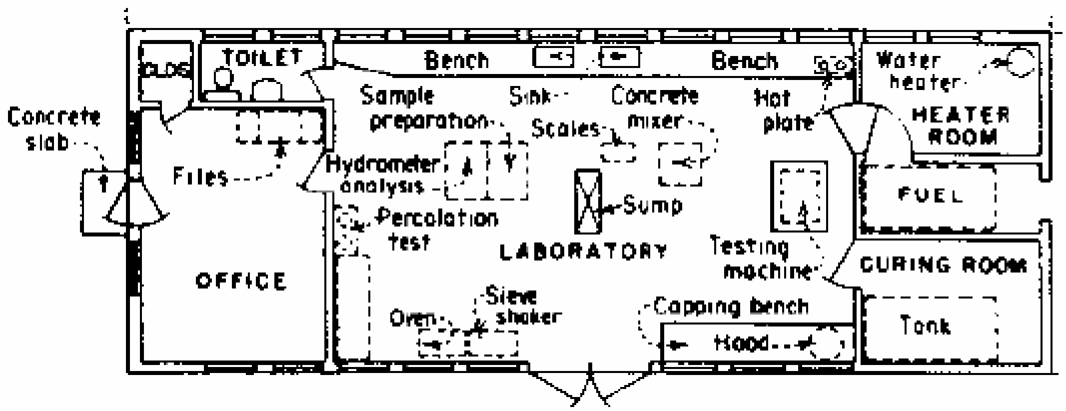


(TYPE B)

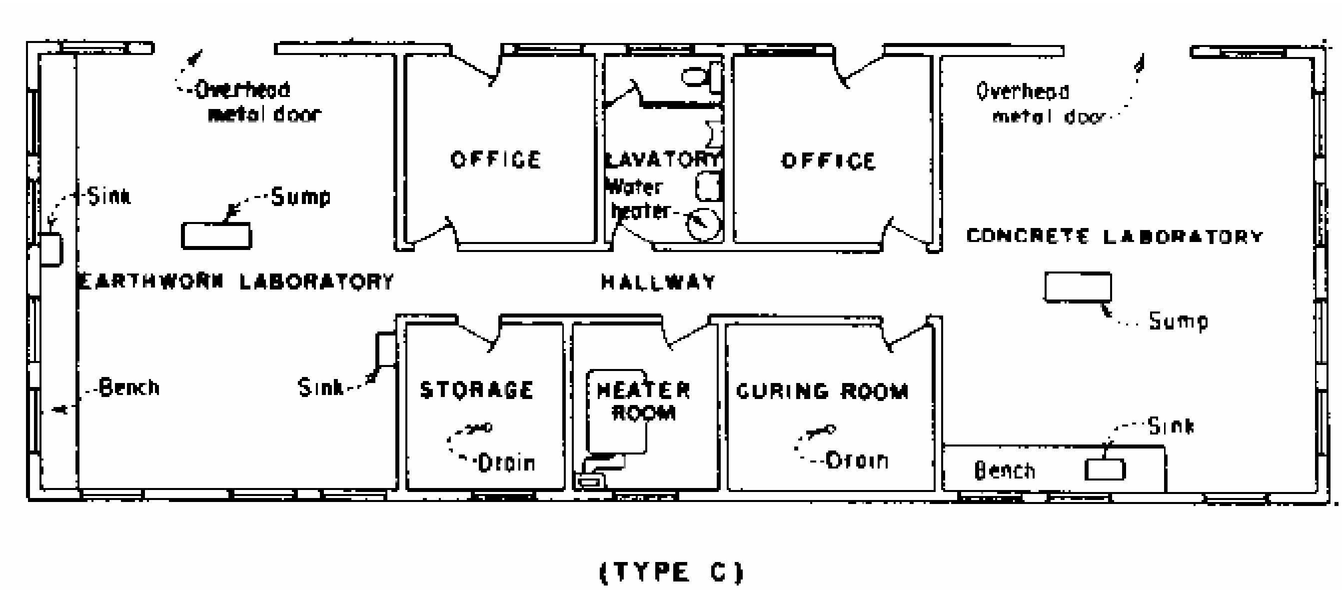


Figure 1, Examples of floor plans for field control laboratories.

a. Semimobile trailer unit used as a field laboratory.

b. Type V laboratory—vehicle equipped for on-site earthwork control tests.

Figure 2, Typical field laboratories.

4. Laboratory Data. Data forms generated in daily operations of field laboratories are important technical and legal documents, they must be properly handled and processed.

Proper data recording means not only writing a measured or calculated number in the correct location, with the appropriate number of digits, but also completing the test identification information on each data form. Observations made during the test that might have an effect on test results must be noted. Erasures should not be permitted on data forms. If a value has been erroneously recorded or calculated, a line should be drawn through the incorrect number, and the correct value is then written above or next to the original value.

All data calculated or transferred from another source must be checked. It is important that the one who checks data uses the same steps or method of computation as the original data reporter. Numbers calculated by "chain" computations (with a calculator or computer) may result in different values than those obtained when values are rounded for each entry and the rounded value is used in subsequent calculations.

When a numerical value is to be rounded to fewer digits than the total existing digits, rounding should be done as follows:

A computer or calculator may display the answer to a computation as 10 digits, and the answer is to be recorded to 2 digits. For example, the number 2. 3456789 would be rounded to 2.3; the first digit dropped would be the 4; other examples are:

2.49999 to 2.5
2.55555 to 3
2.49999 to 2
2.50000 to 3
2.55555 to 2.56

This same policy (rules) applies when rounding a number with many digits to a number with few digits.

The above examples of exactly 5 or 5 followed by zeros are rounded differently than indicated in some references. These references indicate the number is to be rounded to the closest even number. That is, 2.50000 would be rounded to 2 and not 3. Unfortunately, calculators and computers do not follow this rule and always round up. Recognizing the universal use of calculators and computers, the policy as stated should be followed.

5. Reports. A record of construction operations should be maintained as they are indispensable when repair or modification of the structure is required in the future. Also, a record is necessary when claims are made by either the contractor or the owner that work required or performed was not according to the contract. Recorded data are beneficial in improving engineering knowledge and practices for future work. Basic documents of the construction record are:

- Plans and specifications
- Adopted modifications that were considered to come within the terms of the contract
- Amendments made to the contract as extra work orders
- Orders for work changes
- Contractual protests
- Results of: Data sheets, equipment literature, and calibration records must be filed for easy and ready retrieval any time during construction.

To ensure that a proper record of construction is developed and available, various periodic reports are required. By reviewing these reports, supervisory personnel can determine whether proper performance is being achieved or whether deficiencies or misunderstandings exist. Necessary corrections can be made quickly on the basis of such reports. The progress report permits coordination of various operations required for servicing a contract to be performed in a timely and efficient manner. Reports are made of every test performed in the laboratory and in the field.

- Inspectors make daily reports concerning adequacy, progress, and comments on decisions. These daily reports may be of vital importance in subsequent actions.
- Administrative personnel make monthly reports on quantity of work performed, contract earnings, safety, employment records, and various other statistical information as required.
- Intermediate and supervisory personnel summarize these basic data periodically and make frequent informal and monthly formal reports, as required, and they include reports of all decisions made on controversial matters.

Specialized reports on grouting, pile driving, instrumentation, soil-cement construction, etc., are required as needed. These "technical" or "specialist" reports contain:

- Tabulations of test results
- Statements of progress
- Amount of work performed
- Any questions concerning interpretation of requirements or test results
- Descriptions of abnormal conditions or methods that affect either quality or quantity of work accomplished

Copies of all these reports are sent to the engineering office for immediate review by specialists so assistance or advice concerning performance of work or testing can be provided promptly if necessary.

Upon completing an earth dam embankment or other major works, a Final Construction Report (summarizing the work accomplished) should be prepared. In special cases, additional summaries or reports may be required.

The narrative summary of these reports should be restricted to matters of special importance or interest relating to technical control exercised in earthwork construction. All summaries should describe any difficult or unusual experiences encountered during the reporting period. Types of information desired are exemplified in the following list. Of the items listed, only those that warrant reporting should be included, such as:

- Description of foundation conditions encountered and methods, procedures, and equipment used in: preparing the foundation, stripping, excavating, dewatering and unwatering the foundation, placing piling or caissons, consolidating soils in place, etc.; also, methods for protecting the foundation surface prior to placing materials such as installation of drains, wells, filters, geosynthetics, concrete blankets, sealers, etc. Report unusual or unexpected foundation conditions encountered and methods of treatment.
- Describe preparation, clearing and stripping, irrigation, and excavation operations in borrow pits, required excavations, quarries, or other sources of earth materials destined for the construction. List methods used for adding or removing moisture, separating, excavating, and hauling. Indicate sources and delivery means of imported materials such as filter sand, lime, cement and pozzolans, etc. Also, describe borrow area drainage and restoration procedures. Report any difficulties or unusual events and remedies.
- Describe placement operations for embankments or other earth fills, earth linings, structure and instrumentation backfill, pipelines, and other earth structures. The narrative shall include descriptions of: equipment used (particularly the rollers); other compaction or consolidation equipment; methods and equipment for separating, scarifying, mixing, blending, and controlling moisture on the fill; finishing surfaces; placement of riprap, rock blankets, filters, cover materials, and gabion installations; final cleanup, landscaping, and seeding. Include the quantities of earth materials placed in each fill area, length of pipe backfilled, volume of soil-cement placed, etc. Also include the elevations reached for the various zones of major structures. Report unusual or difficult experiences or conditions.
- Where necessary to clarify subjects emphasized in the narrative section, suitably captioned photographs, drawings, and sketches should be attached to the summaries. Additional media such as films, video, etc., may be useful.

B. Earthwork

6. General. Differences in design requirements and construction procedures have led to classifying fill construction into three types: (1) embankments, (2) linings and blankets, and (3) backfill. Usually, division lines between fill types are not distinct. Embankment construction applies primarily to laterally unsupported fills built on top of the natural ground surface; however, refill of cutoff trenches or key trenches is included as embankment construction, especially regarding construction control. Lining and blanket construction applies mainly to relatively thin sheets of fill spread over an area of natural ground, excavated surfaces, or embankment. Backfill refers to refill of excavations below the ground surface or earth placed in confined spaces and against rigid structures.

Based on the amount and kind of work required, each of these groups is further divided into several types. Because of the indistinctiveness of definition, where two or more different types of construction are required involving separate contractual pay items, it is customary to establish arbitrary boundaries for the different types of work. Separation may be definite, or an overlap may exist. The lines of distinction will vary from job to job, so reference to the particular specifications involved is required in all cases.

7. Embankment. Engineering properties of soil can be changed, and often improved, by:

- Selecting
- Controlling moisture content
- Mixing
- Stabilizing using various admixtures
- Compacting

a. Types of Embankment. In the order stated, each of those operations is successively more expensive to perform. Although engineering properties are improved and made more uniform, some of these operations may not be justified for all structures.

In construction, any of the following types of embankment construction may be specified:

- dumped fill,
- hydraulic fill,

- selected fill,
- equipment-compacted embankment,
- rolled earth fill,
- vibratory-compacted embankment,
- blended earth fill,
- modified soil fill.

b. Dumped Fill. The simplest construction operation of moving material from excavation and depositing it in a fill to lines and grades is called dumped-fill construction. This type of embankment is used to construct minor roads, canals, and laterals when necessary engineering properties can be developed in the available soil without special effort. Although selection and distribution of material are not specifically required, dumped fills should be free of tree stumps and other organic matter such as trash, sod, peat, and similar materials. Rocks, cobbles, and similar material should be distributed throughout the section and not nested or piled together. For the fill to be reasonably uniform throughout, materials should be dumped in approximately horizontal layers. "End dumping," a process by which fill material is pushed off the edge of the fill and allowed to roll down the slope, is objectionable because it fosters material segregation and should be avoided wherever possible. Dumped fill is often further classified as dragline-placed fill, truck-dumped fill, or scraper-placed fill. If there is traffic over the fill area during the construction, either by construction equipment or otherwise, it should be routed to distribute compaction as much as possible.

On canal construction, a further requirement is to place the finer and more impervious material on the water side of the embankment. On road work, the gravelly material should be in the top of the fill, and large rock should be well buried. Inspection consists of visual examination to ensure that the above requirements are satisfied and that dimensional requirements are attained. Laboratory testing of dumped fill construction is not required except for record purposes.

c. Selected Fill. Commonly used selected fill types include selected impervious fill, selected sand and gravel fill (pervious fill), rockfill, and riprap. Selected fill is a dumped fill constructed of selected materials; occasionally, compaction is required. This type of construction is widely used where one engineering property is more important than others, and soil with a specific property can be secured by selective excavation. Accordingly, selected fill may be specified to satisfy one of several engineering properties. Construction and inspection requirements vary according to the engineering property being emphasized.
The selected impervious fill type of embankment is used in canal construction when selective excavation of the canal prism will provide a superior structure with little extra effort. Excavation from borrow pits and short hauls may be required, but benefits derived solely from selection usually do not warrant the cost of securing a better material from a more distant source.

Selected sand and gravel fill is specified to improve stability, to prevent frost heave, to provide improved wearing surface on roadways, to prevent wave erosion of underlying embankment, and to provide for removal of seepage water without "piping." Benefits derived from this type of construction often justify securing satisfactory material from a considerable distance, and sometimes processing to improve gradation is warranted. In choosing material for sand and gravel fill, gravel is the more important component, and a well-graded, rather than a poorly-graded, material is preferred. Other characteristics may be required for filter materials. Visual inspection should be confirmed with an occasional gradation test.

Riprap and rockfill are used primarily for surface protection. Riprap is used to protect against erosion by flowing water and from ice and wave action and to protect against rain and surface runoff. Rockfill is used to protect against rain and minor surface runoff and to provide stability to a fill structure; to some extent, it is used as a substitute for sand and gravel fill in dams. Durability and gradation are important requirements for riprap. These are also desirable characteristics of rockfill, but the tolerance ranges are much broader.

d. Equipment-Compacted Embankment. Situations arise where selected fill does not produce an adequate structure, and the addition of compactive effort by routing of equipment will produce an acceptable fill. This type of construction is mostly used on canal embankment and road construction. Specifications for construction of these features usually do not require a definite degree of compaction. Control of moisture in the material may or may not be required. Reference should be made to pertinent specifications. Separate pay items may be used for addition of water. The engineer in charge determines the amount of water to be added largely on the basis of volume of fill to be constructed and the nature of materials being used. Sometimes, visual inspection is supplemented by laboratory tests.

e. Rolled Earth fill. Improving engineering properties to the maximum practical extent by selection, compaction, moisture control, and special processing is generally justified in construction of earth dams and in canal embankments. A failure could result in the loss of life or substantial property damage, and conditions could develop that would cause excessive water loss or expensive maintenance costs. Where pertinent, procedures and equipment are specified to ensure development of desirable engineering properties to the maximum practical extent.

Materials selection will have been based on preliminary investigations, that is, borrow areas will have been designated. Preliminary investigations should have disclosed the nature of the average materials and the probable range of variation. Field personnel in charge of construction operations should review preconstruction investigation results, perform additional exploration and testing (if necessary), and learn to recognize the kinds of material acceptable for the adopted design. Since earth dams and major canal embankments are designed to accommodate materials available in the vicinity, the materials used for various purposes will differ from site to site. If, as a result of more detailed exploration or in the process of construction, materials are encountered whose characteristics differ appreciably from those anticipated, the construction methods and procedures may have to be changed.

For embankment dams, a specific moisture range and density are generally specified. In specifying compaction, requirements will include the general type of roller to be used, the thickness of lifts, and the number of passes. These requirements are based on extensive experience and statistical data and will produce a satisfactory fill when placed using good construction procedures. Specifications include requirements for removal of oversize particles and that the material be homogeneous in texture; that is, free from lenses or pockets of material differing in gradation or classification from the average material. Soil moisture content must be controlled to obtain maximum benefit from compaction. Specifications require water content be uniform throughout the layer to be compacted and that the layer be as close as possible to the water content that results in most efficient densification of material to be compacted. For the specified compactive effort, for earth fill in dams, this water content is often slightly less than optimum water content as determined by the laboratory compaction test.

Inspection is performed to determine whether the:

- Material is uniform and free from oversize particles.
- Compaction equipment complies with specifications and is maintained in working order.
- Thickness of lifts and number of passes are according to specifications.
- Moisture is uniform within the layer.

The inspector should determine that moisture content is correct by performing manual tests to observe how the material compacts in the hand. Inspection observations will be supported with field and laboratory tests to determine the degree of compaction and variation of water content from optimum and to determine permeability.

f. Vibratory-Compacted Embankment. Compaction by vibration is usually specified in the construction of earth dams to improve the engineering properties of strength and consolidation in comparatively permeable soils; that is, clean sands and gravels (GW, GP, SW, and SP). However, several other methods of compaction will produce satisfactory densification when permeability is not a concern. Some of these methods are specified both for highway and airport construction.

In constructing an earth dam, the contractor may propose some method other than vibratory compaction to densify permeable embankment materials. For those accepted proposals, letter instructions covering construction control must be issued by the engineer of record as required.

In selecting filter and drain material to be compacted by vibration, emphasis should be to eliminate soil contamination by excessive amounts of fines minus No. 200 sieve size material. As a result, filters and drains are designed with less than 5 percent fines. Field inspection, therefore, will be directed toward removing overburden fines, avoiding silt and clay lenses or pockets, and preventing excavation into less permeable materials below the sand and gravel deposit. Thickness of placed embankment layers is commonly adjusted to avoid a requirement for removal of oversize material, and maintaining soil uniformity does not require the attention that impervious fill construction requires. However, coarser material should be placed toward the outer slopes. Where vibratory compaction is specified, the material source will be designated, minimum size tractor or vibratory roller described, lift thickness and number of passes enumerated, and moisture requirement defined. Compaction by a vibratory roller or track-type tractor depends on the vibration produced by the equipment in operation. A secondary benefit results both from size and mass; for example, thicker layers can be compacted with larger equipment.

Speed of the vibratory roller is considered beneficial in that the effect of increased vibration with high speeds more than compensates for any detrimental effects of short period application. Maximum compaction of sands and gravels is obtained by vibration when the soil is either completely dry or thoroughly wetted without being saturated. The wet condition is specified because satisfactory density is more readily obtained, and field tests are more likely to be reliable with wet compaction. However, under certain conditions, dry compaction has been permitted. In these circumstances, extra compactive effort is necessary in lieu of thorough wetting. To secure a thoroughly wetted material, excavation of materials from below water table has been permitted in some instances. This procedure is successful when small-size excavation equipment combined with a relatively slow placement rate is used. With large capacity equipment and a rapid placement rate, drainage of excess moisture may be too slow to permit satisfactory compaction.

Inspection consists of noting that proper quality of material is used and that specified thickness of lifts and number of passes are obtained. Water content is sufficient if free moisture appears in the equipment tracks immediately following passage. With proper water content, the compacted fill will appear firm and solid. If the fill remains soft, moisture is too great; if the fill is fluffy, water content is insufficient.

g. Blended Earth fill. Circumstances occur in which two individual materials do not have adequate engineering properties but, when combined, produce a satisfactory material. Other cases arise in which a material having inferior properties may be combined in a mixture with another material of adequate engineering properties to increase the quantity of satisfactory material available. When the materials to be blended occur as strata (one above the other) in the same borrow pit, excavation by shovel or wheel excavator can readily blend them together with little extra effort or expense. In this case, construction control will require maintaining the height of cut to obtain the necessary proportions of each type material and to ensure that the two materials are being blended. With dragline excavation, satisfactory mixing can be accomplished, but greater attention must be given to the operation because the tendency will be to remove material in horizontal cuts. Several types of excavating equipment, such as wheel excavators and belt loaders, have excellent capabilities for blending vertical cut material (see fig. 24).

When scraper excavation is used, the difficulty in attaining a satisfactory mixture is increased; frequently, supplemental mixing is required on the fill by plows, discs, rippers, or a blading operation. Excavation is performed by making a slanting cut across the different types of materials; care must be exercised to effect the proper proportions of each type of material in each scraper load. Loading the materials in an upslope direction usually gives a more effective mix of materials than loading in a downslope direction. A common practice is to spread the materials in half-lift thicknesses to improve uniformity and minimize requirements for mixing on the fill.

The cost considerations limit the procedure for blending materials from separate sources to special situations such as blanket and lining construction. First, one of the materials is stockpiled upon the other so excavation can be made across the two materials, or the materials are placed in thin layers on the fill; then, they are mixed by blading, plowing, or similar procedures before being compacted. Mixing machines may be specified to obtain an optimum mixture for earth linings.

h. Modified Soil Fill. A fill using a soil that has been modified by addition of minor quantities of a selected admixture prior to compaction is called a modified soil fill. The most commonly used additives are lime and cement. The use of modified soils should be considered in lieu of replacing poor soils with selected material from a distant source. For example, to construct a switchyard fill, lime can be used to modify expansive characteristics of a clay soil.

Usually, conventional methods are used to spread and compact the modified fill. In addition, some type of equipment must be adapted to distribute and mix the additive. This equipment requires initial calibration and periodic checks during construction to ensure that the proper amount of additive is being applied. Special tests of the modified soil may be required to verify the amount of additive, and visual observations should be made to ensure that the additive is uniformly distributed. Construction control testing of placement moisture and density is required.

Inspection requirements will be comparable to those required for concrete or other specialized materials, and extensive testing should be anticipated. Considerable experimentation may be required to develop suitable construction procedures.

i. Hydraulic Fill. Most methods of embankment construction all involve control of water content. Some situations require placement of fill material under conditions of excess water content. These situations may involve excavation and transportation of the material by use of flowing water. The more common procedures involve only the placement of material in still water or by the process of washing material into place or densifying it with a stream of water.

Terminology for the different types of hydraulic construction is not standardized. In general, the term "hydraulic fill" is applied to the complete operation of excavation, transportation, and placing by flowing water. When the hydraulic operation is confined to placing, it is described as semihydraulic construction. The term "puddled fill" is applied to special types of semihydraulic fill, and the term "sluiced fill" is used for material washed into place.

There are several severe limitations to hydraulic fills which may make them undesirable for embankment construction. Uniform granular or cohesionless hydraulic fills, without active densification measures, will result in very low degrees of compaction. For undensified cohesionless soils, the typical relative range is 30 to 60 percent. There have been numerous examples of severe distress to hydraulic fills, such as embankments, quay walls, and tailings piles under earthquake loadings. One such example is earthquake-induced liquefaction experienced at Lower Van Norman Dam during the 1971 San Fernando earthquake. Puddled, sluiced, or hydraulically placed, fine-grained soils will be weak and highly compressible. Many years may be required before fine-grained soils are improved by consolidation. In large embankments, the use of high water content, fine-grained soils may result in high pore pressure generation and reduced shear strength resulting in unacceptable deformation or shear failures. Many hydraulic fill structures were built prior to the 1930s. Due to the limitations listed above and the advent of modern compacting equipment, hydraulic fill lost popularity after the 1930s.

Economical use of hydraulic fill construction depends on: (1) availability of a material that is readily sorted by the action of flowing water into a pervious material zone and an impervious material zone, (2) a large volume operation, and (3) a source of inexpensive power. This combination of conditions happens so rarely that control procedures for hydraulic fill construction have not been developed. In the event that hydraulic fill or semihydraulic fill is specified, special instructions will be issued by the Denver Office engineering staff.

Sometimes, puddled-fill construction is used where climatic conditions make moisture control operations impractical, for backfill around pipelines and structures, and in lieu of special compaction against rough and irregular surfaces. The puddling process consists of depositing the soil in a pool of water, stirring the soil-water mixture until a fairly uniform slurry is developed, and then allowing the soil to settle out of the mixture. Puddling is used mainly for soils of low permeability, such as silts and sandy lean clays, where high density is not necessary. Inspection is by visual examination to establish that the proper type of material is used and to ensure that thorough mixing is accomplished. Puddled fill has many undesirable properties. The slurry can consolidate over time, and unacceptable deformations and stress distributions can occur. The slurry is weak, and upon continued loading, high pore pressures can be generated, resulting in further loss of strength. These undesirable properties make the puddling method unacceptable for many structural backfill applications. Puddled clay core dams were often constructed in older dams in the United Kingdom, where wet clay was healed by foot or equipment traffic into thin central clay cores. The wet clay had very low hydraulic conductivity. Many of the dams have performed well, but there are concerns about stress distributions, cracking of shells, and hydraulic fracturing.

Sluiced-fill construction involves working a pervious material into place by a washing operation produced by the flow of a high-velocity stream of water. The method is used to wash sand and gravel or rock fines into the voids of a rock mass such as a rockfill. In lieu of tractor or vibratory rollers, the method may be used for compacting sand and gravel fill. Vibratory-compaction construction is considered superior to sluiced-fill construction; hence, the latter is usually restricted to areas inaccessible to equipment. Inspection consists of visual examination to determine that appropriate materials are used and that the washing operation is properly performed; that is, the materials are actually moved in the process. When sluiced-fill construction is used in lieu of vibratory- compaction construction, relative density tests are required to ensure adequate densification.

Compacted fill is sometimes specified to be compacted in a thoroughly wet condition using equipment such as pneumatic rollers, vibratory rollers, surface vibrators, or tractors. Soils placed by this method must have a relatively high permeability for proper densification. A density requirement is specified and is based on relative density.

Figure 3, A deposit of impervious material which overlies a deposit of pervious material.

8. Linings and Blankets.

a. General. In earth construction, many situations require constructing a relatively thin layer or blanket of selected material. These categories include:

- Layer of riprap on the upstream slopes of earth dams
- Sand and gravel blanket under riprap
- Rockfill blankets or blankets of gravel or topsoil on the downstream slope of dam embankments
- Filter blankets under the downstream portions of dam embankments and under the floors of concrete structures
- Impervious blankets on the floors of reservoirs
- Channel linings for canals
- Linings and covers for hazardous waste disposal areas

Base courses and surfacing on roads or highways, and ballasting for railways, are considered to be in this category. A common requirement of these types of blankets is proper selection; that is, locating and using suitable materials. Sources of material that have established characteristics and in ample quantity for the various blanketing requirements should always be determined before construction.

A requirement of all blankets is careful placement. Requirements may vary widely according to the type and location of blanket placement, but uniformity and thickness are extremely important in every case. On the basis of material used, blankets may be divided into four types:

(1) Rock.
(2) Sand and gravel.
(3) Silt and clay (impervious).
(4) Topsoil.

b. Rock Blankets. The rock fragments used in rock blankets should be essentially angular well graded from a maximum dimension equal to blanket thickness to about one-tenth of blanket thickness, and sound, dense, and durable. Elongated and thin slabs of rock are undesirable, but rock fragments whose minimum dimension is about one-third to one-fourth of the maximum dimension are not objectionable. Although angular rock is the most desirable, subangular and subrounded cobbles and boulders are commonly used. While rounded cobbles and boulders have been used as riprap in certain cases, their use is more appropriate for blankets on flat surfaces. Testing and placement of rock blankets are discussed in more detail in section 24 on riprap.

Grading of riprap and other large stone can be performed in accordance with ASTM procedure D-5519-94. The procedure allows for determination of grading by volume, mass, or a combination of volume and mass. On critical projects, it may be necessary to specify testing and provide for testing facilities such as concrete pads and scales. Generally, blasted rock that has an appreciable quantity of fragments near the maximum size requirement will possess a satisfactory gradation. The condition to avoid is an excess of small-size fragments. Fine material such as rock fines, sand, and rock dust, in a volume not to exceed the volume of the voids in the larger rock, may not be objectionable, depending upon the purpose of the blanket.

Soundness, denseness, and durability requirements may vary somewhat according to usage. The highest quality rock is required for protection of slopes in stilling basins. The soundness requirements for riprap on dams may vary somewhat according to reservoir fetch and operating characteristics; rock size, to some extent, can be substituted for high density. Protection of downstream slopes of dams can be accomplished with any rock that does not break down appreciably by normal exposure to weather. Shale is about the only rock that is unacceptable for downstream slope protection of dams.

c. Sand and Gravel Blankets. The characteristic of sand and gravel mixtures to allow passage of water, while at the same time preventing passage of soil grains, is extensively used in the design of water retaining structures. The properties of resistance to displacement by flowing water, resistance to wear from vehicular traffic, and maintenance of strength and limited volume change over a large range of water contents, make sand and gravel useful in providing surface protection to canal banks, roads, and working areas in the various facilities of irrigation and power projects. The wide range in gradation possible in sand and gravel mixtures, together with the wide range in structural materials to be protected, results in a wide range of acceptability for the materials used for sand and gravel blankets. However, blankets under the downstream portion of an earth dam embankment must be designed as a filter.

Pervious blankets under concrete structures usually are made as thin as possible. This condition makes processing of material from natural deposits almost a necessity so as to produce the specified gradation of material. Two-layer filter systems are frequently used.

Sand and gravel used as base courses and surface courses on roads, highways, and switchyards are normally processed material. Materials used for surface courses differ from pervious blankets for other uses in that a certain amount of clayey material is considered desirable to bind the material together and, in some instances, is actually added to improve the quality of surface courses.

Pervious blankets used to prevent erosion in canal channels require a material predominantly in the gravel range. Appreciable quantities of sand are undesirable, and, in some instances, removal of fine sand may be required. A sand layer may be required between a coarse gravel layer and the subgrade to prevent movement of fine subgrade soils through the gravel. Materials for a pervious gravel blanket and materials for an impervious blanket may be combined to provide an erosion-resistant impervious blanket. Usually, filter criteria are not important in impervious blankets.

Crushed rock may be used for any of the above-described purposes. Crushed rock is a processed material that can be manufactured to fulfill specifications requirements.

Sand and gravel or crushed rock blankets usually are densified by equipment compaction using vibrating equipment or operating smooth rollers. They may be placed either dry or thoroughly wetted. Inspection is commonly by visual examination, but gradation and relative in-place density tests may be required in some cases.

d. Impervious Blankets and Linings. Impervious soils are used on reservoir floors and in canal channels to reduce seepage. Material for these purposes must be:

- Impermeable
- Free from shrinking and swelling characteristics
- Resistant to erosion from flowing water
- Stable when placed on the sides of canals and reservoirs

Probably the best material for these purposes is a well-graded gravel with clay (GW-GC), which offers both impermeability and excellent erosion resistance. A clayey gravel (GC) material or a silty gravel (GM) material is also suitable. Materials in other soil groups can be used according to soil engineering use chart. When satisfactory soils are not available, processing soil by blending or protecting the fine-grained soil with a blanket of sand and gravel is necessary.

Blankets and linings to be permanently underwater need not meet erosion resistance and shrinking and swelling criteria. Blankets and linings placed on essentially horizontal surfaces need not possess high stability characteristics. However, an impervious blanket beneath a reservoir may require a filter beneath it to prevent piping of the blanket material into the foundation.

Thickness of impervious blankets is usually controlled by placement procedures. At high reservoir heads, thickness greater than the minimum required for construction operations may be required. Also, where some swelling or shrinkage is anticipated, thickening impervious blankets may be desirable so an effective thickness of blanket (unaffected by swelling or shrinkage cracks) is maintained. For canal linings, thickness generally is varied depending on design requirements; availability of lining material may affect thickness, and construction methods may need to be revised to attain desired engineering properties. Construction and inspection requirements are similar to those for rolled earth fill and blended earth fill.

e. Topsoil Blankets or Zones. Topsoil blankets or zones sometimes are specified for downstream slopes of dams so slopes can be seeded to protect the underlying zone against wind and rain erosion. Topsoil for restoring borrow areas or other areas,where existing topsoil has been destroyed or removed, is specified to fulfill ecological and environmental requirements.
Normal topsoil sources are materials stripped from required excavation for the dam and appurtenant structures and approved stripping from borrow areas. Generally, it is not feasible to borrow topsoil. Consequently, suitable materials from stripping, which would otherwise be disposed of as waste material, should be selected during excavation and, if necessary, stockpiled for later use.

Normally, specifications require a 1' normal thickness for topsoil blankets; thus, areas requiring topsoil should be brought to within 0.3 m of the prescribed final cross section at all points and finished smooth and uniform before topsoil is applied. Topsoil should be evenly placed and spread over the graded area and compacted in two layers.

Occasionally, for dams requiring topsoil, it will be advantageous and economical to specify a thin topsoil zone for downstream slope protection. A prime advantage is that excavated topsoil can be placed concurrently with the rest of the embankment, eliminating the requirement for stockpiling. When a topsoil zone is specified, construction and inspection requirements are similar to those for rolled earth fill.

In most cases, seeding of topsoil on downstream slopes is required; thus, topsoil should be selected from required excavation or from a stockpile so the most fertile soil is obtained. Topsoil must be free of excessive quantities of large roots, brush, rocks, and other objectionable matter. Topsoil should not be placed when the subgrade is frozen or in a condition otherwise detrimental to proper grading and seeding.

9. Backfill.

a. General. Backfill is earth fill in confined spaces, such as refilling operations about concrete structures. Refilling operations associated with embankment, construction work adjacent to the concrete structure and in other confined areas may be called special compaction rather than backfill. Backfill operations may be divided into three categories: (1) backfill, (2) compacted backfill of clayey and silty soils, and (3) compacted backfill of cohesionless free-draining soils.

b. Compacted Backfill. Compacted backfill covers two types of materials and compacting operations. The *first* is compaction of clayey and silty materials of low permeability. These soils are used for backfilling structures when seepage is to be limited or if drainage is not required. Normally, this type of backfill is compacted by tamping rollers when space is available or by power tampers in confined areas. Generally, the soils are compacted at optimum moisture content and to a given percentage of the laboratory maximum density.

The second type of compacted backfill uses cohesionless free-draining soils of high permeability. These soils are used around or under pipelines and structures when free drainage is desired or when a bedding or a foundation of low compressibility is required. The materials used must be free-draining sandy and gravelly soils. When high stability and low settlement are design requirements, this type of backfill is often preferred over compacted backfill of clayey and silty soils because of the ease and economy of placing, particularly in confined areas. For example, when good support is required under and around concrete pipe, the material can easily be made to flow and can be compacted to high density in the critical area beneath the pipe and between the pipe and trench surfaces, providing proper procedures are used for wetting and vibrating the material. A fill of clean sands or gravels is specified often for use under pumping plants and other structures when necessary to improve the bearing capacity of soft foundation soils. This improvement is accomplished by removing the soft materials and replacing with selected backfill to spread the load onto lower strata. Also, cohesionless free-draining soils are preferred as backfill adjacent to some structures when excessive surface settlements are undesirable or when space is limited. Generally, compaction of these soils is accomplished by pneumatic rollers, vibratory rollers, tractors, surface vibrators, or internal vibrators.

Proper selection of materials is particularly important when compacting using saturation and internal vibration. Excessive amounts of fines will fill the voids between coarser particles and will prevent drainage. Table 1 provides information for preliminary selection of soils suitable for compaction using saturation and vibration.

Table 1, General suitability of soils for compacted backfill by saturation and internal vibration

Soil type	Limitation[1]
GW, GP, SW, SP	All soil types are suitable. Fines in these soils are limited to less that 5 percent by definition.
GW-GM, GW-GC GP-GM, GP-GC	May or may not be suitable, depending on gradation and plasticity.
SW-SM, SP-SM, SP-SC	Test section may be required. Fines are limited to 5 to 12 percent by definition.
SM, SC	Normally unsuitable.

[1] Fines are particles smaller than U.S.A. Standard 75 µm sieve (No. 200).

10. Excavation. Materials for embankment or lining construction are obtained either from required excavations or from borrow pits. Materials from required excavations should be incorporated into embankment or lining construction to the maximum extent possible. When materials from required excavations can be used, the same practices should be followed as in borrow-pit excavations.

Organic material such as plant growth and decaying vegetable matter should be removed from the surfaces of borrow pit within borrow areas before initiating excavation. Depth of stripping varies according to the nature of ground cover, from 2 or 3" for prairie grasslands to 2 or 3' in valley bottoms and forested areas. The removal of all material containing grass and tree roots is not necessary. The fine hair roots of grass and tree roots less than 1/4 " in diameter are normally considered not sufficiently detrimental to justify wasting material containing minor amounts of this matter. Excavating an area greater than required, in one construction season, is inadvisable because of recurring weed growth. The minor amount of weed growth that may develop during the latter part of a construction season on a stripped area is usually considered unimportant.

Occasionally, deposits of impervious soil are covered with a layer of boulders; often, sand and gravel deposits are covered with a layer of silt and clay. Such a deposit is shown on figure 3. A thorough materials investigation for an earth fill dam resulted in the most economical benefit to use the upper clay-silt-sand portion of the deposit for the impervious zone and the lower silt-sand-gravel portion for the pervious zone, thus eliminating the need for another borrow source with its subsequent additional development costs. When impractical to salvage upper layers, they are classed as stripping and wasted even though the layers may be thicker than is normally considered applicable to stripping.

In highway and canal construction, where the normal method of excavation is by scraper, a lower limit to the depth of cut is not required. In all other excavations, cut depths less than 4 to 5' are avoided for economic reasons unless deficiency of material dictates smaller cuts. Except in situations where separation between two different types of material is desired or in which controlled blending is required, maximum depths are usually limited by the range of excavating equipment. However, the maximum depth of cut may be limited by the presence of bedrock or hardpan and, except for sand and gravel excavation, by location of the water table. In stratified borrow areas, excavation must be made so that every load delivered to the point of use contains a mixture of the full designated depth of cut. Extra mixing on a stockpile or on the fill may be required.

C. Foundations

11. General. If conditions in soils supporting a foundation could be completely described as a result of subsurface investigations and designs were prepared accordingly, field inspection, during construction, would be reduced to merely determining that dimensional requirements of designs were satisfied. However, even a thorough investigation program leaves the majority of foundation soils and rock unexplored.

Because of the variety of conditions that may be encountered in natural soil deposits, a complete set of guidelines for judging adequacy of foundation soils is impossible to provide. Individual judgment based largely on experience must be used. The first objective in developing experience is to enhance one' s ability to discriminate between sound and unsound foundation soils. In practice, this means classifying foundation soils as adequate, inadequate, and questionable for the type of structure being built. Initially, most cases seem to fall in the questionable category; but with increasing experience, the number of cases decrease. To supplement judgment, test procedures have been developed for evaluating foundation soils. Such tests should have been performed during the investigation program; comparison by means of index tests and visual examination between tested and untested areas will usually suffice to establish adequacy. Additional tests may be required during construction.

Conditions for which adequacy of foundation soils should be established include bearing capacity, stability, settlement, uplift, deterioration, and permeability. The degree to which each of these items is important depends on the nature of the proposed construction, character of foundation soils, and structural characteristics of the foundation.

12. Bearing Capacity. Foundations for rigid structures are usually evaluated on the basis of bearing capacity. Bearing capacity in this sense involves both shear strength of the soil and its consolidation characteristics. Bearing capacity may be determined in the laboratory by shear and consolidation tests on undisturbed samples or approximated in the field from in-place dry density tests, standard penetration tests, vane shear tests, cone penetration tests, borehole shear tests, or pressuremeter tests. The design of rigid structures should include the type of foundation treatment required based on information available from the investigation program. Where foundation soils appear to be the same as those disclosed at locations of the investigations, they may be assumed to be equally competent. If soil conditions are disclosed that are observably different from those disclosed by previous investigations, the lateral and vertical extent of this different condition should be determined before construction continues.

The inspector should be aware that a pocket or lens of harder or firmer material in foundation soils can be equally as detrimental as softer deposits. Therefore, common practice is to specify that if such material is encountered, it will be removed to some depth and be refilled with compacted soil to provide a more uniform soil deposit at foundation grade. This requirement should include beds, layers, or lenses of indurated gravel or sand, shale, indurated clay, hardpan, caliche, and other erratic cemented deposits such as calcareous or siliceous material. The most desirable soil deposit for a structural foundation has a uniform character, and a moderate amount of additional excavation to develop such a condition is usually justifiable.

13. Stability. Excavation for a foundation can be divided into two parts, the base and the side slopes. Except for structures such as chutes of canals and spillways, the load of the structure rests on the base of the excavation and settlement usually limits development of full shear resistance of foundation soils. Consequently, stability of the base of an excavation may not present a problem. However, foundation excavations adjacent to bodies of water or directly above water bearing strata should be investigated for loss of support caused by artesian pressure.

For most construction, cut slopes should be determined by practicing geotechnical engineers using data available from the investigation program together with appropriate analyses and design experience. Federal, State, or local requirements must be considered in the selection of excavated slopes. Use of slopes not previously analyzed or pre-established should be discouraged because of the many variables involved in determining stable slopes such as soil type, ground-water conditions, depth of cut, and intended use. Both experience and engineering analyses are required for this determination.

In determining stable slopes, a great deal of information can be gained from examining previously excavated slopes and natural slopes in similar soil in the same geographic area. Stable slopes in sand and gravel are independent of height of cut, but are very dependent on local ground-water conditions. Likewise, safe cuts in saturated clay are dependent on cut slope height and relatively independent of slope. Because most soils are combinations of sand, gravel, and clay, safe cuts will be controlled both by height and by slope. Evaluation of natural slopes should consider that the safety factor against failure is variable and that any increase in slope (i.e., placing fill at the top of a slope or excavating at the toe of the slope) results in reducing the safety factor.

Therefore, a natural slope should be steepened as little as possible.

The presence of water has a significant effect on slope stability. This can be in any combination of seepage, rainfall, or water being conveyed or stored. An example of failure of a natural slope (landslide) is shown on figure 4. Excessive moisture was allowed to enter the soil, and removing soil at the toe of the slope for constructing a stilling basin may have triggered the slide. Where an excavation cuts across the ground-water table, sloughing may occur. To overcome this condition, the usual practice is to remove ground water from the area before excavation by any combination of well points, deep wells, or sump pumps. Figure 5 shows well points and sump pumps used in trench excavation; figure 6 shows removal of ground water using drainage ditches and sump pumps. Damage to a slope may result from high hydraulic gradients produced by seepage. Where pumping is not practical, excavations may be accomplished by making repeated cuts, which result in gradually lowering the hydraulic gradient as ground water is drained away. The success of any method is dependent on a thorough knowledge of ground-water behavior and of local soil conditions.

Where permanent cut slopes intersect the ground-water table, some type of treatment is usually necessary to prevent slope failure or sloughing. Required treatment involves moving the free surface of the water table back from the face of the slope or controlling the water as it exits the slope. This may be accomplished either by drilling drainage holes into the slope, excavating and placing drains in the slope, or protecting the surface of the slope with free-draining materials such as a gravel or rock blanket on geosynthetic drainage materials. Unless the potential for seepage is high, a rock or gravel blanket will usually suffice. A blanket 1 to 2' thick placed on a geosynthetic fabric or granular filter could be used in such cases that are discovered during construction because cost is relatively small. Where material in the slope is well graded and free draining, a blanket will develop naturally in many situations, and special treatment may not be needed.

Figure 4, Landslide (natural slope failure).

Figure 5, Cutoff trench at a dam in Arizona.

Figure 6, Cutoff trench at a dam in Texas.

Rainwater damage commonly takes the form of surface erosion. Preventive measures may include:

- Divert surface runoff away from slope faces with small collector ditches a short distance back from the intersection of the slope with the ground surface.
- Install surface sodding or blanket with a thin layer of gravel or rock on a layer of geosynthetic fabric.
- Construct ditches to carry water off laterally from the slopes.
- Steepen the slopes, as long as the slope remains stable.

However, when rainwater can infiltrate foundation soils to produce a rise in ground-water levels and resultant seepage, both the sodding and slope steepening procedures may actually be detrimental and, therefore, should not be used in conjunction with a seepage condition without an adequate analysis of the situation.

Erosive action from flowing water in a canal or wave action in a reservoir is potentially the most severe type of water damage. Protection involves flattening of slopes or protection with gravel or rock blankets placed on appropriate granular filters or geosynthetic materials depending on the severity of the potential damage. Situations exist where sloughing of slopes is not particularly important such as in borrow pits within the reservoir area. Such slopes should be flattened sufficiently to reduce the hazard.

Situations may arise where selection of slopes is controlled by criteria other than stability alone. For example, some state highway departments use cut slopes closely approximating natural slopes as a method of drift control for sand and snow. Some state highway departments require flattened slopes for grass cutting equipment. In some canal work, 2h: 1v slopes are specified (depending upon the types of materials) instead of the more conventional slopes to minimize maintenance and to provide more stable channels. Sometimes, slopes are flattened in connection with earth dams and high canal fills to distribute stresses caused by high embankments.

14. Settlement and Uplift. If a structure is not on bedrock or supported by means of piles, piers, caissons, or walls, the structure must be capable of withstanding some vertical movement in the foundation soils. The usual method for reducing such movement to within acceptable limits is by spreading the base or footings of the foundation so that unit pressures are reduced. Uplift of structures may occur from several causes such as hydrostatic forces, wetting of expansive soils, and frost action.

Structures and canal linings must be protected against excessive upward movement. For light structures, such as chutes and lined canals, protection against hydrostatic uplift must be provided. This is commonly accomplished by underdrains or drainage blankets of sand and gravel and/or geosynthetic drainage materials. Where possible, these drains will discharge into adjacent drainage channels or into the structures. For certain lined canals, this may require flap valves on outlets into the canal. Special care should be taken to keep the drains open and the drainage medium free from contamination.

Expansive soils may cause uplift upon wetting, particularly when the loads are light, when the initial in place dry density is high, and when the initial moisture content is low. Usual methods for reducing movement of rigid structures are by using long friction piles or belled caissons attached firmly to the structure, by increasing unit footing pressures, or by removing expansive soils and replacing with nonexpansive soils to a depth sufficient to control movement. Flexible canal linings such as earth linings or geomembrane linings are used in preference to rigid canal linings on expansive soils. In some cases, the structure load is less than that of the original earth removed, and elastic rebound may occur. This is true, particularly when saturated expansive clays are involved. In some cases, where structural loads are small, as in canal linings, conditioning the expansive clay is possible by prewetting the foundation and maintaining a moist condition so as to minimize future expansion.

Frost action may also cause undesirable heaving and uplift. The susceptibility of soil to frost action depends on: (1) soil type, (2) temperature conditions, usually expressed in terms of a "freezing index" (a summation of the degrees below freezing multiplied by the days of below-freezing temperatures), and (3) water supply. Uplift, from the development of ice lenses in foundation soils, may cause damage to such works as concrete canal linings, spillway apron slabs, and other lightly loaded structures. Frost action can also cause a decrease in the dry density in compacted earth canal linings. All soils, except those classified as GW, GP, SW, or SP, may be susceptible to frost action in cold climates and with water available. The susceptibility of gravelly and sandy soils varies with the amount of fines. Silts and clays, with a plasticity index of less than 12, and varved silts and clays are particularly susceptible to frost action.

Collapsible/low density soils can result in sudden settlement of a foundation upon wetting (see sec. 17). This condition is particularly severe in very arid portions of the Western United States, where these types of soils were usually deposited as Loess or as quickly deposited materials on the outer limits of alluvial fans. More than 3'of settlement is common in widespread areas of collapsible/low density soils. The usual treatment for collapsible/low density soil areas is to prewet the foundation areas for weeks or months in order to precollapse the soils at depth. The best method of prewetting is through ponding (preferably with wick drains); however, other methods such as sprinkling, water injection, or dynamic compaction can be effective. Explorations and analyses need to be performed in areas of potentially collapsible/low density soils to determine the areal extent, depths, and percentages of collapsible/low density soils that can be expected.

Subsidence due to long-term fluid withdrawal is also an area of potential settlement concern. Generally, subsidence occurs at such a slow rate that foundations are not immediately affected. However, the long-term use of a structure, such as a canal, can be severely impacted. Again, extensive explorations and analyses need to be completed in order to mitigate the effects of subsidence on the long-term operation of a structure.

15. Deterioration. The surface of excavated slopes in both soil and rock should be preserved in their natural state as much as possible. A cover of soil should be maintained over the excavated surface until final cleanup, and the structure or foundation should be placed immediately. Drying of the excavated surface should be avoided. Some indurated clays and shales will dry and crack when exposed to the air and then turn into a soft slurry upon rewetting. When it is impossible to place the structure or foundation immediately after such surfaces are exposed, a spray-on cover of pneumatically applied mortar or other approved material has been satisfactory in some situations.

16. Permeability. Water retention structures, particularly reservoirs and canals, depend on foundation soils for a part of their water barrier. If access to a water-bearing stratum exists, either through the process of excavation or from natural conditions, some corrective action is indicated.However, no soil formation is water-tight, and some water loss can always be expected. Movement of water from a reservoir or canal may result in piping or excessive water loss. These problems can be reduced by: (1) controlling the velocity of flow; (2) controlling the volume of flow; and (3) controlling exit conditions. The treatment required may or may not be the same for problems (1) and (2).

Damage to foundation soils resulting from water movement may be caused by gradual removal of soil particles either by solution action or by mechanical movement. Solution action is a problem only when foundation soils contain substantial amounts of soluble minerals.

The existence of solutioning in a foundation can be a serious problem. Whether such a condition involves primarily bedrock or soil solutioning can produce sinkholes in surficial soils. These sinkholes are often small in extent; and unless positive evidence is apparent that they are watertight, sinkholes should be stripped to sufficient depth and filled with compacted impervious soil when they are found along canal alignments. In reservoir areas and dam foundations, further explorations and more positive methods of treatment may be necessary. A sinkhole is shown on figure 7. Similarly, in the excavation process, when bedrock surfaces are exposed (i.e., contain solution channels, open holes, or cracks), such openings should be sealed with concrete or blankets of compacted impervious soil. An extensive grouting program may be required to seal the bedrock surface.

In limiting water loss at dams, as a rule of thumb, at least 95 percent of the pervious cross-sectional area of the soil foundation must be blocked. However, where a soil foundation consists of zones (or strata) of differing permeability, and 1 zone is more than 10 times as permeable as another, blocking of the more pervious zone is usually effective in reducing water loss even though it may be only a small part of the total permeable area. When water conservation is required, a permeable strata only partially excavated may be unacceptable. The strata may need to be completely cut off by continuing the excavation down to a lower elevation so that the entire permeable zone will be blocked, or by cutting the strata completely off by use of a cut off wall, or other similar means.

Figure 7, Typical sinkhole.

Similarly, on canals, any lined reach should be extended to cover the entire reach of a pervious stratum. Consideration should be given to fully lining (or using pipe) all constructed water conveyance systems. Thorough justification is required for using an unlined waterway. A study considers:

- Value of water saved
- Operation and maintenance cost
- Required drainage
- Right-of-way
- Allowable velocities
- Structure costs
- Cost of various types of lining correlated with other conditions

- Amount of tillable land removed (for canal alignment and because of seepage)

The unit value assigned to each item varies with local conditions; therefore, the amount of water loss allowed before lining a canal may vary considerably in different areas.

In constructing long canals, explorations are often insufficient to reveal fully all pervious strata that will be intersected. Where pervious canal reaches are encountered, an impervious lining should be considered provided that: (1) suitable material is available, and (2) slope and right-of-way limitations will permit. If space limitations, availability of suitable impervious materials, or other considerations indicate a requirement for another type of lining, or for a more expensive lining, appropriate authorities should be consulted.

17. Unsuitable Foundation Soil Conditions.

a. General. Materials normally considered unsuitable for foundations include:

- Organic matter such as topsoil, swamp deposits, or peat
- Low density materials such as loose deposits of silt, sand, loess, talus deposits, spoil piles, and dumps
- Some clays classified as highly plastic, active, sensitive, or swelling
- Soils in a soft and saturated condition

Whether a soil is deemed unsuitable depends upon several factors. Most important is the type of structure being contemplated. Another important factor is the geographical location of the intended structure. For example, some soils may be determined to be unsuitable in one climate, while the same soils in another climate may be deemed undesirable or even suitable depending upon the climate and the availability of suitable soils. In other words, the shortcomings of a soil can often be overcome through prudent design.

For most structures, unless the deposits of unsuitable materials prove extensive, avoiding these materials for use in structural support may prove economical and satisfactory. The more common methods to avoid using unsuitable materials include:

- Removal and replacement with compacted select material
- Full penetration with piles, piers, caissons, or walls
- Displacement; that is, remove a mass of material equal to the mass of the structure

If feasible, the structure can be relocated to provide better soil conditions. Treatment with additives may also be effective for some soils.

b. Topsoil. As in borrow deposits, the usual practice is to remove topsoil from below all foundation elements. Topsoil in this sense is the surface layer of material containing decaying vegetable matter and roots. Removing all soil containing fine hairlike roots is unnecessary but is required of the rather heavy root mat. On prairie topsoil, which contain light grass cover 2 or 3”, stripping will suffice. Agricultural lands are stripped to the bottom of the plowed zone, usually 6 to 10”. Valley bottom lands and brush-covered land commonly require stripping up to 2 or 3’ for removal of inadequate material. Forest land requires removal of stumps, resulting in stripping requirements of 1 or more meters.

c. Swamp Deposits. Marshlands, river backwaters, lakebeds, and flood flow deposits are included in this category. Foundation soils formed by these deposits contain appreciable quantities of vegetable matter, clay, silt, or sand lenses deposited in water.

Shallow deposits should be removed; however, deposits may be deep, and their depth and extent should be established before adopting a method of treatment. Relocation is recommended to avoid such deep deposits, particularly where roads, canals, pipelines, or transmission lines are involved. For dams, powerplants, and similar large structures, such materials may be excavated and wasted or used for topsoil; for bridges, pumping plants, and medium-size structures, where a water barrier is not required, fully penetrating piles are commonly used. Sometimes, adequate structural performance can be obtained by excavation to firmer material and placing and compacting a pad of select material.

d. Silt and Sand. While dense silt and sand deposits generally will be satisfactory, low density deposits of these materials may be undesirable. Usually, such low density materials are found as surficial deposits occurring as loess, sand dunes, sandbars in stream channels, alluvial fans, and deltas at the head of reservoirs and lakes. The surface layers of soil subject to frost action are low density in some areas.

Low density, poorly-graded sand, silts, and gravels that are saturated and located in an seismically active region are highly susceptible to the phenomenon of liquefaction when disturbed by vibration. Many slope and bearing capacity failures have occurred during earthquakes because of liquefaction in low density materials.

Loose sands and wet silts will consolidate quite readily when loaded and may not require special treatment when used for flexible structures such as road fills and canal banks up to 20' high. However, dry, low density silty soils require removal or prewetting to support a structure. Figure 8 shows cracking and settling of a canal bank in dry, low density silt after filling the canal. Foundation soils composed of low density materials and wet silts must be removed for earth dams and for fills higher than 20'. For foundation soils of dry, low density silt, either removal or consolidation by ponding is required as shown on figure 9. Consolidation of saturated sands can be facilitated by draining water from the soil.

Rigid structures usually are not founded on loose silts and sands. These materials are either removed or consolidated, or footings or piles for the structures are extended to firmer material. Normally, designs will show the method to be used.
Where loose sands or silts occur in foundation soils of water retention structures, such as canal banks and dams, lining or cutoffs are indicated. Permeability and the value of water are usually the basis for determining the need for treatment. A long canal may require treatment for a soil condition that would not need treatment for a short lateral construction.

Figure 8, Cracking and settling of canal bank in dry, low-density silt.

Figure 9, Ponding dry foundation of a Dam in Nebraska.

e. Talus and Spoil Piles. The term "talus" describes the buildup of soil and rock debris at the base of a slope. Generally, such deposits consist of low density materials and seldom provide adequate support for any structure more important than a simple roadway. The usual treatment is to remove the talus. Excavations into talus deposits are likely to be unstable if cut slopes are appreciably steeper than the natural slope of the talus deposit.

The processes used for disposal of excess or unsatisfactory materials in spoil piles are similar to the processes by which talus deposits developed, so the two have similar characteristics. Spoil piles and talus deposits should not be used for support of any structure without thorough investigations, testing, analysis, and design.

f. Highly Plastic Clays. Clays that create foundation problems have one or more of the following characteristics:

- plasticity indexes greater than 25 percent,
- colloid content greater than 20 percent,
- high sensitivity, and
- shrinkage and swelling characteristics as noted by their tendency to shrink and crack when drying.

Also, a relationship appears to exist between moisture content and strength of clays.

g. Soft or Saturated Soils. Almost any soil foundation is reduced in quality when it contains large amounts of water. An apparently firm dry foundation soil may become unstable when saturated. Foundation soils subject to saturation may be protected by either preventing water from entering the soil through use of drains to divert surface runoff or by removing water that reaches the soil using underdrains, drainage blankets, or relief wells. Structures whose foundation soils will be permanently underwater are designed against hydrostatic uplift.

Because of the difficulty in preparing and inspecting soils located beneath the water table. Typically, specifications contain provisions that require dewatering of foundation soils before excavation. However, some work in water is generally necessary when constructing cofferdams, pump sumps, and drains. These structures should be kept away from planned excavation surfaces so the surfaces will not be damaged by the construction or by the action of dewatering facilities. Saturated materials can be damaged by permitting an upward flow of ground water into the excavation or by permitting heavy equipment traffic in the area.

18. Foundation Improvement Technology and Dam Rehabilitations. Undesirable foundation conditions can sometimes be alleviated by treatment in place. New technologies have been developed to remediate foundation problems. As part of the safety of dams program, the U.S. Government has had to re-examine many of its older structures. These examinations have revealed foundation concerns at many structures including excessive seepage and possible piping; collapse of dry, low density abutment soils; and potentially liquefiable soils during earthquake shaking. Many dams have been rehabilitated using ground improvement methods. These ground improvement techniques are also applicable to foundations for new structures. This section contains a brief discussion on several ground improvement methods, and their applicability to new or existing foundations is given. For more details on these technologies, consult references provided.

Each method of ground improvement has unique advantages and disadvantages relative to the ground conditions requiring improvement. In many cases, differing methods can be combined to overcome disadvantages of the methods and benefit from the positive aspects of each method. For example, at Steinaker Dam, the government desired to treat dirty sands and silts with dynamic compaction. Wick drains were used in combination with the dynamic compaction to effectively treat those soils. Another example is Jackson Lake dam where a block treatment approach was used. Dynamic compaction was used for shallow treatment under the center of the dam, while mixed-in-place, soil-cement columns in a honeycomb pattern at depths up to 90 feet were placed under the toes of the embankment. The use of the two combined methods resulted in satisfactory ground improvement to preclude liquefaction failure.

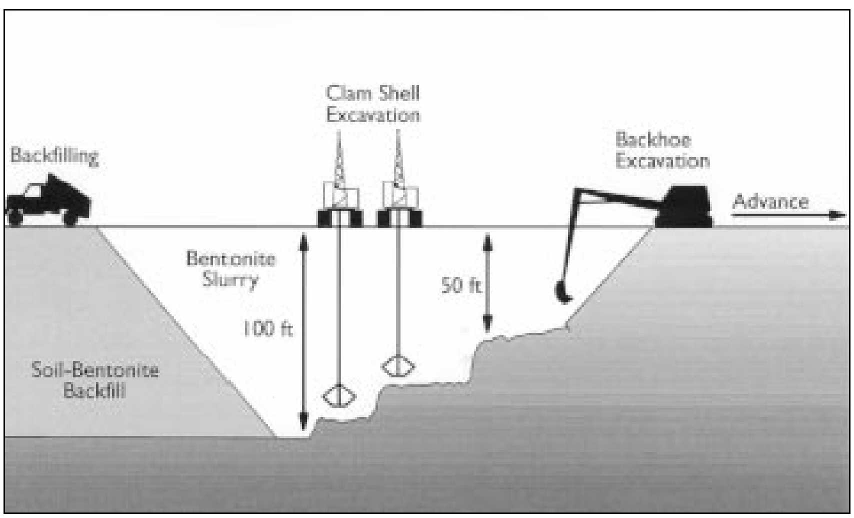


Figure 10, Schematic drawing showing slurry wall construction methods, backhoe and clamshell excavation and surface backfill method.

a. Slurry Walls. Slurry walls can be used to provide positive cutoff for new dams or can be installed to alleviate seepage problems in existing dams. A deep trench is excavated with various types of equipment, depending on the depth and consistency of the soil or rock. The trench is supported with bentonite or biopolymer slurry. Backfill materials can range from soil/cement/bentonite (SCB) mixtures to structural grade concrete. Often, slurry wall backfill is designed with strength and deformation properties similar to those of the surrounding soils. Material with soilconsistency can be excavated in long trenches with backhoes, draglines, or clamshells (see fig. 10). Clamshells can be used in a wider range of deposits and can be used to excavate "cells" or "panels." Material with rocklike consistency can be excavated with chisels or rock mills with hardened cutterwheels. Rock mills are most often used to make cells or panels for cutoff walls (fig. 11). The U.S. Government has applied slurry wall technology to existing dams and new dams. An interesting application of slurry wall technology for a new dam was performed at Diamond Creek Dike, where an embankment was built with a sand core which was later cut with a slurry wall for installation of a SCB backfill to provide the water barrier or "core" zone in the dike. This construction method was selected due to a shortage of clay borrow and to accelerate embankment construction.

Design considerations for slurry wall applications should include determination of the geologic and geotechnical conditions, establishment of the hydraulic conditions, requirement of the design flow per unit surface area, determination of the wall size and geometry, and design and determination of the properties of the backfill material.

Quality control issues with slurry walls include verticality, caving, foundation contact and connection of panels or cells, permeability, compressive strength, and voids or windows

Additional quality control issues include excavation and repayment balance, width and depth measurements, and cleaning of foundation contacts. During construction of slurry walls, sounding devices can be dropped into the slurry to sound the width of the trench to determine if caving has occurred. Fresh SCB backfill material cylinders may be cast for compressive strength testing. The heat of neutralization test can be used to check cement contents

Samplers can be used to check for clean foundation contact conditions prior to backfilling. Drilling, water testing, and drill coring programs are often employed to check backfill quality. However, if the backfill material is weak (less than 200 lb/in^2), core recovery may be difficult and precautions must be made to avoid wall fracture. Cross borehole tomographic P wave imaging can be used to look at wall integrity and guide investigations.

b. Fill Loading with Sand or Wick Drains. Fill loading has long been used to consolidate soft silty and clayey deposits for embankments. The time required for consolidation can be determine by consolidation theory. In past practice, vertical sand drains were constructed to accelerate the process. Recently, synthetic wick drains have been developed to replace sand drains. The wick drains are plastic drainage elements enclosed in a filter fabric. Wick drains are installed by vibratory pile driving equipment.

These drains are also a useful complement to other densification methods such as dynamic compaction.
Design considerations for wick drain application include establishing the geologic/geotechnical site conditions, determination of the driving and installation methods, testing of undisturbed samples for consolidation rates and strengths, determination of proper spacings, depths, and types of drain elements, and determination of the required foundation properties and fill loading times. Quality control testing depends on the application. For fill loading to improve soft clays, piezometers are often installed to measure excess pore pressure dissipation over time. Field strength tests, such as vane shear or lab tests, can be used determine strength gain over time. When drains are combined with other ground improvement methods, there may be no quality control test needs other than assurance of proper placement.

c. Dynamic Compaction. Dynamic compaction can be a relatively low cost method for increasing the densities of foundation soils. Dynamic compaction has been used to treat low density, potentially liquefiable and collapsible soils. Weights (steel or concrete) of up to 35 tons are repeatedly dropped from heights of up to 100’ in a pattern across a site to create high intensity surface impacts (fig. 12). Cranes are generally modified from standard configuration to accommodate the heavy weights used to perform dynamic compaction. Weights are dropped in patterns and sequences until densification is judged to be sufficient. There are a number of case histories documenting the use of deep dynamic compaction to remediate seismically sensitive embankment foundations.

Effective treatment depth is usually limited to depths between 30 and 35 ft, depending on foundation soil types, ground-water conditions, site geometry, weight drop patterns, equipment, and equipment sequencing. Dynamic compaction is typically used at sites with nonplastic materials. The potential for effective treatment becomes less for sites having soil horizons with high fines contents (greater than 20 to 30 percent fines). Effective treatment of sites having soils with high fines content can be achieved by installation of wick drains or other drainage features prior to deep dynamic compaction.

Design considerations include determination of the geologic/geotechnical conditions, evaluation of water control for working pad, procuring backfill material, determination of the drop weight, height and pattern, and determination of monitoring parameters such as penetration resistance testing.

Quality control issues include crater depths (mapping), surface heave or settlement monitoring, hammer energy, pore water pressure buildup, and vibration monitoring. Control testing for dynamic compaction can include load testing, piezometric monitoring, penetration testing, shear wave velocity determinations, and soil sampling for evaluation of engineering properties.

Figure 11, Slurry wall construction using panel excavation and tremie placement, view of a hydromill and crane with 30-inch x 10-ft clamshell.

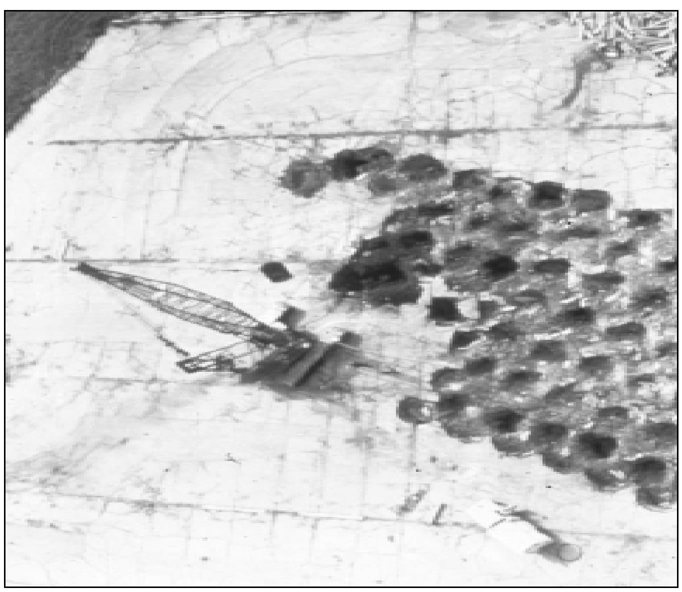


Figure 12, Photograph of dynamic compaction showing crane, weight, and impact pattern.

d. Vibratory Densification. There are many forms of vibratory densification which can be used to improve foundation conditions. These include vibro-compaction or -replacement, vibrating probes (vibrating rods with small diameter columns), and blasting.

1. Vibro-Compaction/Replacement (Also Known as Vibroflotation and Stone Columns). Loose foundation soils at some damsites may be densified for low to moderate costs by inserting a type of vibrating tube or probe into the foundation at a variety of depths and locations across the site (fig. 13). Water and/or air jets may be used to aid insertion of the vibratory probe and densification of surrounding soils. The effectiveness of these vibro-compaction methods depends upon soil type (gradation) and depth, spacing of treatment, vibration levels, backfill added (if any), and pore pressure response during treatment. In general, these methods are effective in treating cohesionless soils with less than 20 percent fines. For soils higher in fines content, stone columns are used to improve the soil.

In the vibro-replacement method, sand and/or gravel can be introduced into the foundation during the compaction process, which displaces and intrudes into soils around the probe, resulting in densification of foundation soils. Introduction of sand or gravel can also improve pore pressure dissipation and improve shearing resistance of the treated zone. The probe is repeatedly withdrawn and inserted at the point of treatment to maximize foundation improvement. This process results in construction of a "column" in the foundation, the characteristics of which vary with the nature of the treated materials and those introduced during construction. Columns are typically constructed in triangular or rectangular grid patterns, at center-to-center spacings of 7' to 10', and are generally 3'or greater in diameter.

Vibro-compaction and replacement methods using water can have substantial containment and/or cleanup costs during the construction, compared to dry methods. It may be necessary to handle large quantities of excess mud, water, silt, sand, and gravel, depending on the final construction methods specified, foundation and construction material types, ground-water conditions, and site surface characteristics.

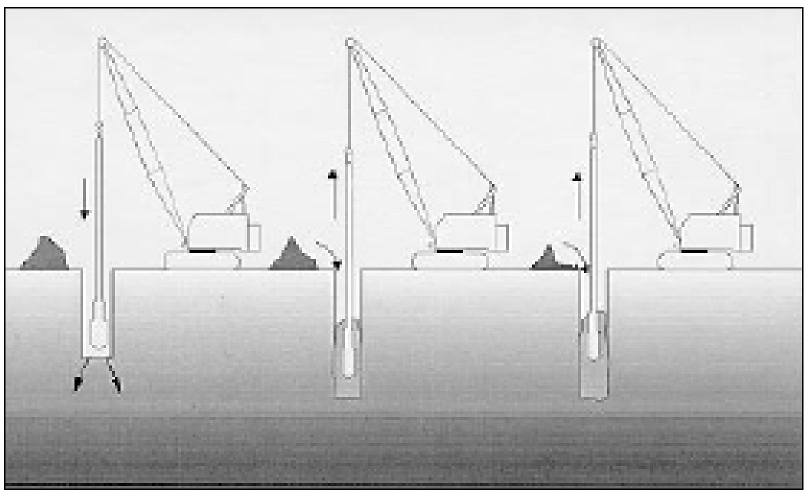


Figure 13, Schematic drawing of vibro-replacement, backfill can be added by top feed (illustrated) or bottom feed methods.

2. Vibrating Probes (Small Diameter Vibratory Columns/Drains; i.e., TerraProbes, Vibrorods, Vibrowing). Small diameter (less than 1.5') columns or drains may be constructed in loose soils, densifying them through displacement, vibration, and the introduction of sands or gravels in the process. In addition to densifying surrounding soils, the small diameter columns that result also improve pore pressure dissipation and provide some increase in shearing resistance. Columns are usually spaced on close centers, generally less than 3', in rectangular or triangular patterns. This method is moderate to high cost when compared to other foundation improvement methods. As the depth of required foundation increases to around 70'or greater, this method may become more cost effective as equipment limitations begin to impact the effectiveness of other methods.

There is little data available to evaluate the performance of dam foundations remediated by this method. There no are data from testing of posttreatment hydraulic conductivity in treated foundation materials versus embankment performance during earthquake shaking. Posttreatment testing of dam foundation densities has also not been completed. Therefore, this method should generally be considered as an economical secondary defensive design feature, counting primarily on the columns to improve pore pressure dissipation during an earthquake. Series of columns can be used adjacent to areas treated by other methods to prevent high pore pressures from adjacent untreated areas from adversely affecting the treated zone. This method has been used in the design and construction of seismic remediations for Mormon Island Auxiliary Dam near Sacramento, California. Typical control tests for verification are penetration resistance tests, shear wave velocity, and undisturbed soil sampling.

Design considerations for vibratory ground improvement methods should include determination of the geology and geotechnical properties, establishment of ground improvement requirements, selection of optimum vibratory ground improvement method(s), evaluation of ground settlement, determination of backfill properties and location of backfill sources, and establishing baseline data for quality control testing.

Quality control issues for vibratory ground improvement include column locations, spacing and verticality, energy input required, and quantities of backfill placed.

Acceptance testing for vibratory ground improvement is primarily based on penetration resistance testing, standard penetration tests, cone penetration tests, or Becker penetration tests. Improvement in shear wave velocity can be determined. Undisturbed samples can be procurred to evaluate density and moisture changes. Stress levels and modulus changes can be evaluated with a flat plate dilatometer or pressuremeter. It is wise to never base acceptance on a single test. Acceptance should be based on average properties, not minimum or maximum properties.

3. Blasting. Blasting, when used as a foundation improvement method, densifies soils as a result of shock waves, vibration, limited liquefaction, displacement, remolding and settlement in the treated soils. This is a relatively low cost foundation improvement method but results in nonhomogeneous soils may be erratic. It is also difficult to assess any potential damage that may occur to, or beneath, an existing embankment or an appurtenant structure. The use of blasting as a foundation improvement method is much more likely to be practicable for sites priorto construction of a new embankment. An example of blasting at the Jebba damsite is given in the references. This method has been used in ground improvement projects where rubble and large boulders preclude the use of other vibratory methods.

Design considerations for blasting should include determination of geology and geotechnical properties; evaluation of charge size, burial depth, charge spacing, number of coverages, and surface settlements; evaluation of required performance parameters; evaluation of blasting; and vibration safety requirements.

Typical verification testing includes piezometric measurements, penetration resistance tests (SPT, CPT, BPT), shear wave velocity determinations, and undisturbed sampling. Settlement is usually monitored.

e. Grouting and Soil Mixing. Numerous technologies exist for grouting and soil mixing in ground improvement. The most common of these technologies include compaction grouting, jet grouting, and soil mixing.

1. Compaction Grouting. This method of soil densification utilizes a low slump mortar grout (usually 0 to 2”), that is injected under high pressures 300 to 700 lb/in^2, to form a series of grout bulbs that displace and compact loose foundation soils between and adjacent to the bulbs.

The cost of compaction grouting, when compared to other methods of foundation improvement, is moderate to high. The relative cost of compaction grouting becomes more economical when treating identifiable thin lenses or layers where treatment of competent overlying layers is not required, or in small areas adjacent to structures.

Compaction grouting is often used in tunneling and underpinning operations for settlement and deformation control. The method is rarely used in the construction of earth dams but has been used to remediate older dams. Prior to selection of compaction grouting for ground improvement associated with dam embankment foundations, the following issues need to be considered:

- Compaction grouting may not be practical on the downstream side of an embankment since a series of impermeable grout bulbs, while not interconnected, might impair drainage in the downstream direction. This could adversely impact embankment stability during static or dynamic loading.
- For remedial seismic designs of embankments that might use compaction grouting as a method of foundation treatment, it is recommended that compressive strengths and shearing resistance provided by grout bulbs should be carefully evaluated. How the individual grout bulbs respond in treated dam foundation materials during strong seismic shaking is presently not well understood.

Design considerations for compaction grouting should include definition of the geotechnical parameters and structure interaction considerations, location of the compaction zones, definition of densification parameters, estimation of grout volumes required, and development of a plan for sequencing along with monitoring programs.

Quality control testing includes monitoring of grout consistency (slump and compressive strengths), pressures and volumes injected, and measurement of ground heave. On critical programs, site-specific evaluation of grout bulb effectiveness may be made. After installation, settlement is monitored and the improvement in soils adjacent to the columns can be monitored with penetration tests (SPT, CPT). Increases in stiffness and horizontal stresses can be monitored by flat dilatometer and pressuremeter tests.

2. Jet Grouting and Deep Soil Mixing. Jet grouting is performed in foundation soils by high pressure, and high velocity jets of water and/or air, which "excavate" the materials, inject them, and mix them with some type of stabilizer (fig. 14). This procedure can be used to construct columns of improved soil in a wide variety of foundation soil types (gravels, sands, silts, and some clays).

Columns and panels can be interlocked to form walls, to cutoff/reduce seepage through a foundation, or form cells within a potentially liquefiable foundation. The cost of jet grouting can be relatively high when compared to costs of other foundation improvement methods, depending on the design goals of foundation remediation and the distribution and type of soil to be treated.

Deep soil mixing methods can also be used to construct columns, walls, or cells of fairly precise final dimensions, in materials ranging from sand to clay. This method utilizes hollow stem auger flights and mixing paddles, which are advanced into the soil, while grout is pumped through the tip of the auger. The auger flights and mixing paddles mix the grout with the surrounding soils during penetration to design depth and withdrawal. The cost of deep soil mixing relative to that of other foundation improvement methods is moderate to high.

Jet grouting or soil mixing methods may introduce impermeable elements into the dam foundation that may adversely affect embankment stability (especially on the downstream side of a dam). In several cases, these methods have been used as cutoff wall elements for dams. In seismic strengthening applications, the stress-strain compatibility of the columns with encapsulated or adjacent foundation soils is poorly understood (use of compressive strengths or shearing resistance should be closely scrutinized in any analysis). Finally, ground motions reaching an embankment may be amplified through these stiffer elements, resulting in stronger shaking of the dam.

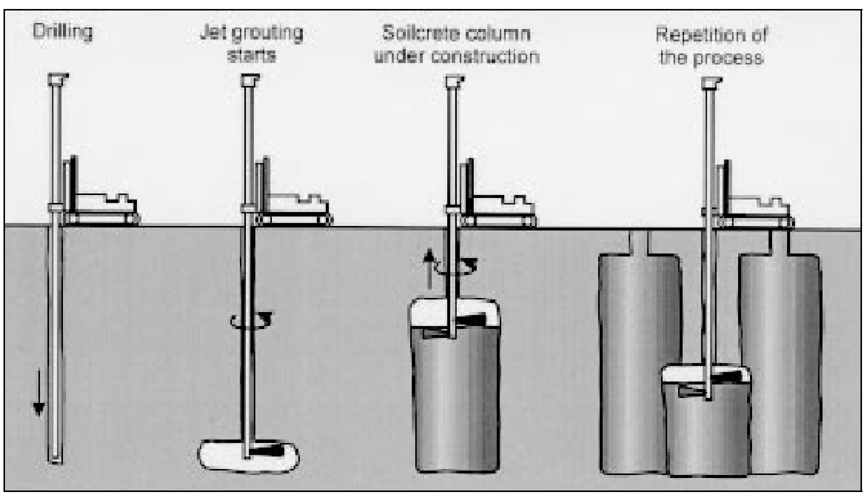


Figure 14, Schematic drawing of jet grouting process showing construction of columns.

Design considerations for jet grouting and soil mixing include determination of the range of soils to be treated, required stabilizing additive dosage through trial mix design studies, reasonable strengths and permeability that can be obtained, treatment areas and depths and volumes, and required performance parameters and quality analysis and control monitoring plans.

Typical field testing consists of either waste or fresh column sampling of the soil additive mixture. Waste sampling can provide insight as to volume of the treated area and conservative assessment of in situ characteristics. Core samples are often procurred from cured columns, and compressive strength is evaluated. Water testing can be performed in boreholes, but there are often problems with proper location of the holes for reliable tests. Records of treatment methods (columns installed, volumes injected) are maintained to monitor production and document construction.

D. Embankment Dams

This section provides information according to standard practices established by the U.S. Government in the early 1990s. Clearly, the engineer acquires expertise as knowledge and experience develops, technology advances, and investigation and construction practices improve.

19. Foundation Treatment.

a. Design Features. An embankment dam is often the most feasible type of structure for a particular damsite because of relatively poor foundation conditions that are unfavorable for rockfill or concrete dam construction. Embankment dams are frequently founded on alluvial deposits that consist of layers of varying thickness of coarse sand and gravel, silts, or clays before bedrock is reached. Deeply weathered or faulted rock may also be encountered in embankment dam foundations. Foundations of thick deposits of windblown silts and fine sands present especially difficult problems. Foundation design must consider three main problems: (1) seepage, (2) stability, and (3) control of deformation.

Cutoff trenches, cutoff walls, or combinations of trenches and walls are most commonly used in foundations of permeable materials. These cutoffs are designed to stop the flow of seepage water or to lengthen the path of percolation under the dam. Blankets of impervious material that extend upstream from the impervious zone of a dam on the reservoir floor and cover all or part of the abutments are sometimes used for lengthening the percolation path.

Cutoff features are typically located at or upstream of the centerline of the crest of the dam beneath the impervious zone of the embankment. The location and design of the cutoff feature should provide an adequately long percolation path through both the embankment and the cutoff feature. Where the embankment is founded on or near bedrock, "curtain" grouting and "blanket" grouting are usually necessary to extend the cutoff into rock. Closing of joints and other openings in bedrock is usually necessary and is accomplished by injecting grout (under pressure) to form a curtain cutoff. Comparatively shallow low-pressure grouting ("blanket grouting") may be used to seal near surface fractures in the rock beneath the impervious foundation area.

Two general types of cutoff trenches are (1) sloping-side trenches and (2) vertical-side trenches. Depth, location, and type of cutoff trench used depend on the nature of foundation materials and on their permeability. Common practice for foundations, with bedrock at a shallow to moderate depth, is to excavate a sloping-side trench located at or upstream from the centerline of the crest of the dam. For trenches located upstream, the trench centerline parallels the centerline of the dam crest across the valley floor but converges toward the crest centerline as it extends up the abutment so that the cutoff is fully beneath the impervious zone of the embankment. When bedrock or an impervious stratum is relatively close to the surface, the cutoff trench may be widened to the full width of the impervious zone of the embankment. Sometimes, the entire embankment is founded on bedrock when the top of bedrock is located at a shallow depth. Where bedrock cannot be reached economically, the cutoff excavation is extended either to a stratum of relatively impervious material or to a depth that will provide the desired length of the seepage path. Care must be exercised when analyzing a dam foundation to ensure that liquefiable materials are not left in the foundation.

Sloping-side cutoff trenches are excavated using front-end loaders, shovels, backhoes, draglines, or scrapers. The excavation is backfilled with selected impervious materials equivalent to the impervious zone in the dam embankment and compacted in the same manner as rolled earth fill embankment material. Properly designed filters and transitions must be provided between the downstream face of the cutoff and alluvium (or rock surfaces) to preclude any piping into foundation alluvium (or bedrock). Rock surfaces must be shaped to provide gradually changing surfaces and must be treated to seal fractures.

Vertical-side trenches may also be used as cutoffs. In some instances, the depth of a sloping-side cutoff trench may be extended by continuing the excavation as a narrow trench with vertical side walls. Vertical-side cutoff walls require special design and construction techniques where the two types of trenches are connected or where the wall connects to the base of the impervious zone. Sometimes, vertical-side cutoff walls are used near the upstream toe of a dam or in conjunction with upstream impervious blankets in which case they must be connected to the impervious zone of the dam. Use of cutoff walls has increased in recent years because of improved technology and construction techniques. There are basically three types of vertical-side cutoff walls: (1) soil-bentonite slurry-trench cutoff wall, (2) concrete cutoff wall, and (3) cement bentonite/plastic concrete cutoff wall (see sec. 18a).

In addition to forming cutoffs, concrete walls are sometimes used to treat certain geologic features such as faults or fractured zones. Particular geologic features may require placing short, independent wall sections at certain locations on the abutments or within the river channel. Concrete cutoff walls may be required:

- When bedrock is hard, but intensely fractured and irregular
- For bedrock dipping steeply, especially on the abutments
- Across shear or fault zones
- Across weathered zones

Careful consideration must be given to foundation contacts in the design of an embankment. Any continuous void spaces caused by lack of intimate contact between the impervious portion of the embankment and the foundation may result in dangerous seepage. Overhanging rocks and undesirable material should be removed; slopes should be flattened and smoothed to permit embankment materials to bond properly to the abutments. In thin core dams, the impervious section may be widened at the points of contact with the abutments.

Foundation grouting is a process of injecting suitable grouting materials into the underlying foundation through specially drilled holes of seal or fill joints, bedding planes seams, fissures, or other openings. Grouting may be done with neat cement (the most common method), mixtures of cement and sand, or cement mixed with bentonite or other clay. In special cases, chemical or bituminous grouting may be used. The adopted program may consist of grouting along a specified line or multiple lines to create a deep, relatively impermeable curtain; comparatively shallow or "blanket" grouting over wider portions beneath the impervious zone of the foundation area; or both. Every foundation presents a unique grouting situation that depends on composition and nature of the material and on the geologic structure.

In foundations of unlithified permeable materials, the path of percolation can be increased by constructing a blanket of compacted impermeable material upstream of the dam. Blankets are usually used when cutoffs to bedrock, or to an impervious layer, are uneconomical because of excessive depth. Sometimes, they are used in conjunction with partially penetrating trenches designed to increase the length of the path of percolation. The topography just upstream from a dam and the availability of impervious materials are important factors in the design of blankets. The length of upstream extension is determined by desired reduction of volume of seepage and of hydraulic gradient. In zoned embankments, the blanket must extend beneath the upstream shell as a continuation of the central, impervious section. Blankets may be constructed to cover either or both abutments or particular abutment areas. The blanket thickness will vary with permeability of the material and with the head of water. Filters may be necessary to prevent piping of the impervious blanket layer into the foundation. Methods for improving foundation stability include:

- Removal of unsuitable material
- Pre-consolidation by dewatering or by irrigation and loading
- Adequate drainage of the downstream toe by means of drainage blankets and filters, toe drains, and relief wells to avoid piping and to reduce hydrostatic pressures
- Construction of stabilizing fills to prevent displacement of the foundation
- In place densification

The minimum requirement for foundation stripping is removal of vegetation, sod, topsoil having high organic content, and other unsuitable material that can be removed by open-cut excavation. In cases where overburden is comparatively shallow and composed of soft clays, loose fine sands and silts, or extremely pervious sands and gravels, or liquefiable materials, the entire foundation area of the dam may be stripped to bedrock.

In special cases, water may be added to the foundation by irrigation to cause consolidation. Dry loessal (wind deposited) soils and other low density silty soils have been successfully treated by this method. Sometimes, the entire foundation area is saturated and preloaded to cause consolidation and stabilization of otherwise unsuitable foundation materials when the cost is less than the alternative of excavating and replacing the material with selected borrow material. Usually, such treatment is accomplished by staged construction of the embankment.

Loose, fine sand deposits below ground-water table have been successfully consolidated by deep compaction techniques such as compaction piles and dynamic compaction, usually in conjunction with dewatering. Sometimes, these procedures are used to treat liquefiable sands and silts (see sec. 18).

The purpose of dewatering is to permit construction in the dry and to increase the stability of excavated slopes. Clean, coarse to medium sands can be readily dewatered. Clean, fine sand can be dewatered with some difficulty. Dirty, fine, medium, and coarse sands are difficult to dewater. Silts and clays require extraordinary methods to dewater. Usually, dewatering consists of drains, drains with sumps, deep wells, and wellpoints either alone or in combinations for maximum effectiveness.

Dewatering shall maintain a sufficiently low water table to allow for satisfactory excavation and backfill placement. Dewatering systems must be carefully designed to ensure the adequacy of the system. If sufficient effort is not made in dewatering, foundations can be damaged, excavation safety can be jeopardized, and costly construction delays can occur. The design engineer should develope dewatering schemes in certain critical situations to avoid costly construction delays.

Preventing all seepage under a dam is not possible. Inevitably, water will escape somewhere in the vicinity of the downstream toe. When foundation materials in that area consist of or contain appreciable quantities of fine sands, silts, or clays that may be carried away, by water escaping under pressure, a filter or drainage blanket is provided. The filter is constructed of selected materials designed to satisfy filter criteria. The filter allows the necessary passage of water but prevents movement of fine soil particles from the foundation. A filter design accounts for grain sizes of foundation soils and head loss. Two, or more, layers of different sizes of sand and gravel may be required to prevent piping and to provide adequate drainage capacity. Another method to relieve hydrostatic pressures near the downstream toe of an embankment or high pressures beneath impervious foundation layers in the vicinity of the downstream toe is by installing vertical relief wells. The purpose of the wells is to relieve excess water pressure in the foundation, and thereby prevent formation of boils and heaving or piping.

Toe drains are commonly installed in a trench along the downstream toe of a dam as part of the filter or drainage blanket to collect and convey water to measuring devices. Perforated concrete, high density polyethylene, or other types of pipe may be used for the drains. Beginning with small diameter pipe laid along abutment sections, the lines progressively increase in size; with lines of maximum diameter being placed across the valley floor. Toe drain pipe must be laid in a trench of adequate depth and grade to be effective. Drainpipe is encased in a filter (usually sand) and drain material (usually gravel) to prevent piping of surrounding material into the drain. Access points should be installed along the drainpipe and outlet line for inspection and maintenance and, if desirable, at other locations for observation and measurement of the amount of seepage water and detection of any piping. The function of drains is to allow uninhibited escape of seepage water, thereby preventing saturation and consequent reduction of stability of the downstream embankment and foundation of the dam. Toe drains, placed sufficiently deep in the foundation keep the ground-water level low, preclude high exit gradients, and prevent formation of boggy areas near the downstream toe of a dam.

To make foundations of low strength materials suitable as support for a dam, stabilizing fills are often used to provide mass at the upstream, downstream, or both toes of a dam. These fills act to restrain foundation materials from displacing under load by increasing the normal effective stress across potential failure surfaces. The depth and extent of these fills depend on the properties of the foundation materials. Stabilizing fills are generally contiguous and integral with embankments.

b. Specifications Provisions. During construction, diversion and care of the river (stream, or surface runoff) is an important factor in the design of an embankment dam. Figure 16 shows a temporary diversion channel through an embankment dam. Specifications require that the contractor' s plan for diversion be approved, or the diversion plan should be provided in the specifications.

Figure 16, Temporary diversion channel through A Dam.

Underwater excavation for the foundation of a rolled embankment dam is not permitted. The contractor is required to un-water the foundation in advance of excavating an embankment cutoff trench. The un-watering is required to be accomplished in a manner that will:

- Prevent loss of fines from the foundation
- Will maintain stability of the excavated slopes and bottom of the cutoff trench
- Will result in all construction operations being performed in the dry

Dewatering is to be continuous until the excavation has been filled, except that in filling the cutoff trenches, the ground water may be permitted to rise within 5' of the top of the compacted embankment after at least 10'of fill have been placed in the dry. This provision enables the contractors to remove the majority of well points if they use that type of dewatering process. Usually, it is impractical to prevent water from seeping out along the contact of the cutoff trench slopes with the foundation. Hence, some surface drains are needed to ensure that backfill is placed in the dry. Open-jointed or perforated pipe drains surrounded by gravel and leading to sumps can be used to dry up the foundation contact. Placement of drains that will be left in place should be very carefully controlled and must be meticulously filled with grout to avoid leaving any potential leakage paths. The layout of these drains should be carefully planned so that the contact of the impervious zone is never penetrated, transversely, by more than one-third of its width. The distance between the ends of transverse drains from opposite sides of the impervious zone should be no closer than one-third the width of the pervious contact.

The requirements for excavation of embankment foundation provide that the area to be occupied by the embankment shall be excavated to a sufficient depth to remove all topsoil, rubbish, roots, vegetable matter of every kind, and all other perishable or objectionable material that might interfere with proper bonding of the embankment with the foundation. Earth foundation surfaces are to be prepared by leveling and rolling in the same manner as specified for placing embankment materials. Rock foundation surfaces are to be cleaned of all loose and objectionable materials. Foundation surfaces of shale and other materials that tend to slake or loosen on contact with air are to be cleaned just before placing earth fill materials or protected from moisture change. All foundation surfaces are to be moist but free from standing water. Figure 17 shows a foundation being prepared by excavation and water jetting to remove loose rock and to smooth the impervious foundation prior to placing earth fill.

Figure 17, Foundation preparation for the impervious (zone 1) foundation.

c. Control Techniques. The weak points in embankment dam construction are generally found within the foundation at the foundation contact with the embankment and at the contact of the natural ground surface with placed embankment. Construction of foundation seepage control and stability features included in the design must be carefully supervised by the inspection force to ensure compliance with specifications. Dewatering methods used in connection with the excavation of cutoff trenches and for stabilizing foundations should be carefully checked to verify that fine material is not being washed out of the overburden materials because of improper performance of drainage wells. To avoid creation of a "live" bottom due to upward flow of water, continuous pumping may be required.

Where overburden is stripped to firm material, the foundation surface (including all pockets or depressions) should be carefully cleaned of soil or rock fragments before placing the embankment. This may require handwork water or compressed air-water cleaning or both. Cleaning is particularly important beneath the impervious zones. Surfaces that disintegrate rapidly on exposure should be covered immediately with embankment material or an approved protective covering. If stripping operations indicate the presence of unstable or otherwise unsuitable material, test pits should be excavated for further exploration, and generally this material will require removal.

Before placing the first layer of impervious zone material on an earth foundation, the foundation surface should be leveled, moistened, and compacted. Foundation surfaces of firm material should be moistened, but standing water should not be permitted when the first lift is placed. An earth foundation surface may require scarification by disks or harrows to ensure proper bonding. When the foundation can be penetrated by tamping roller feet, scarification is not usually necessary. Where the surface would be damaged by penetration of the tamping-roller feet, a thicker layer of earth fill may be permissible for the first compacted layer, or the first layer may be compacted with rubber-tired (pneumatic) rollers. However, the first layer should not exceed a thickness that will prevent attainment of the specified density and should never exceed 15” loose lift.
When thicker lifts are placed, additional roller passes are usually required to ensure that proper compaction is attained. Special compaction methods should be used in pockets that cannot be compacted by the specified roller instead of permitting unusually thick first lifts to obtain a uniform surface for compaction. Figure 18 shows compaction of earth fill (zone 1) near the foundation contact with a pad foot roller.

Figure 18, Earth fill (zone 1) compaction by a model 925 C Caterpillar pad foot roller.

On steep, irregular rock abutments, wetter-than-optimum material may be necessary or desirable to obtain complete and proper bond. The rock surface should be properly moistened. Rock abutments should be shaped by excavation or dental concrete to allow compaction by pneumatic tire equipment or rollers directly against the contact. The fill surface should be sloped or ramped to about 1v:6h near the foundation surface to direct the force of the pneumatic roller wheel towards the foundation surface. Tamping by hand operated equipment should be avoided. If absolutely unavoidable, when hand-held compactors are used, care must be exercised to ensure that bond is created between successive layers of material. This may require light scarification between lifts of tamped material.

20. Compacted Earth fill.

a. Design Considerations. Rolled embankment dams vary considerably in proportion and physical characteristics of soils from which they are constructed. Some dams contain essentially one kind of material and are called homogeneous impervious structures, while others are constructed of zones of sand, gravel, rockfill, and miscellaneous soils in a variety of quantities and locations within the fill, in addition to an impervious zone. The impervious materials include clays (CL and CH), clayey sands or gravels (SC or GC), and silty clays (CL-ML). Occasionally, more pervious materials such as silts (ML) and silty sands or gravels (SM or GM) are used in the impervious zone. This impervious zone of earth fill, usually designated zone 1 in dams, provides the water barrier and is common to all rolled embankment dams. Homogeneous dams are virtually nonexistent in modern day dam engineering—for dams of any size. Internal filters and drains are always provided except for extremely small dams.

In controlling construction of impervious earth fill zones, the following design criteria must be satisfied:

(1) The material must be formed into an essentially homogeneous mass free from any potential paths of percolation through the zone or along the contacts with the abutments or concrete structures.
(2) The soil mass must be sufficiently impervious to preclude excessive water loss through the dam.
(3) The material must not consolidate excessively under the weight of superimposed embankments.
(4) Differential settlement that would cause shear planes must be avoided.
(5) The soil must develop and maintain its maximum practicable shear strength.
(6) The material must not consolidate or soften excessively upon saturation.

These design objectives are achieved by proper use of equipment and methods of borrow-pit conditioning, excavating, placing, compacting, and foundation shaping.

To satisfy design criteria of imperviousness, low consolidation, and resistance to softening, a high density is desirable. However, to control development of pore pressure, which would reduce shear strength, and to avoid formation of slickensided layers, which may become paths of percolation, material that is too wet must be avoided. Based on knowledge of available materials gained from exploration and testing, the designer will specify moisture control requirements near laboratory optimum moisture content. The inspector should observe the fill placement operation to ensure that specifications requirements are satisfied and that an adequate fill is obtained. The plan of control of the impervious earth fill for high dams must be a compromise that will result in a proper balance among all the design criteria. Experimentation with different moisture-density control procedures may be necessary to obtain proper deformation properties of earth fill (zone 1). For example, the California Department of Water Resources compaction control procedure was used at New Waddell Dam, Arizona to obtain deformation properties slightly stiffer than standard laboratory tests would have yielded.

b. Specifications Provisions. The specifications for open cut excavation require that all suitable excavated materials shall be used for permanent construction. Materials suitable for earth fill must be excavated separately and in the same manner as those fill materials excavated from borrow areas. Provisions are made for direct placement of materials in the embankment or for stockpiling.

The specifications designate which borrow areas to use for earth fill material and provide control for both location and extent of borrow activity to obtain suitable materials. Designated borrow pits within a borrow area are required to be cleared and stripped of unsuitable material and are to be irrigated or drained to obtain the proper water content before excavation. The contracting officer designates the depth of cut in all borrow pits; the contractor' s operations must be such that earth fill material delivered on the embankment is equivalent to a mixture of materials obtained from an approximately uniform cutting from the face of the borrow-pit excavation. When necessary, separation of oversized material from the earth fill is specified.

The specifications prohibit placing frozen soil in the embankment or placing unfrozen material upon frozen embankment. The maximum allowable difference in elevation between zones during construction is specified. The design engineer controls all openings through the embankment, limits the slopes of bonding surfaces, and requires careful preparation of sloping surfaces before additional material is placed. Slopes of bonding surfaces in earth fill material usually are limited to 4h:1v or flatter. Each load of embankment material must be placed in the location designated by the design engineer.

Specifications provide for placing earth fill only on dewatered and prepared foundations. All cavities within the foundation area are to be filled. Materials to be used for earth fill are described. In placing earth fill, distribution and gradation of material must avoid lenses, pockets, streaks, or layers that would prevent homogeneity of the fill. If the earth fill material contains only occasional cobbles or rock fragments, provisions are made to remove such cobbles or rock fragments over 5”from the fill before rolling. When an appreciable percentage of oversize is known to exist in the source of earth fill material, a separation plant is specified, as shown on figure 19. When a separation plant is required, separation may be specified on either the 3”or 5”size. Earth fill placement is to be in approximately horizontal layers not more than 6”in compacted thickness. Fill surfaces that are too dry or too wet are required to be reconditioned before additional materials are placed.

Moisture control provisions of earth fill require uniformity of water content throughout the entire layer before and during compaction. Water content of a soil is limited to a range based on design considerations. In general, the average water content will be near the standard laboratory optimum moisture content or, in some cases, near optimum of an alternative moisture-density control test such as the California Department of Water Resources compaction test. The contractor is required to perform all operations necessary to obtain the proper water content by irrigating or draining borrow pits, by performing supplementary sprinkling on the fill, and by working the material in the borrow area or on the fill prior to compaction. Tamping (sheepsfoot) rollers are generally required for compacting earth fill. However, many other acceptable rollers have recently been developed and used. If a roller other than the specified roller is proposed, a test fill should be constructed to ensure that the proposed roller will perform adequately. Normally, the tamping roller is still specified. The size, mass, feet arrangement, and safety features of the drums are specified. The number, location, length, and cross-sectional area of the tamping feet are described. The contractor is required to keep the spaces between the tamping feet clear. The number of passes of the roller on each layer of material is specified. Provisions are made for situations where materials are too dry or too wet. Where rolling with a specified roller is undesirable or impractical, provisions are made for special compaction by tampers or other equipment and for procedures that will obtain the same degree of compactness as the rolling process. Figures 20 and 21 show operations in placing, spreading, conditioning, and compacting earth fill.

The specifications provisions for control of placement water content and compaction must be implemented by establishing procedures to ensure their attainment. Prior to construction of earth fill, all information available from field and laboratory investigations is analyzed and reviewed in light of design considerations to formulate tentative control procedures. Special limiting moisture tests are performed on representative samples of materials to be used in high embankment dams for determining placement conditions that will prevent development of excessive pore-water pressures during construction and for settlement of the loaded soil when saturated by water from the reservoir. Based on field and laboratory investigations and on experience with similar soils, the Denver Office will recommend placement moisture limits and minimum density for representative types of materials.

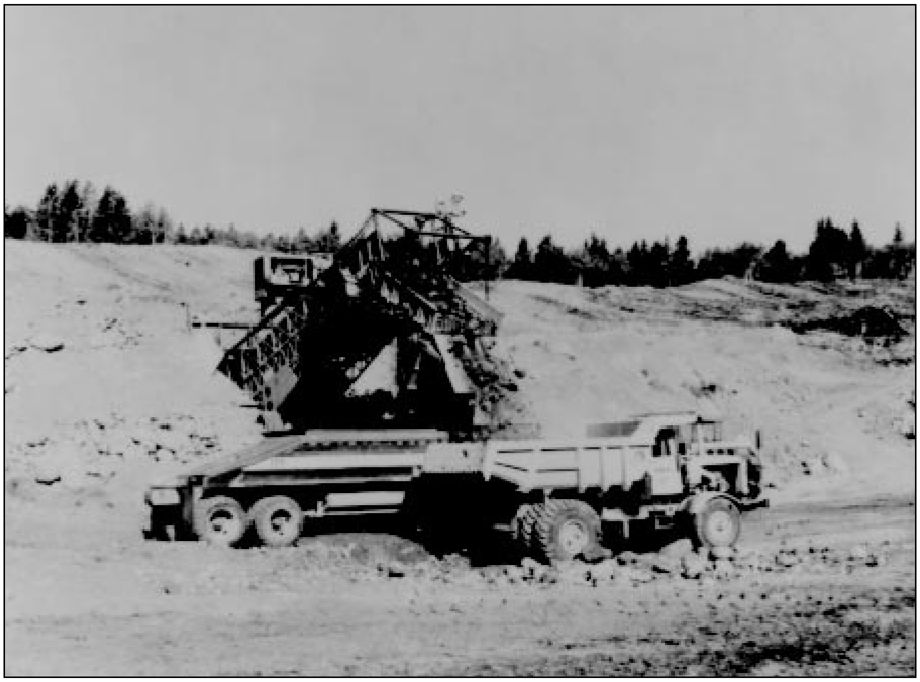


Figure 19, Typical separation plant.

c. Control Techniques. Although fill conditions are approximated by laboratory tests, it is always necessary to check planned control against actual results obtained on the embankment and then to make any required adjustments. This is done by careful observation and analysis of operations, control test results, and embedded instruments, especially during the early stages of earth fill construction.

Figure 20, Earth fill operation on an impervious zone

Figure 21, View of a Dam under construction.

In structures where unusual conditions are present, the specifications may include provisions for the contractor to construct one or more test sections of earth fill material. The purpose of a test section is to determine the most effective excavating, processing, placing, and compaction procedures for representative soils under jobsite conditions. By varying placement procedures within certain limits, by exercising rigid control over the relatively small volume of the section, and by maintaining complete, accurate records of tests, the most applicable procedures for the rolled earth fill portion of embankment may be determined during the initial stages of construction. Results of field density tests made on test sections will provide the necessary information for establishing construction control procedures consistent with design requirements. Figure 22 shows a test section under construction using a pad foot roller to compact zone 1.

Figure 22, Pad foot roller on a test fill of zone 1 material

After proper placing procedures have been determined from the embankment test section or from initial placing operations when test sections are not required, construction can proceed at full scale. Throughout construction, the contractor' s operations must adhere strictly to limits and conditions set forth in the specifications and to recommendations made by the design engineering staff. A complete record of all operations must be kept so the most efficient methods may be perfected. Relationships between laboratory compaction curves and field roller curves obtained by analysis of control test results are shown for three dams on figure 23.

The most important variables affecting embankment construction are:

- Selection of materials in the borrow area
- Distribution of materials on the embankment to obtain uniformity
- Placement water content, and uniformity of this moisture throughout the spread material
- Water content of borrow material
- Methods for correcting borrow material water content if too wet or too dry
- Roller characteristics
- Number of roller passes
- Thickness of layers
- Maximum size and quantity of rock in the material
- Condition of the surface of layers after rolling
- Effectiveness of power tamping in places inaccessible or undesirable for roller operation

To maintain control of earth fill construction, an adequate number of inspection and laboratory personnel are essential. For each shift, at least one inspector should be on the embankment and one technician in the laboratory. Borrow operations on small jobs may be inspected by the embankment inspector. On a large project, an inspector is required at the borrow pit, especially when moisture control is critical. On large projects, additional inspectors and technicians may be required in all areas.

The borrow-pit inspector selects the areas in the borrow pit to be excavated, determines the depths of cut, and specifies the zone of the embankment where a particular material should be placed according to an approved distribution plan. The inspector checks adequacy of any mixing or separation methods performed by the contractor and, as required, the inspector cooperates with the contractor in determining the amount of moisture to be added (or removed) to the borrow pit to attain proper water content in the materials prior to placement. Moisture content determinations of borrow-pit materials should be made well in advance of excavation so that corrective measures can be taken promptly. As a rule, to adjust to the final moisture content in the borrow pit is impractical because of potential changes in water content caused by (1) rain, (2) mixing or separation operations, and (3) evaporation. However, an effort should be made to have the excavated material as close as possible to the desired water content before delivery to the embankment. In some cases, specifications will require that no more than 2 to 3 percent water can be added on the fill.

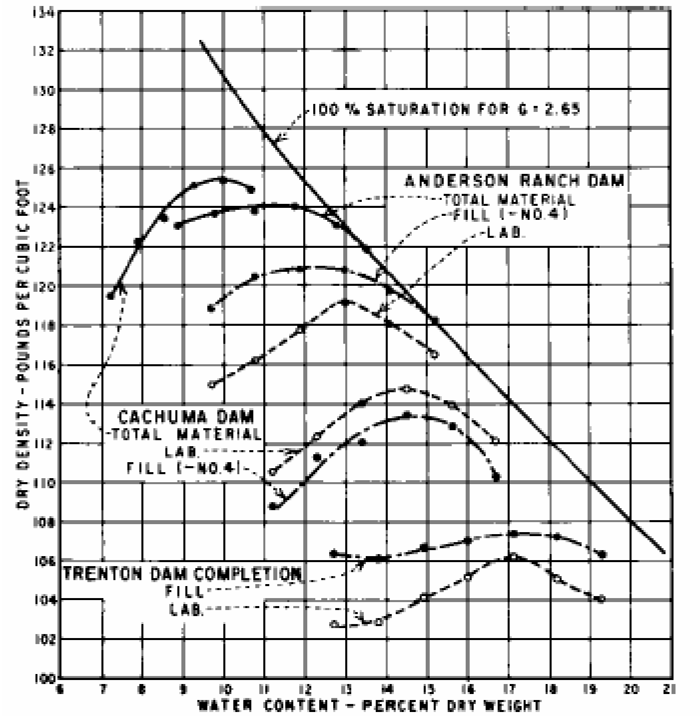


Figure 23, Average field and laboratory compaction curves.

The foregoing inspection principles also are applicable to structural excavations where materials are to be placed directly in the embankment or are to be first processed or stockpiled. Figure 24 shows the cut face of a borrow pit where mixing of soils is accomplished during excavation; figure 25 shows another method of excavating that produces a good mixture from a layered deposit. Also, figure 3, shows a cut face of a borrow pit in which controlled excavation allowed material from the same pit to be used for two separate embankment zones, pervious and impervious.

The embankment inspector should determine the location and elevation of tests made on the embankment and record the location of the contractor' s operations. Horizontal control, or station and offset, should be established. Vertical control can be benchmarks and hand levels and stadia rods used to establish elevations on the fill. When materials are brought onto the embankment, the inspector must verify placement in the proper zone, as indicated by the approved distribution plan. Lines of demarcation can be painted on rock abutments or marked by colored flags.

After materials are placed in the correct location, the embankment inspector determines if the material contains the proper amount of moisture before compaction. When materials arrive on the embankment too dry, water can be added by sprinkling before, during, or after spreading. The contractor' s operations in sprinkling and mixing water with soil will vary. However, regardless of the method used, the proper water content must be uniformly distributed throughout the layer before compacting.

Another important inspection task is to determine the thickness of the compacted layer. A layer spread too thick will not have the desired density for given conditions of compaction. For initial placing operations, the test section will have determined the proper spread thickness of a layer which will compact to specified thickness; this specified compacted thickness should never be exceeded. A method for determining average thickness of placed layers is to plot (daily) a cross section of the fill at a reference Station(s). The inspector's report for that day will contain the number of layers placed at that section; hence, average thickness can be determined.

When oversize rock content is greater than about 1 percent, removal of oversize rock from earth fill embankment material is best accomplished prior to delivery of the soil on the embankment. Smaller amounts of oversize rock can be removed by handpicking or, under favorable conditions, by various kinds of rock rakes. Generally, oversize rock, which had been overlooked before rolling, can be detected by the inspector during rolling by observing a bounce when the roller passes over a hidden rock. The inspector should be sure that the contractor removes the rock from the fill.

Figure 24, A Holland loader excavating a vertical face mixing silt and fine sand deposits for impervious fill.

Figure 25, Two model 992C Caterpillar front end loaders loading zone 4 (gravel and cobble fill) from borrow into a model 540 Dart belly dump truck.

The inspector must ensure that each lift receives the specified number of roller passes. An oversight in maintaining the proper number of passes may lead to undesirable low densities. Insistence upon orderliness of fill procedures and the establishment of routine construction operations will help to achieve the required number of roller passes.

The final check on the degree of compaction is determined by field density tests and the rapid method of construction control. If the degree of compaction of material passing the No. 4 sieve is at or above the minimum allowable percentage of laboratory maximum dry density and the water content is within allowable limits, the embankment is ready to be scarified and moistened, if necessary, to secure a good bond between layers before the next layer is spread. Time required to obtain results of the field

density test by the rapid method should be about 1 hour or less.
Mechanical tamping, used around structures, along abutments, and in areas not accessible to rolling equipment, should be carefully observed and checked through use of frequent density tests. The procedure to be followed for mechanical tamping depends greatly on the type of tamper used. Some factors affecting density are:

- Thickness of layer being tamped
- Time of tamping
- Water content of the material
- Mass of tamping unit
- Condition of tampers
- Air pressure (if air tampers are used the manufacturer' s recommended pressures should be maintained)

The embankment inspector must be alert for areas of low density and have them remedied by sprinkling, scarifying, removing, or rerolling, as required. An important function of inspection is to determine when and where to make field density tests. These tests should be made:

- In areas where the degree of compaction is doubtful
- In areas where embankment operations are concentrated
- For every 2,000 yd^3 of embankment when no doubtful or concentrated areas occur
- For representative tests of every 30,000 yd^3 of earth fill placed
- For "record" tests at all embedded instrument locations
- At least once during each work shift

Areas of doubtful density are sometimes detected by the inspector' s observations. Insufficiently compacted locations may include the following:

- The transition between areas of mechanical tamping,or special compaction, and rolled earth fill embankment along abutments or cutoff walls
- Areas where rollers turn during rolling operations
- Areas where too thick a layer is being compacted
- Areas where improper water content exists in a material
- Areas where less than specified number of roller passes were made
- Areas where dirt-clogged rollers are being used to compact the material
- Areas where oversize rock has not been removed from the fill
- Areas where materials with minor amounts of frost have been placed, or were placed at nearly freezing temperatures
- Areas that were compacted by rollers that have possibly lost part of their ballast
- Areas containing material which differ substantially from the average

The number and location of field density tests should be such that the extent of the doubtful area is determined. All tests made in areas of doubtful density should be identified by the letter D.

When an embankment operation is concentrated in a small area, i.e., if many layers of material are being placed one over the other in a single day, tests should be made in this area in every third or fourth layer to ensure the desired density is attained. In such an area, all tests should be identified by the letter C.

Even if areas of doubtful compaction do not exist or tests are not required because of other reasons, at least one field density test should still be made for each 2,000 yd^3 of compacted embankment, and it should be representative of the degree of compaction being obtained. Such tests should be identified by the letter R.

Regardless of the number and purpose of other field density tests, for large dams, one test is required for each 30,000 yd^3 of earth fill placed; the tests should be representative of the conditions of the fill. Percolation settlement tests and specific gravity tests are made on the same sample; the tests should be identified by the letter R ("record"). Additional material must be excavated adjacent to the field density test hole to obtain enough soil to make these additional tests.
"Record" tests are made at all instrument installations and are recorded separately.

Test pits should be excavated periodically during construction to visually check the fill for:

- Uniformity
- Laminations
- Contact areas

- Moisture content
- Density
- Interface zones

The effect of rolling and of superimposed loading is evaluated by making field density tests as the test pit is excavated. The degree of uniformity of moisture content with depth is found by testing samples from various depths. Visual inspection indicates whether successful bonding of layers had been accomplished. As shown in figure 26, undisturbed samples should be taken periodically for laboratory testing to determine engineering properties and so comparisons can be made to values used in the design. The pit should be photographed. During a seasonal shutdown period, test pits, as shown in figures 26 and 27 should be excavated in the compacted embankment to determine the net result of the season' s operations. Any unusual conditions observed in the embankment test pit hole should be reported.

21. Compacted Pervious Fill.

a. Design Considerations. Permeable materials are used in rolled embankment dams in internal filters and drains and transition zones to control seepage and piping and in outer shells to provide high strength to support the impervious core. Usually, pervious fill is free-draining cohesionless sand and gravel containing less than about 5 percent fines (material passing the No. 200 sieve). Four criteria must be satisfied in controlling construction of zones of sand and gravel:

- The material must be formed into a homogeneous mass free from large voids.
- The soil mass must be relatively free draining depending upon its intended use.
- The material must not consolidate excessively under the mass of superimposed fill.
- The soil must have a high angle of internal friction.

If chimney drains containing filter and drainage layers are not used (recently, chimney drains almost always are a requirement), homogeneity of the shell is important to ensure distribution of the inevitable seepage from the impervious zone throughout the length of the dam since concentration of seepage water into a few channels may induce dangerous piping. The permeability of the permeable zone must be much greater than the impermeable zone. In addition, permeability of the free-draining zone should be high enough to preclude development of construction pore-water pressures in this zone. The requirements of low consolidation and high shear strength for a permeable zone can be obtained by properly compacting the materials. Because excessive pore-water pressure is impossible to develop in pervious free-draining materials, the maximum practicable compaction of such materials is desirable.

Figure 26, Block sample from test pit in zone 1. Final saw cut being made beneath the sample with a chain saw equipped with special tungster carbide bit.

Figure 27, Test pit at the interface of a sand filter and gravel drain.

b. Specifications Provisions. Specifications on opencut excavations provide that suitable materials shall be used for permanent construction. Provisions are made for temporarily stockpiling such materials. When permeable soils are to be obtained from borrow pits, provisions for compacted earth fill materials apply equally for these soils, except that irrigation of borrow pits is not required.

Specifications prohibit use of frozen soil and do not allow placing unfrozen material on frozen embankment. These provisions are applicable to all kinds of soils. Provisions for maximum allowable difference in elevation between adjacent zones control placement of permeable material. Control of openings through embankments, slopes of bonding surfaces, and preparation of bonding surfaces apply equally to all zones within the embankment. Distribution of material within a permeable zone is controlled by the inspection staff.

Written directives are in the specifications pertaining to permeable material used as a zone in the embankment. The material will be described and its source designated. Allowable thickness of the layer will be specified, either in the form of compacted thickness or placed thickness. Vibratory steel-drum rollers in the mass range of 5 to 10 tons are the best equipment for compacting pervious clean coarse-grained soils. Drum-drive self-propelled vibratory rollers may be effective on fine uniform sands when other vibratory rollers are not. Rubber-tire rollers may be used if they can produce the required densities. At times, track-type equipment are used effectively in rough areas or in confined areas where a vibratory roller cannot operate effectively, for example, in compacting a horizontal drainage layer on an irregular foundation. The contact pressure of the equipment should be at least 9 lbf/in^2 and should operate at a speed which imparts greatest vibration to the fill. Track-type equipment should be discouraged for compacting drainage blankets and filter drains because of potential breakdown of the soil particles. Sometimes, alternate methods of compaction are permitted by the specifications. In such cases, a minimum relative density requirement is specified.

c. Control Techniques. Workability of a permeable soil is reduced considerably by the presence of even small amounts of silt or clay. The contractor' s operations in the borrow pits and on the fill must be such that contamination of the pervious soil is minimized. As the material is brought on the fill, it must be directed to the proper zone. Within pervious shell zones, individual loads should be directed so coarser material will be placed toward the outer slopes. This does not apply to filter or drain material in chimney drains or blankets.

When compacted thicknesses are specified, the required thickness of the loose layer, which will result in the specified final thickness, must be determined by the inspector during initial construction stage. Since in-place density will be checked by relatively few tests after satisfactory placing procedures have been established, the required thickness of the loose layer must be maintained within close limits throughout construction. The specified thickness of the compacted layer is usually great enough to accommodate the maximum size of rock encountered in the borrow. In cases where cobbles or rock fragments occur that are greater in size than the specified thickness of layer, provisions are made for special embedding, for removal to outer slopes of the pervious zone, or for removal to other zones. The inspector must ensure that provisions for disposal of oversize rock are followed.

After material has been placed, spread to the desired thickness of lift, and oversize cobbles or rock fragments removed, the next important step is application of water. Thorough and uniform wetting of materials during or immediately before compaction is essential for best results. The best method for adding and distributing water on the fill should be determined during initial placement. Relaxing requirements for thorough wetting may result in densities far below the minimum requirement, even with great compactive effort. Different pervious materials require different amounts of water for thorough wetting and best compaction. In general, adding as much water to the material as it will take is desirable. Too much water cannot be added to an extremely pervious soil; however, permeable soils containing small quantities of silt or clay may become temporarily boggy if an excessive amount of water is used. For these soils, care must be exercised in wetting the soil; the contractor' s operations must be carefully controlled to avoid creation of temporary boggy conditions.

Vibratory compaction units should be checked frequently to ensure they are operating at a frequency which produces the highest possible density. For cohesionless materials, the frequency of vibration generally should fall between 1,100 and 1,500 vibrations per minute. If the track-type equipment is used, operating at the highest practicable speed is desirable during the compaction operation. High speed is conducive to greater vibration and aids compaction. When inspecting compaction made by track treads, ensure that the operator makes complete tread coverage of the area to be compacted before making the second and subsequent passes.

During initial placing operations, relative in-place density tests and gradation analyses are recommended at a frequency of one test to represent each 1,000 yd^3 placed. After placement procedures have proved satisfactory, and unless significant changes in gradation occur or doubtful conditions occur, one relative density test for every 10,000 yd^3 of material placed will suffice. In the event of significant gradation changes in borrow materials, increased frequency of field tests may be needed to ensure satisfactory compaction of the variable materials.

22. Semipervious Fill. Semipervious materials silty sands or gravels (SM or GM). Sands with dual symbol classifications such as SW-SM, SW-SC, SP-SM, or SP-SC [i.e., sands having as high as 12 percent passing the No. 200 sieve (5 percent is the usual upper limit for a material to be classified as pervious)] may have the characteristics of semipervious material even though such materials may be allowed by the specifications in embankment zones designated "pervious fill." Generally, compaction curves indicating adequately defined optimum water contents and maximum dry densities can be developed using standard compaction tests on impervious and semipervious materials.

Also, in some dams, outer embankment zones composed of coarse-grained sands and gravels, which contain appreciable amounts of fines (i.e., > 5 percent), are sometimes designated as "pervious zones." Compaction equipment, procedures, and control for such materials can be either those presented for pervious fill or those presented for impervious fill. The decision to use which control method depends on whether the soil behaves like a pervious material or more like an impervious material.

23. Rockfill. Embankments with large rockfill zones are becoming more common. This is primarily because of:

- Necessity for using sites where rock foundation conditions are unsuitable for concrete dams
- Suitability of modern construction equipment to handle rock
- Increasingly higher dams being constructed
- Economic benefit that is obtained by maximum use of rock from required excavation.

Rock that does not easily break during handling, transportation, and compaction is considered sound and results in a pervious fill. Such sound rock is desirable for rockfill dams, or rockfill zones, especially if the source is from required excavations such as a spillway.

a. Sound Rock in Rockfill Dams.

1. Specifications. The specifications for pervious rockfill sections generally require that rock be sound, well graded, and free draining without specifying gradations; but, the maximum permissible size is specified together with lift thickness. Normally, rock fill is placed by dumping from trucks; bulldozers spread the material to the desired lift thickness. The required placement and spreading operations should avoid segregation of rock sizes. Type of roller, lift thickness, and number of passes are specified, preferably based on results of test fills.

2. Placement Operations. When rock is dumped on the fill surface and pushed into place by a bulldozer, the fines are moved into the upper part of the lift, thereby creating a smoother working surface for the compacting equipment. If a layer of fines of significant thickness is produced that might seal off the upper part of the lift and prevent distribution of fines throughout the lift, the rock may be dumped directly in place.

All oversize rock must be removed before compaction. Usually, this is done with bulldozers fitted with special "rock rakes." Oversize rocks are often pushed into a specified zone in the outer slopes. Sometimes, excessively large rocks are hauled off to be dumped elsewhere or they are broken with a drop weight or explosives and used in the rockfill or riprap zone. Another common way of breaking oversize rock is with hydraulic chisels or spliters. Oversize rocks should never be allowed to accumulate along the contact slope of a closure section.

Close inspection is required to ensure that the material does not contain excessive fines. Excessive fines can cause excessive postconstruction settlements when the reservoir is filled. Specifying a limited amount of fines in the specifications is difficult; consequently, this is rarely done. However, the design office should provide guidance to field personnel based on results from rock-test fill studies which will aid in determining if excessive fines are present. The design and construction staff must be alert for material variations that could result in undesirable changes in gradation of the material brought to the embankment. If this occurs, the contractor should be informed so a change can be made in quarrying techniques.

3. Compaction. Current practice is to use smooth steel-wheel vibratory rollers to compact all sound rockfill in comparatively thin lifts with minimal application of water. The lift thickness specified is dependent on size and type of rock and on type of compaction equipment used; usually, thickness is determined from results obtained during construction of a test fill. Less desirable rocks may break down in varying degrees upon excavation, handling, and compaction. These unsound rocks may degrade because of:

- Lack of induration
- Structure (such as thin bedding)
- Degree of fracturing
- Weathering
- Other physical and chemical properties

Some examples of material to be avoided are:

- Shales
- Mudstones
- Siltstones
- Claystones
- Chalk
- Earthy limestones
- Other sedimentary rocks which may be poorly cemented

Ordinarily, sound rocks that are quite weathered and very fractured may not be acceptable for rockfill zones because of their susceptibility to breakdown caused by weathering and fracturing.

Because of general unpredictability of breakdown of most types of rocks, use of test quarries, test fills, and laboratory tests for designing and constructing rockfill dams or zones is widespread. Generally, the lift thickness specified will be no thicker than 24" unless test fills show that adequate compaction can be obtained using thicker lifts. The maximum particle size allowed in the lifts should not exceed 90 percent of the lift thickness.

Scarification of compacted lift surfaces is not necessary and should not be allowed because it disturbs the compacted mass.

b. Unsound Rock in Rockfill. Formerly, use of soft or unsound rocks has been dictated by availability of large quantities from required excavations. The main concern about these materials is the tendency to weather and soften with time when exposed to air and water within the embankment. However, dams with large portions of embankments composed of unsound rocks attained adequate shear strength, and the rock experienced insignificant breakdown after placement when the rock was placed in random and semipervious zones. Where unsound rocks will constitute a significant structural portion of a fill, their properties and the best methods for compaction should be determined by means of a test embankment constructed during design studies. Some rockfills composed of unsound rocks have been compacted by first rolling the loose lift with a heavy tamping roller equipped with long spike or chisel-type teeth ("shale breaker") and then compacting the lift with conventional tamping or rubber-tire rollers.

24. Riprap. Riprap is the commonly specified material for slope protection. Properly graded riprap, placed to provide a well-integrated mass with minimum void spaces so that underlying bedding cannot be washed out, provides excellent slope protection.

The primary factors that govern successful construction are:

- Loading from the quarry to provide a good mixture of different sizes within the required gradation in each load. Proper production from the quarry requires that blasting operations or processing produce proper rock sizes and that loading operations are properly inspected. Gradation testing is often specified. Grading of riprap and other large stone can be performed in accordance with ASTM procedure D5519-94 (see sec. 8b). Gradation tests are best performed at the placement site by sampling several trucks. Hauling vehicles should be weighed for determination of tonnage hauled.
- The rock should be placed on the slope to provide uniform distribution of different sizes without segregation to provide a rock mass without large voids. Placement should be accomplished by placing loads along the slope against previously placed riprap; this reduces size segregation which otherwise occurs if loads were dumped in separate piles. Dumping rock at the top of the slope into a chute should never be allowed because this results in segregation. If dumping is done from trucks it is usually necessary to winch load haulers down the slope to the placement location. Dumping should proceed along horizontal rows and progress up the slope; loads should not be dumped to form rows up the slope. If large rock is specified, a crane with an orange-peel or clam shell attachment operating from a platform built on the slope can be used. Recently, large backhoes have been used to place riprap; experienced equipment operators can do a good placement job. In using backhoes, riprap must be placed as the embankment is constructed because riprap placement must stay within about 5 m (15 ft) of the top of the fill. The process of end dumping without manipulation is generally not sufficient for good placement. Reworking with backhoes or other acceptable equipment will be required. Close visual inspection or actual measurement is required after dumping and spreading riprap to determine the adequacy of the distribution of different sizes. Reworking may be minimized by exercising care when loading to ensure that each truck load has the proper amount of each size rock (i.e., the proper gradation). Placement should result in a low void ratio, on the order of 0.35 or smaller. The void ratio over a placement area can be estimated by using mass tickets from each truck and the area of placement. If the volume of an area is considered a solid volume, then the void ratio is determined by proportion of the solid volume and the actual mass. For example, assume an area of 100 by 100 by 3 feet. Assume the density of riprap as 160 lb/ft^3 (or use the specific gravity of the rock). The solid volume is then 160x100x100x3 = 4,800,000 lb = 2,400 tons. If the mass tickets equal 1608 tons, the void ratio is (2,400- 1,680)/2,400 = 0.33. This method of void ratio ensures a tight structure but does not ensure good distribution of grain sizes. Strict enforcement of specifications is required, especially during early stages of riprap placement to ensure a well-graded mass without large voids.
- A bedding layer (or layers) designed as a filter must be provided between the riprap and embankment material to protect the embankment material from erosion by wave action and to provide a stable base for riprap. Many riprap failures have occurred because bedding material was not large enough to preclude washing out through riprap interstices by wave action. Removal of any bedding causes settlement and dislodgement of overlying riprap, which further exposes bedding and embankment material to direct wave action. Good practice is to place rock spalls or crushed stone of like size between the bedding and the riprap if they are available from quarry or required excavation. The term "spalls" refers to the finer materials resulting from rock excavation for materials such as riprap. Spalls must be durable fragments of rock, free of clay, silt, sand, or other debris. Spall gradation varies and must be specified for each particular job. End dumps with spreader boxes provide the best means of placing bedding and spalls.

25. Miscellaneous Fills. Dam embankments on very plastic foundations may require toe berms to improve stability. Excavations for dam foundations or for appurtenant structures often produce material unsuitable for, or in excess of, requirements for the structural zones of a dam. Such excavated material can be used in toe berms of the dam. In localities where good quality riprap is expensive, fill materials from structural excavations have been used to flatten the upstream slope of the dam to permit use of lesser quality riprap and, in some cases, no riprap at all. In a few cases, excess low quality material from required excavation has been used in a noncritical zone, in the downstream portion of a dam, merely to replace material which otherwise would have had to be borrowed at greater expense.

Permeability of miscellaneous fills is not important in the design; they may not specifically require compaction. However, full use should be made of compaction obtainable by routing hauling and placing equipment on placed layers of the material. Sometimes, the nature of available materials or the design may require that some compactive effort other than routing of equipment be used. For example, sheepsfoot rolling has been used to break up fairly large pieces of soft rock to avoid excessive settlement. Compaction should be specified if the miscellaneous fill is to function as an impervious blanket or is within the limits of the embankment.

For miscellaneous fills, specifications should describe the materials to be used and state their sources. Thickness of layers will be specified, and the requirement for routing construction equipment over each layer will be stated. The specifications will also state if any moisture or special compaction requirements are needed.

Inspection of miscellaneous fills is entirely visual. Ordinarily, control tests are not made on such fills. A few fill density tests may be required for record purposes, and instruments may be placed to obtain performance characteristics. When inspecting miscellaneous fills, it is important to ensure that specified lift thickness is not exceeded; also make sure that hauling equipment never concentrates on a roadway but is routed over the entire breadth of the placed layer.

26. Instrument Installations. The behavior of many embankment dams is monitored during construction and throughout operation by instrumentation placed in the embankment, in the foundation, and in or on appurtenant structures.

a. Instruments. Data obtained from the instruments are used to assess performance of the structures by comparing the results to design assumptions and to normal performance expectations of embankment dams and their foundations. Besides visual inspections, performance monitoring with instrumentation is used to ensure safe, longterm performance of dams.
The various types of instrumentation installations can be grouped into five categories:

- Devices to observe pore-water pressures in the embankment and in the foundation
- Devices to observe earth pressures in the embankment
- Internal and external devices that provide data on deformations and displacements of the embankment and foundation
- Devices for monitoring seepage flows
- Vibration recording devices to indicate the result of transient stresses on the embankment

Type and quantity of instruments installed at a dam will vary with:

- Design
- Purpose
- Foundation conditions
- Embankment materials
- Embankment size

b. Installation of Embankment Dam Instrumentation. Work required for installing the instruments is usually performed by the contractor under provisions of the specifications for construction or modification of a dam.

c. Inspection. Inspectors are required to inspect and test all the equipment, to oversee all installation work, and to obtain instrument readings during construction. Close inspection is essential to ensure correct installation of equipment and proper operation of the instrumentation.

d. Record Tests. Record tests of embankment and foundation materials at internal instrument installations are necessary to obtain data on soils adjacent to the instruments. One field density test should be made at each unit or element of displacement or deformation monitoring device in earth fill material. Two test samples should be obtained from soil near the element or unit as well, one in the rolled embankment near the element and one in specially compacted material placed around the unit. When all or part of a dam is founded on highly compressible materials, representative undisturbed samples should be obtained in the foundation at the locations of the vertical movement and baseplate installations unless suitable samples have been obtained previously. Tests are not required when these devices are placed on a rock foundation unless specifically requested. Tests at the elements or units should be designated "record rolled," "record tamped," or "undisturbed," depending on the material being tested.

One field density test is required near each embankment earth pressure cell and piezometer tip. This test should be made in the rolled fill at the piezometer tip location prior to excavating the offset trench for the tip for closed-system piezometers. Each hole drilled for foundation piezometers should be logged throughout its length, and a record sample of the material should be obtained at the depths where piezometers will be located.

In addition to field density tests on embankment materials and in-place density tests on foundation materials, the following four tests should be performed on the samples taken:

1. Gradation analysis
2. Specific gravity
3. Liquid and plastic limits
4. Percolation-settlement

This laboratory testing may be performed during seasonal shutdowns when workload permits. Sufficient material must be obtained while making the density tests so that additional tests can be made. Before testing, foundation samples should be sealed to prevent moisture loss. Special care should be exercised to attain desired accuracy in all record tests.

Embankment field moisture and density conditions must be duplicated for the percolation-settlement test and in-place conditions must be maintained for foundation materials. During and after construction, the dam designer may request that some record test samples or other special samples (core, blocks) be sent to special laboratories for testing and that the remaining samples that are not moisture sensitive be permanently stored at the damsite, in accordance with standard guidelines. All stored samples should be properly marked and referenced to the pertinent density test.

These identifying symbols should be marked on the container in addition to identification normally required on record samples. Record samples obtained from the location of foundation piezometers should contain representative materials taken from the location of the tip of the piezometer. Quantity of sample will depend on drilling equipment used to excavate the holes.

27. Records and Reports. The inspector must make daily, periodic, and final reports covering construction activities work.

a. Daily Reports. The daily reports should record the progress of construction, provide pertinent information for the inspector (who is beginning a shift) about shutdowns, construction difficulties, and tests in progress. These reports will also furnish data for use in compiling the construction progress report. The form of a daily report will vary to suit the requirements of each job, but all information in the periodic earthwork progress report should be based on day-to-day records. A consistent, systematic method is desirable for identifying field density tests made on the embankment. Results of tests made daily on the embankment are reported in the progress report. Each test should be designated by the date, shift, number on that shift, and purpose.

b. Periodic Progress Reports. Adequate technical control of construction is an important and essential function of field forces on an earthwork embankment project. Complete records of all data pertaining to methods and procedures of achieving satisfactory control must be maintained as a normal part of the inspection. Construction reports are required of each project, for each month during the working season that embankment is being placed, except in special cases additional summaries or reports may be required. Copies of the Construction report may be required for review by design team members. A Summary of Construction report consists of:

1. Narrative and Photograph. The narrative shall consist of a description of earthwork operations during the reporting period. Important features of normal embankment operations to be included in the narrative of initial reports are outlined below:

- Borrow-pit operations (each borrow area):
- Description of material
- Equipment used
- Natural water content
- Method of adding moisture
- Depth of cut Stratification Mixing, separating
- Transporting
- Embankment operations (each zone):
- Equipment used
- Spreading, mixing
- Method of adding moisture
- Maximum size of rock fragments
- Method of removing oversize rock fragments Compaction method
- Thickness of layers

To avoid repetition, many items reported in the above outline should be described only once. Thereafter, only significant changes in materials or in methods and procedures should be described. Any difficulty or unusual conditions encountered should be fully described. The narrative should clarify any results reported on the various forms that require special mention, and it should report pertinent information not covered by the forms.

Photographs should be taken far enough in advance to ensure no delay in submitting the reports and should include:

(a) A view from an abutment point showing the extent of operations on the dam during the period of the report. The camera position should be located sufficiently high so that location can be used until the dam is complete.
(b) An overall view showing the extent of operations in each major source of material.
(c) Closeup views of each operation such as dumping, spreading, adding moisture, disking, rolling, or mechanical tamping. Photographs of these operations need be submitted only once. Thereafter, only photographs showing different or

unusual procedures should be submitted.
(d) Closeup views of typical borrow-pit cut banks. These should be submitted once.

c. Final Embankment Construction Reports. Upon completion of embankment construction, a report should be prepared summarizing work accomplished. Final Construction Reports are required to be prepared in duplicate and submitted to the Denver Office for all major works, including dams. A detailed summary of embankment construction need not be transmitted separately but should be included in the Final Construction Report. A suggested outline for the embankment portion and related construction requirements of the report follow. Items in the outline not applicable to a particular project should be disregarded, and variations or additions should be made to suit individual job conditions.

1. General.
    - (a) Location and purpose of structure.
    - (b) Description of dam and appurtenant works, including dimensions and quantities involved.
2. Investigations.
    - (a) Foundation explorations.
        - (1) Description of foundation conditions.
        - (2) Itemized summary of foundation explorations showing number, depth, and coordinate location of: drill holes, test pits, shafts, and tunnels.
        - (3) Description of any special tests and methods of taking undisturbed samples.
    - (b) Construction materials.
        - (1) If a preconstruction earth materials report were prepared, a general description of materials investigation should be prepared and reference should be made to the previously prepared report. However, a complete and detailed description of additional investigations made subsequent to the preconstruction earth material report is required.
        - (2) If the preconstruction earth materials report was not prepared, a complete and detailed description of material investigations of borrow areas and rock sources with test results and quantities available shall be included in the Final Construction Report.
3. Construction history (include photos within text.
    - (a) Required excavations, except borrow areas.
        - (1) Stripping of foundation(s).
        - (2) Cutoff trench(s).
        - (3) Grout cap trench.
        - (4) Miscellaneous excavations.

The construction history shall include discussion on depth, quantities, dewatering methods, disposition of materials excavated, and type of equipment, including number used and rated capacity.

- (b) Foundation drilling and grouting.
    - *(1) General*. Include discussion on type of equipment used and capacity. For the grouting plan, show location and depth of holes.
    - *(2) Pressure grouting*. Include discussion of:
      grouting procedures, type of equipment and pressures used, grout take and results obtained, and any special problems and methods used to obtain satisfactory results.
- (c) Borrow area operations.
    - *(1) General*. Discussion of plan and use of specified material sources and any problems in borrow areas not anticipated at the time specifications were issued.
    - *(2) Borrow areas*. Detailed report of: initial conditions of borrow pits, methods of excavating, mixing, screening, and transporting of material. Include: type, size, and rated capacity of all equipment used—with special emphasis on any unusual requirements for blending or excavating procedures. Also, note: borrow pit yield, methods of controlling moisture, disposition of oversize material, and material from stripping operations.
- (d) Embankment operations.
    - *(1) Zones*. Discussion to include: construction methods and sequence, material source, and any variation from the approved materials distribution plan and reasons therefore. For riprap and rockfill, record gradation.
    - *(2) Placing*. Description of methods used including:
      scarifying, spreading and leveling, rolling, power tamping, removal or reworking, and any other part of the work considered peculiar to the placing operation.
    - *(3) Equipment*. Discussion of type, capacity, size, number used, efficiency, and suggested improvements (if any) for all equipment used in placing embankment.
    - *(4) Testing*. Performance of field laboratory, including number of testing personnel and number and type of tests performed should be summarized. Discussion of control tests, such as moisture content and density, and any special tests should include: procedures in making tests and reporting methods, extent of testing, adequacy of tests, and

difficulties encountered.

4. Embankment Dam Instrumentation. Brief description of systems installed such as piezometer tips, horizontal and vertical movement crossarms, and embankment and structure settlement measurement points.

Embankment Dam Instrumentation Reports. A monthly report, separate from other construction reports, should be required during construction of a dam to present progress of placement of embankment dam instrumentation. This report should include:

- A short narrative on progress of work
- As-built information
- Reports of problems or abnormalities during installation work
- Readings made during the preceding period c. Representative photographs showing current details of the work

Within 6 months after completion of instrument installations, a Report on Embankment Dam Instrumentation is required and should include the following:

- Document materials and equipment actually used in instrumentation installations.
- Records of preinstallation testing of equipment.
- Drill logs for holes in which instruments were installed.
- Record test information.
- Problems or abnormalities that occurred during instrument installations.
- Deviations or modifications from specification requirements relative to instrument installations.
- Contractor claims that arose relative to instrumentation work.
- Photographs as available.
- Complete as-built information.
- Comments, criticisms, and suggestions so that future instrumentation design work can be improved.
- All instrumentation readings taken during construction.

28. Control Criteria. The concept of limiting the moisture condition for placement of compacted earth fill in dam embankments has been used by the government for many years. The idea of limiting the magnitude of pore-pressure buildup in cohesive soils during construction, by controlling the placement moisture, has been a major consideration since 1946. The relationships among moisture control, pore-water pressure potential, and stability of the structure are well established and are generally recognized as a basis for an upper placement moisture limit.

Localized shear planes may develop under certain conditions of placement moisture and compactive effort. Specific procedures for compaction of earth fill should be developed, these shear planes occur when the soil is at optimum water content or wetter when construction pore pressures normally increase rapidly with increased moisture content. A typical example of laboratory tests on one soil is shown on figure 29. The net effect of development of localized shear planes would be a significant loss of shear strength; hence, placement moisture should be so controlled as to avoid any possibility in developing local shear planes.

The phenomenon of loss of effective strength of dry, low-density soil as the water content is increased is also well known. This reduction in strength may result in appreciable volume change of a loaded soil, either in the foundation or in the compacted embankment. In the case of embankment dams, wetting the foundation or the embankment following construction does not occur uniformly. Hence, such volume changes usually will result in differential settlement and may result in cracking of the embankment. Many failures of small embankment dams are attributed to cracking due to differential settlement. Placement moisture should be maintained in a range between the specified upper and lower moisture limits so that the soil will not exhibit additional consolidation upon saturation, will not develop excessive pore pressures, and will not develop shear planes under field compactive effort. Generally, this range is from about 2 percentage points dry to 1 percentage point wet of optimum moisture content.

High shear strength, low consolidation, and permeability are all related to dry density obtained in a fill. Design is usually based on impervious or core material being placed at or slightly below average laboratory maximum dry density of the minus No. 4 fraction. An average D-value (ratio of dry density of fill to laboratory maximum dry density, expressed as a percentage) near 100 percent is expected for most soils containing less than about 30 percent plus 4.75 mm material. For gravelly materials containing more than about 30 percent plus 4.75 mm material, an adjusted field density of the control fraction less than laboratory maximum dry density of the minus 4.75 mm fraction may be expected during construction and is considered in design.

Overall quality of the embankment being placed can be evaluated by statistical analysis of test results. A simple frequency distribution analysis of the variables $_{wo\text{-}wf}$ and D-value, as outlined on table 2, can be maintained throughout construction to aid engineers and inspectors in their efforts to control placement within recommended or specified limits. Figures 30 and 31 show worksheets designed for such an analysis. Forms of this type, on which data from each test are entered as results become available, should be maintained in the laboratory or in some other suitable place where they will be readily available for inspection. By means of this analysis, the frequency distribution of test results is obtained, and statistical values such as the mean, the standard deviation, and the percentage of tests falling outside specified limits can be determined. Computer software programs are available which calculate statistical information on test data such as average D-value, average percent of accepted tests, and several other parameters. Computer programs can readily perform such statistical compilations and generate distribution graphs as shown on figures 30 and 31.

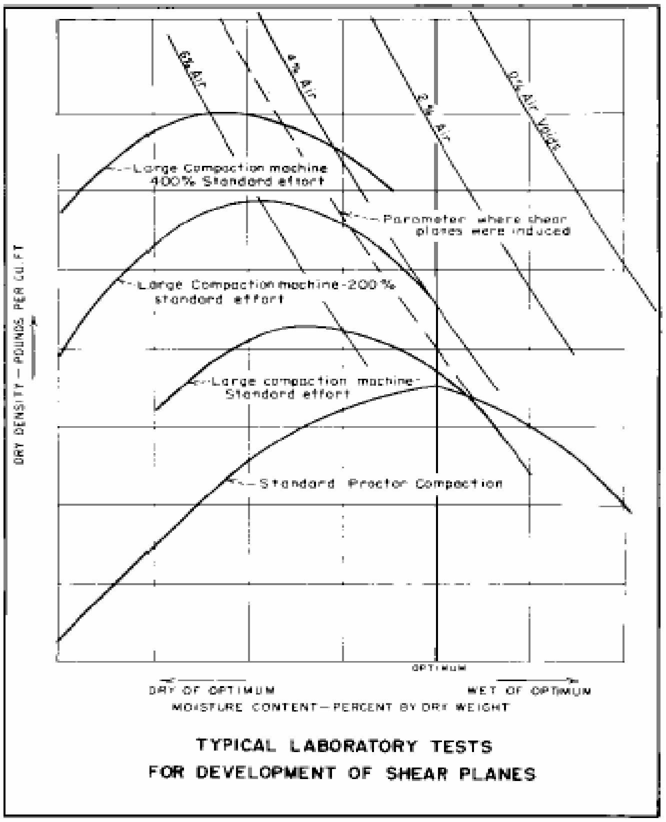
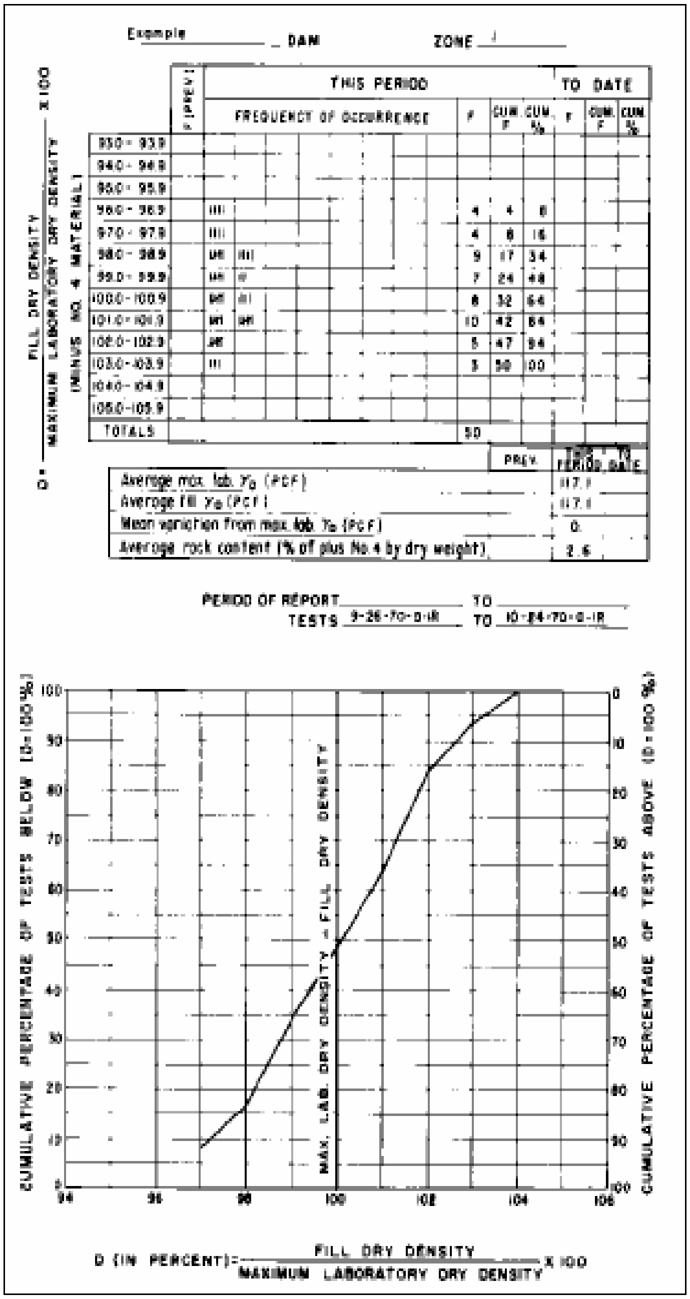


Figure 29, Typical laboratory tests for the development of shear planes.

Various other criteria for quality control have been proposed. Table 2 lists suggested limits of density and moisture control. In the absence of instructions to the contrary, criteria given in this table may be used.



Example _ DAM ZONE 1

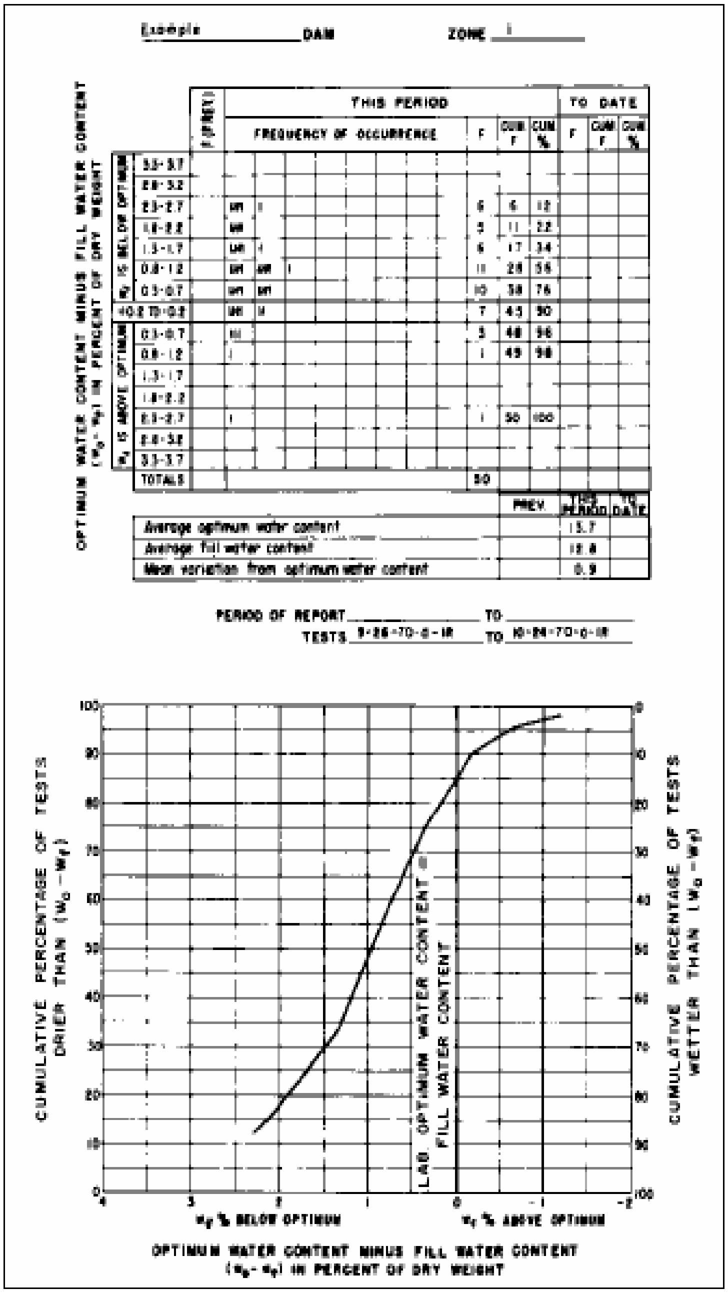
$D = \frac{\text{FILL DRY DENSITY}}{\text{MAXIMUM LABORATORY DRY DENSITY}} \times 100$ (MINUS NO. 4 MATERIAL)

	F (PREV.)	THIS PERIOD: FREQUENCY OF OCCURRENCE	F	CUM. F	CUM. %	TO DATE: F	CUM. F	CUM. %
93.0 - 93.9								
94.0 - 94.9								
95.0 - 95.9								
96.0 - 96.9			4	4	8			
97.0 - 97.9			4	8	16			
98.0 - 98.9			9	17	34			
99.0 - 99.9			7	24	48			
100.0 - 100.9			8	32	64			
101.0 - 101.9			10	42	84			
102.0 - 102.9			5	47	94			
103.0 - 103.9			3	50	100			
104.0 - 104.9								
105.0 - 105.9								
TOTALS			50					

	PREV.	THIS PERIOD	TO DATE
Average max. lab. γ_o (PCF)		117.1	
Average fill γ_o (PCF)		117.1	
Mean variation from max. lab. γ_o (PCF)		0	
Average rock content (% of plus No. 4 by dry weight)		2.6	

PERIOD OF REPORT ______ TO ______
TESTS 9-26-70-D-IR TO 10-24-70-D-IR

Figure 30, Statistical analysis of test results for density control of fill.



Example DAM ZONE 1

OPTIMUM WATER CONTENT MINUS FILL WATER CONTENT ($w_o - w_f$) IN PERCENT OF DRY WEIGHT

		F (PREV)	THIS PERIOD: FREQUENCY OF OCCURRENCE	F	CUM F	CUM %	TO DATE: F	CUM F	CUM %
w_f IS BELOW OPTIMUM	3.3-3.7								
	2.8-3.2								
	2.3-2.7		卌 I	6	6	12			
	1.8-2.2		卌	5	11	22			
	1.3-1.7		卌 I	6	17	34			
	0.8-1.2		卌 卌 I	11	28	56			
	0.3-0.7		卌 卌	10	38	76			
+0.2 TO -0.2			卌 II	7	45	90			
w_f IS ABOVE OPTIMUM	0.3-0.7		III	3	48	96			
	0.8-1.2		I	1	49	98			
	1.3-1.7								
	1.8-2.2								
	2.3-2.7		I	1	50	100			
	2.8-3.2								
	3.3-3.7								
	TOTALS			50					

	PREV.	THIS PERIOD	TO DATE
Average optimum water content		15.7	
Average fill water content		12.8	
Mean variation from optimum water content		0.9	

PERIOD OF REPORT ________ TO ________

TESTS 9-26-70-d-18 TO 10-24-70-d-18

Figure 31, Statistical analysis of test results for moisture control of fill.

Table 2.—Criteria for control of compacted dam embankments

		Percentages based on minus 4.75 mm (-No.4) fraction					
		15 m (50 ft) or less in height			15 m (50 ft) or greater in height		
Type of material	Percentage of plus 4.75 mm (+No.4) fraction by dry mass of total material	Minimum acceptable density	Desired average density	Moisture limits, w_o-w_f	Minimum acceptable density	Desired average density	Moisture limits, w_o-w_f
Cohesive soil: Soils controlled by the laboratory compaction test	0 to 25 percent	D = 95	D = 98	-2 to +2	D = 98	D = 100	2 to 0
	26 to 50 percent	D = 92.5	D = 95	-2 to +2	D = 95	D = 98	Note[2]
	More than 50 percent[1]	D = 90	D = 93	-2 to +2	D = 93	D = 95	
Cohesionless soil: Soils controlled by the relative density test	Fine sands with 0 to 25%	D_d = 75	D_d = 90	Soils should be very wet	D_d = 75	D_d = 90	Soils should be very wet
	Medium sands with 0 to 25%	D_d = 70	D_d = 85		D_d = 70	D_d = 85	
	Coarse sands and gravels with 0 to 100%	D_d = 65	D_d = 80		D_d = 65	D_d = 80	

The difference between optimum water content and fill water content of dry mass of soil is w_o- w_f, in percent.
D is fill dry density divided by laboratory maximum dry density, in percent.
D_d is relative density as defined in chapter I.

[1] Cohesive soils containing more than 50 percent gravel sizes should be tested for permeability of the total material if used as a water barrier.
[2] For high embankment dams, special instructions on placement moisture limits will ordinarily be prepared.

E. Canals

29. Design Features. Canals are constructed channels of widely varying capacities used to convey water for irrigation, power, domestic uses, or drainage. Distributory channels conveying water from the main canal to farm units are called laterals and sublaterals. The design capacity of an irrigation canal or lateral is based on maximum daily consumptive use rate downstream of the section under consideration, including allowance for seepage losses. Water loss from seepage is a major problem in canal design. On a typical irrigation project, about 20 to 40 percent of water diverted into the canals and laterals can be expected to be lost by seepage unless lining is provided. Conversely, water seeping into a canal can (under favorable conditions) be used to augment the water supply. Also, inflow seepage is the means by which subsurface drainage is accomplished for open-ditch drainage systems.

Canal construction involves a variety of earthwork problems. Because of the great longitudinal extent of the work, many different kinds of earth foundation and material conditions are encountered, all of which must be treated differently. Thus, some reaches of a canal may require only uncompacted embankments made of unselected materials excavated from the prism, while other sections may require careful control of cuts in borrow areas, strict moisture control, and compaction of earth in the form of linings or entire embankments. The following paragraphs summarize major design features related to earthwork in canal construction.

Canals are classified as unlined or lined, depending on the extent to which special provisions are made to prevent seepage. An unlined canal or lateral is an open channel excavated and shaped to the required cross section in natural earth without special treatment of the subgrade. Construction of compacted embankments at locations where the canal water surface is above natural ground is considered a normal seepage control measure in unlined canal construction.

The cross section of a canal is selected to satisfy the most desirable criteria for bottom width, water depth, side slopes, freeboard, and bank dimensions, and to facilitate operation and maintenance. The ratio of bottom width to depth ranges from 2:1 for the smallest laterals to 8:1 for large canals; side slopes on unlined canals range from 1. 5h: 1v to 2h: 1v, or flatter where unstable soils are encountered. Design of canals is based on development of a steady-state seepage condition from the normal high water surface, followed by rapid drawdown of the water surface. Dimensions of the banks and the soils of which they are composed must ensure stability under these extreme hydraulic conditions.

The water section of a canal may be entirely in cut, entirely in fill, or partially in each, depending on economics of the route, necessary gradient, structure design, and requirements of safety and water distribution. If the water section is partially or entirely in fill, compacted embankments may be used to prevent excessive amounts of seepage.

Requirements for compacting an embankment under compacted earth lining depend upon (1) height of fill, (2) earth construction materials used, and (3) method of fill construction. These requirements are reflected in the specifications. Operation and maintenance roads are usually provided on the lower bank of large canals. In some cases a maintenance berm may be needed on the upper bank. In deep cuts, berms are often used to reduce bank loads, to prevent sloughing of earth into the canal, and to lower the elevation of the operating road for easier maintenance. Berms also provided between deep cuts and adjacent waste banks. Even where operating roads are not planned, canal and lateral banks are smoothed by blading to allow power mowers and other power equipment to control vegetation.

A lined canal or lateral is one in which the wetted perimeter of the section is lined or specially treated. Materials used to line canals include:

- Unreinforced or reinforced concrete
- Pneumatically placed mortar
- Buried geomembranes
- Compacted earth
- Earth treated with additives (stabilized soils)

Various types of asphalt, brick, stone, and other materials have been used in the past.
The great need to reduce water losses in canals means a continuing search for effective low-cost linings. In addition to water savings achieved by lining canals, other advantages include:

- Prevention of water logging adjacent land
- Lower operation and maintenance costs
- Less danger of canal failures
- Reduced storage with less diversion requirements In the case of hard-surface linings, further advantages include:
- Possible smaller canal sections and structures
- Less right-of-way
- Higher permissible velocities
- Steeper gradients

Side slopes on earth-lined canals are 2h: 1v or flatter, and on hard-surface linings usually 1. 5h: 1v.
Lined canals can be built in either cut or fill or partially in each; the economic advantages of balanced cut and fill must be weighed against special treatment of fills to support the lining properly in a canal. The lining above the water surface varies with height and depend upon:

- Flow capacity
- Water velocity
- Alignment curvature
- Inflow of storm water

In addition, height increases with water surface elevation caused by:

- Canal checks
- Checks in pumping plant forebays
- Surge waves
- Wind action

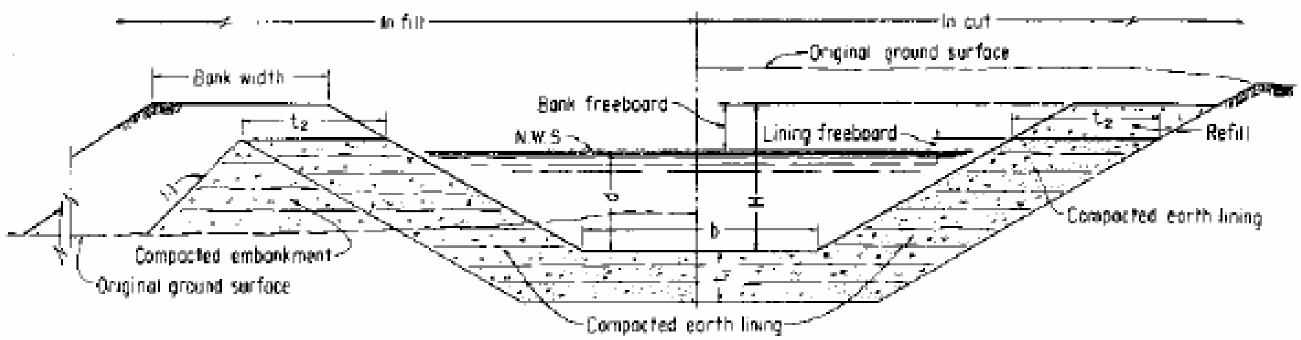
Various types of earth linings have been used to reduce water loss. The simplest method is silting (sediment sealing), in which fine-grained soils are dispersed into the water by hydraulic or mechanical means and are deposited over the wetted perimeter of the canal. Although it is effective in reducing seepage, silting is not considered a permanent seal because the silt layer is easily destroyed. Sometimes, thin, loose earth blankets of fine-grained soils have been used as linings. Such linings are inexpensive and are temporarily quite effective, but they erode easily unless protected by a cover layer of gravel. Bentonite applied as a buried membrane, or mixed with permeable-type soils, and buried geosynthetic or asphalt membranes have been used; but, such linings must be protected by gravel or soil cover. When bentonite is used for the membrane, a high-swell bentonite (sodium montmorillonite) is required. Acceptance criteria for this material are contained in the specifications.

Compacted earth linings 6” or 12” thick usually satisfy design requirements from a seepage standpoint; but, they require gravel cover or protection against erosion and are relatively expensive because of difficulty in compacting them on side slopes.

Of the earth-type linings, heavy or thick compacted earth linings have proved most desirable because:

- Conventional construction equipment and methods can be used.
- Wider variety of soils may be used without erosion protection.
- Lining is not easily destroyed by cleaning operations.
- Excellent seepage control can be achieved.

Heavy compacted earth lining is placed in the canal bottom in 6” compacted layers to a depth of 1’ to 2’and on the canal slopes to a horizontal width of 3’ to 8’. In areas of potentially serious frost action, a minimum lining thickness of 600 mm is recommended. Sheepsfoot rollers are normally used for compaction. When a thick compacted lining is placed on a 2h: 1v slope about 2 ½’of fully compacted soil, normal to the 2:1 slope, is obtainable for an 8’horizontal width. In some instances, layers of compacted fill on the side slopes are sloped downward toward the canal to provide additional width needed for wider rollers. The slope of the layers cannot be so steep as to cause movement or segregation of the earth lining materials during placing or compacting. Figure 32 shows a typical canal section with compacted earth lining. Figure 33 shows a thick compacted earth lining under construction, and the completed canal is shown on figure 34.



TYPICAL EARTH LINED SECTION

TABLE FOR EARTH LINING

d	t_1	t_2
2.0' or less	1.0'	3.0'
>2.0' to 4.0'	1.5'	4.0'
>4.0' to 6.0'	2.0'	6.0'
Over 6.0'	2.0'	8.0'

NOTE

If lining material requires a protective cover of gravel or riprap to prevent scour or erosion, excavation shall be extended to provide for the designated thickness of lining plus the gravel or riprap cover.

Figure 32, Typical canal section and compacted earth lining.

A wide range of impervious soils has been used successfully for compacted earth linings. The most desirable soils combine the most favorable properties of gradation, plasticity, and impermeability.

- Best results are obtained from a WELL-GRADED GRAVEL WITH SAND AND CLAY (GW-GC). Good stability and erosion resistance characteristics are provided by the well-graded, sandy-gravel fraction as well as by the cohesive binder. The fines content should be high enough to reduce seepage (12-20 percent).
- CLAYEY GRAVELS (GC) are next in quality.
- WELL-GRADED SAND WITH CLAY (SW-SC) follows the above.
- CLAYEY SANDS (SC) is next in quality.
- SILTY GRAVELS (GM) have been used with success.

The best materials contain just enough fines to make the soil impervious and have sufficient gravel for erosion resistance. Of the fine-grained soils, a LEAN CLAY (CL) is desirable due to its imperability and workability.

Figure 33, Placing and compacting a thick earth lining.

Figure 34, A Completed thick earth lining.

A good cohesive binder usually means the soil must have significant plasticity as measured by the plasticity index (PI). However, some low PI (7 to 12 percent) earth linings have performed reasonably well. Additional side slope protection (gravel or riprap) is always required on the outside bank of curves and downstream of in-line structures and sometimes on straight reaches.
The gravel cover layer sometimes used for erosion protection of earth linings may vary from 6” to 12”thick. Pit-run sand and gravel up to 3”in size and containing a small amount of fines has been found satisfactory. In some instances, placing a 6”thick sand and gravel filter layer on an open foundation structure has been necessary to ensure that loss of earth lining material would not occur. A WELL-GRADED GRAVEL WITH SAND (GW) with a maximum size of 3”is used.

Usually, riprap or coarse gravel is required to protect wetted earth surfaces in the vicinity of structures where high water turbulence may occur. The material will vary in size and thickness, depending upon degree of protection required, but should be graded from the smallest to the largest particle size.

The type of material considered suitable for covering buried membranes depends on grade and velocity in the canal and on steepness of side slopes. Cover materials must be stable soils when saturated. Side slopes are normally 2h:1v or flatter. In some cases, earth cover (sandy, silty, or clayey soils) is adequate; in other cases, where erosive forces are high, gravelly soil or gravel cover is required. Sometimes, earth cover is placed directly on the membrane, and a gravel cover is placed over the earth. The total thickness of cover is usually from 6” to 12”. The canal subgrade must be relatively smooth, without abrupt breaks or protruding materials, and open cobble structure must be filled with a suitable finer filter material. Cover materials must be carefully placed so the membrane is not punctured or does not slip. Shear strength in the form of friction between the different layers of material must be great enough to prevent sliding of one material on the other. Laboratory direct shear tests can be used to determine the interface friction between different membrane and earth cover materials. Installation of a buried geosynthetic membrane lining is shown on figures 35 and 36.

Figure 35, Geomembrane being placed over prepared subgrade.

Figure 36, Placement of a protective earth cover over a geomembrane.

30. Specifications Provisions.

a. General. The following paragraphs summarize various standard specifications provisions used for earthwork in canal construction. Their purpose is to clarify and to relate the different types of work. However, this discussion should not be considered a substitute for the published specifications for any particular job. Each set of specifications includes only those standard provisions that apply to the local situation; in addition, modifications are made to fit each job.

b. Subgrades and Foundations for Embankments and Compacted Earth Lining. Provisions for canal excavation require the contractor excavate to sections shown on the drawings. If, because of undesirable bottom or slope conditions, changes can be ordered for removal of unsuitable material and for refilling as necessary.
Provisions for treatment of embankment foundations cover required items of work like stripping, scarifying, and compacting the foundation surface. Stripping includes removing all material unsuitable for embankment foundation. Necessary clearing of right-of-way, grubbing of stumps and other vegetable matter and their roots, and burning or other disposal of the material are required of the contractor.

As required, excavation for canals will allow slopes of excavations and embankments for canals to be varied during construction. Access ramps must not be cut into canal slopes below the proposed water level in unlined canals or below the top of the lining in lined canals. Above the canal section, rock excavation need not be finished and will be allowed to stand at its steepest safe angle. In unlined sections of canal excavated in rock, sharp points of rock extending not more than 150 mm (6 in) into the water prism are sometimes permitted. Earth slopes are to be finished neatly to lines and grades, either by cutting or by constructing earth embankments.

Surfaces under all canal embankments, except for rock surfaces, are required to be scored with a plow to make furrows not more than 3' apart and not less than 8"deep. For compacted embankments, the entire foundation surface is scarified or disked to a depth of not less than 6"in lieu of scoring. Foundations for embankments are stripped of unsuitable material.

c. Earth Embankments and Linings. In specifications for construction of earth embankments and linings, general provisions are made to:

- Adhere to the dimensions shown on the drawings
- Provide allowance for settlement
- Grade the tops of canal banks
- Distribute gravel, cobbles, and boulders uniformly throughout the fill

Prohibit use of frozen materials and frozen working surfaces for canal embankments and built-up canal bottoms
Provisions are made for limiting the amount of backfill or compacted backfill that can be placed about structures that are built prior to excavation of the canal and construction of canal embankment. Using all suitable materials from excavations for construction of fills is required. Excess material from excavation is used to strengthen canal embankments or is wasted. Material, from cut not needed to strengthen adjacent embankment and excavated material in excess of requirements and unsuitable material, is deposited in waste banks on Government right-of-way. Stockpiling of suitable materials for use in earth linings or earth cover for geomembrane linings may be required.

Where canal excavation at any section does not furnish sufficient material for required fills, the contractor is required to borrow materials from areas designated by the Government. Adequate berms are left between embankment toes and adjacent borrow pits with 1-1/2h: 1v slopes in the pits unless otherwise specified. Borrow pits may require drainage by open ditches. Specifications for earth canal embankments and linings require that construction operations result in an acceptable and uniform gradation of materials to provide for low permeability and high stability when compacted, and that all rock larger than maximum size specified (usually 3" or 5") be removed before compaction. These requirements are particularly important in lining construction because permeability is controlled by a relatively thin layer of compacted soil. When blending fine-grained and coarse-grained soils (from different sources) is required to secure the lining material desired, specifications usually require that the fine soil be placed first, followed by the coarse soil, after which the two are blended. This procedure is used so that coarse lenses will not be left at the bottom of any finished layer. Another requirement is that blending be accomplished by a mixing machine built for the purpose of blending soils or by equipment capable to produce comparable results and specifically approved for the work. Weight of mixing equipment is important because light-weight equipment rides upward and is incapable of accomplishing proper blending.

An item covering excavation and refill of inspection trenches is often provided in the specifications. This provision allows proper inspection of materials, and blending and compaction operations can be made from time to time, as required, to ensure an acceptable lining product.

Sometimes, provisions are made for compacting clayey and silty subgrade soils of low permeability for the base of a compacted earth lining in the canal bottom. This base is considered equivalent to about one 6”compacted layer in the canal bottom.

Construction of fills for canals in the forms of earth embankments, linings, backfill, and refill is covered by a number of specifications paragraphs. Table 3 summarizes pertinent provisions and shows relationships among the different types of fills.

d. Riprap, Protective Blankets, Gravel Fills, and Gravel Subbase. Riprap or a coarse gravel cover may be required:

- On canal side slopes
- At in-line structures
- On the outside banks of curves
- In places where wind-generated waves may cause erosion
- In areas where water velocity is otherwise high

Riprap should be hard, durable, sound rock that is predominantly angular and equidimensional. If a source of unknown quality is being considered, representative samples of the material should be sent to the Denver laboratories for testing. A guide for evaluating potential riprap sources is presented in ASTM D 4992. Properly graded riprap will provide long-term, stable protection. Gradation of rock particles should be well distributed between the maximum and minimum sizes permitted so placement will result in a firm mass with minimum void spaces. (See sec. 24.)

A bedding layer of sand and gravel is normally required between riprap and the embankment or lining. If the bedding is designed as a filter, it will provide resistance to erosion from runoff or wave action while serving as a stable base for the riprap.

A geotextile may be used as a filter, but a layer of sand and gravel is still required between the riprap and the geotextile to protect the geotextile.

For some canals, a coarse granular cover can be used as a beach belt or as a complete cover for an earth lined canal. This gravel protection not only protects the banks against erosion and wave action, but also reduces maintenance for weed control and improves the appearance of the canal. Coarse gravel cover for a canal lining should:

- Have a maximum particle size between 3” and 6”
- Have between 5 and 50 percent of particles passing a No. 4 sieve,
- Contain 10 percent or less fines passing a No. 200 sieve.

Gravel fills or gravel subbases can be pit-run gravel obtained by the contractor from approved borrow sources on Government right-of-way or from sources provided by the contractor. The material should be less than 3”in size, reasonably well-graded, and contain a minimum amount of fines. Sometimes, gradation limits are specified so there is a reasonably good distribution of particle sizes from the coarsest to the finest without a major deficiency of any size or group of sizes. Uncompacted gravel fill is placed in uniform layers to the required thickness. If compaction is specified, it is accomplished according to the provision for compacting cohesionless free-draining materials.

Table 3, Materials and placing requirements for various fills used in canals

Type of construction and where used	Material	Placement requirements
Embankment not to be compacted. For lined and unlined canals.	Not specified	Approximate horizontal layers. If built by excavating and hauling equipment, travel is routed over the embankment. If built by excavating machinery, thickness of layer is limited to depth of material as deposited by excavating machine. Water may be required, either added at site of excavation or by sprinkling of banks. Working may be required to secure uniformity of moisture and materials.
Embankment to be compacted. For lined or unlined canals.	Suitable material from required excavation or borrow; remove plus 125-mm (+5-in) material; gradation must be acceptable.	Prepare foundation; if clayey or silty soils, 150 mm (6 in) compacted thickness at uniform optimum moisture; tamping rollers, 95-percent compaction. If cohesionless and free-draining cohesionless and free-draining soils, horizontal layers 150 mm (6 in) compacted thickness if rollers or tampers are used; 300 mm (12 in) compacted thickness if tractor treads or other heavy vibrating equipment is used; thorough wetting; 70-percent relative density.
Loose earth lining. For lined canals.	Impervious soil from canal excavation or borrow; uniform mixture required; depth of cut in borrow areas to be designated; uniform cutting of the face of the excavation; no plus 75-mm (+3-in) rock; may require blending.	Not specified.
Compacted earth lining. For lined canals.	Impervious soils from canal excavation or borrow; uniform mixture required; depths of cut in borrow areas to be designated; uniform cutting of the face of the excavation; no plus 75-mm (+3-in) rock; may require blending.	Prepare foundation; if clayey or silty soil, 150 mm (6 in) compacted thickness at uniform optimum moisture; tamping rollers, 95-percent compaction.
Refill above lining. For lined canals.	Selected from excavation or borrow.	Not specified.
Loose backfill. About structures.	Approved material from required excavation or borrow.	Amount can be limited by the contracting officer; method of placement not specified, but subject to approval.
Compacted backfill. About structures.	Selected clayey and silty material; from required excavation or borrow; no plus 75-mm (+3-in) rock.	Horizontal layers; compacted thickness at uniform optimum moisture shall not exceed 150 mm (6 in) or two-thirds the length of roller tamping feet, whichever is the lesser; compacted thickness shall not exceed 100 mm (4 in) where compaction is performed by hand or power tampers; 95-percent compaction.
	Selected cohesionless free-draining material; from required excavation or borrow; no plus 75-mm (+3-in) rock.	Horizontal layers 150 mm (6 in) compacted thickness if rollers or tampers are used; 300 mm compacted thickness if tractor treads or surface vibrators are used; and not more than the penetrating depth of the vibrator if internal vibrators are used; wetting by hoses, flooding, or jetting is required; 70-percent relative density.

Provisions for one-course gravel surfacing (of prescribed thickness), for operating roads and parking areas, require material consisting of sand and gravel from which all rock having a maximum dimension of more than 1-1/2”have been removed. Material from gravel sources is selected to include a sufficient quantity of natural cementitious or binding material such that the surfacing will bond readily under the action of traffic. The desirable gradation conforms to ASTM D 1241 for Type I, Gradation C, surface-course material that satisfies the requirements of note 4 in D 1241. This gradation produces a good distribution of all particle sizes with 8 to 15 percent fines to provide imperviousness and binding material. These fines are recommended to be in a limited range of plasticity such that the PI is from 4 to 9, and the liquid limit, LL, is less than 35. Surfaces to receive the gravel surfacing should be bladed or dragged to secure a uniform subgrade. Surfacing material is placed and spread uniformly on the prepared subgrade to required depth and dimensions and moistened if required. Geosynthetic fabrics may be used between the subgrade and surfacing material to improve performance of the surfacing material. Usually, minimum compaction is required such as routing hauling and construction equipment over the entire width of surfacing to distribute the compacting effect of equipment to the best advantage. However, where imperviousness and wearing surface are important, the higher degree of compaction usually specified for roller compaction is required.

31. Control Techniques.

a. General.—Recognition of the importance of controlling earth placement in embankments and canal linings has increased concurrently with knowledge of factors affecting stability of such structures. Not only must the inspector be thoroughly familiar with provisions of the specifications, but one must have a good understanding of design and construction principles involved and of field and laboratory tests used for control. The inspector should:

- Obtain all available information on design assumptions
- Obtain letters of instruction, laboratory reports, other reports, and specifications
- Check whether field conditions revealed are compatible with assumptions made on the basis of preconstruction investigations

The principles are equally applicable to foundations, whether under earth or concrete structures.

b. Subgrades and Embankment Foundations. The subgrade and embankment foundation for a canal may consist of rock or soil. Canal sections excavated in rock should be inspected to determine whether open structure or fissures exist that will cause excessive seepage or piping which would require canal to be lined. Rock foundation surfaces on which compacted earth is to be placed should be moistened before placing the first layer of earth, but standing water should not be allowed.

When canal sections are excavated in soils, or embankments are constructed on soil foundations, greater care is required for inspection than for rock subgrades and foundations because of the nature of the material. In addition to noting possible seepage or piping conditions, the inspector must identify and locate conditions that could lead to large settlement and slope instability. During investigations for design of a canal, it is not economically possible to predetermine all conditions which may be encountered. Therefore, during excavation or stripping operations, the inspector should call to the attention of supervisors the necessity for, additional exploratory holes, field penetration tests, permeability or density tests, and other sampling and tests to determine the extent of any materials of doubtful suitability for the foundation of embankments, for cut slopes, or for canal subgrades.

Some canal specifications require that reaches of canal to be lined shall be determined immediately in advance of construction by exploratory excavations, such as bulldozer or backhoe trenches dug by the contractor. In such cases, the inspector must be familiar with the permeability characteristics of soils encountered, making use of in-place permeability tests as required.

The inspector should examine canal subgrade soils on canal slopes and bottom with respect to possible future seepage and erosion. If high seepage losses are anticipated or if soils may be anticipated, to erode badly under proposed operating conditions, the inspector should call these facts to the attention of supervisors. If, upon further consideration, these conditions appear to be critical, advice from the designers should be sought. Lined sections in areas of high ground water must be protected against uplift by using drainage systems, and these are normally specified. If such areas previously unknown are observed during construction, provisions should be made for installing pressure relief devices. Similarly, when high ground water is encountered near silty or fine sandy subgrade soils of concrete-lined sections in freezing climates, attention should be directed to this fact so frost-action preventive measures can be instituted, if necessary.

If fine-grained soils are to be placed as a sublining under concrete or membrane linings, or will be used for earth linings, the natural subgrade should be inspected for open voids as in cobbly soils or fissure formations into which the fine soil might erode. In such cases, the inspector should call the conditions to the attention of supervisors so filter blankets or geosynthetic filters can be installed.

The inspector should make certain that all organic matter and any soils that may become unstable upon saturation, such as highly organic soils, loose silts, fine sands, and expansive clays, are removed or properly treated for embankments and canal linings to the extent necessary to provide a safe, stable foundation or subgrade under operating conditions. Specifications requirements for stripping and scoring earth foundations for all canal embankments, and special treatment for preparation of foundation surfaces for compacted embankments and earth linings, are important details that require careful visual inspection during construction. Determination of depth of stripping requires experience and good judgment because of compromise between what would be desirable from a design standpoint and excessive cost. Good bond between the foundation and first layer of fill is achieved by moistening the foundation rather than using a very wet embankment layer.

Foundation control information should be included in the monthly construction progress report. If field density tests are made, the data can be reported along with the compacted earthwork control data, if properly identified. Other test data, logs of holes, and observations can be included in the narrative portion of the report. Any type form can be used as long as all pertinent information is shown. Information on current computer software programs is available and can be obtained from various sources.

c. Earth Embankments and Linings. Depending upon design requirements, earth embankments for canals and laterals may consist of impervious or pervious soils placed loose, partially compacted by equipment or well compacted by rollers, or a combination of these. Earthwork control inspection of loose embankments consists only of making sure that proper types of materials are being used, that material is uniformly placed, and that layers are of proper thickness when layer thickness is specified. Normally, the finest available material is placed on the canal prism side, and coarser materials are placed toward the outside of the embankment. When specifications require partial compaction by routing equipment, inspection requirements noted above are applicable, and the inspector should make sure that hauling and spreading equipment is routed so as to provide the most uniform coverage and best compaction. In some instances, uniform moistening of partially compacted fill material is specified. Normally, the water required is that amount which will provide a total moisture equivalent to optimum moisture content for the compactive effort being applied. Standard laboratory optimum moisture content may be used. Density tests are not taken for fill placed in this manner.

Control of compacted impermeable soils in embankment and compacted earth lining consists of:

- Inspecting materials used
- Checking amount and uniformity of soil moisture
- Maintaining thickness of layer
- Determining the percentage of standard laboratory maximum dry density of the fill

Using the available logs of explorations and careful observation of excavations for the canal, appurtenant structures and borrow areas will be useful in selecting the best soils for use in compacted embankments and earth linings. In borrow areas, the depth of cut can be regulated to obtain high quality and uniformity of soil. Deposition of each load of material on the fill is directed to control uniformity. The thickness of loose lift required to result in a 6”compacted layer must be determined and regularly checked.

Blending of two or more materials is sometimes specified to produce suitable soils for earth linings. Most commonly, fine soil from borrow is added to pervious coarse soils obtained from excavation to decrease permeability. Coarse soils from borrow may be added to soils from excavation to improve erosion resistance for lining or cover purposes. Proportions of soils to be blended are determined earlier by laboratory testing. The inspector has responsibility to ensure that materials are properly proportioned and mixed. Normally, excavation materials, to be used for one of the blended materials, are required to be completely removed from the section and to be placed in the lining layer to the depth required for proper proportioning. The correct depth of fine soil is placed first, followed by the correct depth of coarse soil; then, blending is allowed.

Generally, blending is specified to be done by a machine designed for mixing soils or by other equipment which obtains comparable results. Therefore, the inspector must ensure that equipment being used provides a uniform blend of soils for the total layer depth. This is done by frequently digging holes through the blended layer before compaction to observe uniformity of the layer after blending. When soils of different colors are blended, color of the mixture at different depths in the layer is a useful index indicating effectiveness of the mixing operation. Compaction should not be allowed to proceed until a satisfactory blend has been attained. Inspection trenches should be requested frequently at the beginning of a blended lining contract, and at lesser frequency as the job progresses, so results of the contractor’s blending and lining operations can be visually observed. Cobbles larger than specified, usually 3” or 5”, must be removed before compaction. This is especially important in all types of compacted earth linings where a relatively thin section is relied upon for seepage control. Pockets of cobbles provide certain access for water loss and piping. Often, cobbles are buried in the layer being placed and are difficult to detect or to remove. Therefore, if the percentage of oversize material is large, some means should be provided for removing this material at the point of excavation.

The inspector is responsible for controlling water content of the soil to that which is "optimum for compaction" and to ensure that moisture is uniform throughout the layer to be compacted. If the soil is several percentage points dry of desired moisture content, it is more efficient to add most of the water at the location of excavation with only supplemental sprinkling after the layer has been spread. After sprinkling, mixing is required to produce a uniform moisture condition throughout the layer before compaction can proceed. Moisture uniformity should be evaluated frequently by digging holes in the loose layer just before compaction. Unless otherwise specified, optimum moisture requirements must be enforced for canal embankments and linings even though required density can be obtained at other moisture conditions. Adverse settlement and permeability properties may result if placement moisture is too low; and adverse shear strength properties may result if placement moisture is too high. Therefore, the inspector must be prepared to request application or removal of moisture, as required.

Control of pervious material includes visual inspection of material for free-draining characteristics and uniformity of material. Thickness of layer is controlled, depending on type of compaction. A thickness of:

- Not more than 6”after compaction is specified if smooth or pneumatic-tire rollers are used
- Not more than 12”after compaction is specified if vibrating rollers or tractor treads are used
- Length of vibrator if internal vibrators are used

Each layer must be thoroughly wetted during the compaction operation for all types of compaction.

Adequacy of compaction and moisture control of impervious or pervious soils is controlled by field density test Unless otherwise specified, minimum acceptable density is 95 percent laboratory maximum density for the minus -No. 4 fraction of clayey and silty soils and 70-percent relative density for the minus -3” fraction of pervious sand and gravel soils. For soils that are borderline between silty and clayey soils controlled by the compaction test and pervious sand and gravel soils controlled by the relative density test, control is based on either 95 percent laboratory maximum density or 70-percent relative density, whichever produces the highest in-place density. Table 1 is a guide for determining materials for which the relative density test is applicable.

Inasmuch as the adequacy of compaction is specified in terms of soil density achieved, the inspector is responsible to arrange or perform sufficient tests to ensure adequacy of compaction for acceptance purposes. At the beginning of any work, a considerable number of tests are required to ensure construction operations are producing required results. Then, the number of tests required is that necessary to ensure that specifications requirements are being satisfied. Because of the widespread operations of canal work, the number of field density tests required for adequate control cannot be positively stated. The following is a guide for the minimum number of field density tests:

- For all types of earthwork, one test for each work shift
- For canal embankments, one test for each 2,000 yd^3
- For compacted canal linings, one test for each 1,000 yd^3
- For compacted backfill or for refill beneath structures:
  i. Hand tamped (mechanical tamping), one test for each 200 yd^3
  ii. Roller or tractor compacted, one test for each 1000 yd^3
- One complete permeability settlement test should be made in the laboratory for each 10 density tests for compacted canal linings, impervious embankments, and impervious backfill.

When field density tests are performed in relatively narrow compacted earth linings where equipment travel is essentially along a constant route, density tests should not be taken in the tractor or wheel tracks. At such locations, density may be considerably higher than the average density of normally rolled lining.

Horizontal and vertical control must be available so the inspector can adequately locate each field density test. Usually, locations are defined in terms of station, offset from canal (or structure) centerline, and elevation above bottom grade. The inspector is responsible for locating these tests so a complete representative record of finished work is available. Companion laboratory tests by the rapid method of construction control for clayey and silty soils and relative density tests for free-draining sand and gravel soils should be made for each density test. A laboratory compaction test or a relative density test is required for each control field density test so the percent compaction or relative density, respectively, can be computed.

F. Pipelines

32. Design Features. Buried pipelines are used for pumping plant discharge lines, irrigation laterals, and distribution systems for domestic, municipal, and industrial deliveries. These systems may be constructed using one or more types of pipe such as:

- Reinforced concrete pipe
- Reinforced concrete cylinder pipe
- Steel pipe with a number of combinations of coatings and linings

- Pretensioned concrete cylinder pipe
- Ductile iron pipe
- Fiberglass pipe
- Polyvinyl chloride (PVC) pipe
- High-density polyethylene (HDPE) corrugated pipe

Usually, the type of pipe is selected by the contractor from several acceptable types described in most specifications. Buried pipe is a structure that incorporates both the properties of the pipe and the properties of the soil surrounding the pipe. Structural design of a pipeline is based on certain soil conditions; construction control is important to ensure these conditions are satisfied. Two basic types of pipe are used rigid and flexible. Pipe of 10”nominal inside diameter and smaller can be considered either rigid or flexible, but the engineer should be designing and sizing pipe to be relatively independent of soil conditions. Installation requirements for pipe are different for each condition: rigid, flexible, and 10”diameter and smaller. Rigid pipe must be supported on the bottom portion of the pipe. Flexible pipe must be supported both on the bottom and on the sides of the pipe.

Rigid pipe includes:

- Reinforced concrete pressure pipe
- Ductile iron pipe 20”and less in diameter
- Reinforced concrete cylinder pipe

Rigid pipe is designed to transmit the backfill load on the pipe through the pipe walls to the foundation beneath the pipe. The pipe walls must be strong enough to carry this load. A concentrated load at the top *and* bottom of the pipe is the worst possible loading case. If load can be distributed over a larger area on the top and at the bottom of the pipe, the pipe walls do not have to be designed as strong as for a concentrated load. Normally, soil backfill load is well distributed over the top of the pipe. However, proper support must be constructed on the bottom of the pipe to distribute the load. Proper soil support on the bottom of the pipe is necessary to maintain the grade of the pipe and to prevent unequal settlement.

Flexible pipe includes:

- Steel pipe
- Pretensioned concrete cylinder pipe
- Ductile iron pipe over 20”in diameter
- Fiberglass pipe
- PVC pipe
- HDPE corrugated pipe

Generally, steel pipe is lined with cement-mortar, either in the factory or in place after installation. The coating can be cement-mortar or some type of flexible coating such as polyethylene tape wrap.

Flexible pipe is designed to transmit the load on the pipe to the soil at the sides of the pipe. As load on the pipe increases, the vertical diameter of the pipe decreases, and the horizontal diameter increases. The increase in horizontal diameter is resisted by the soil at the sides of the pipe. Adequate soil support on the sides of the pipe is essential for proper performance of the pipe. The side-soil support must be strong enough to carry the applied load without allowing the pipe to deflect more than the allowable amount for the specific type of pipe. Proper soil support on the bottom of the pipe is necessary to maintain the grade of the pipe and to provide uniform support.

Pipe 10”nominal inside diameter and smaller do not require any compaction of the soil placed around the pipe. While pipe less than 10”may be a type otherwise considered rigid or flexible, pipe of this size should be strong enough to withstand the backfill load without any special soil support. Figure 37 shows typical installation requirements for pipe 10” diameter and smaller. Pipe 10”diameter and smaller includes PVC, steel, and ductile iron.

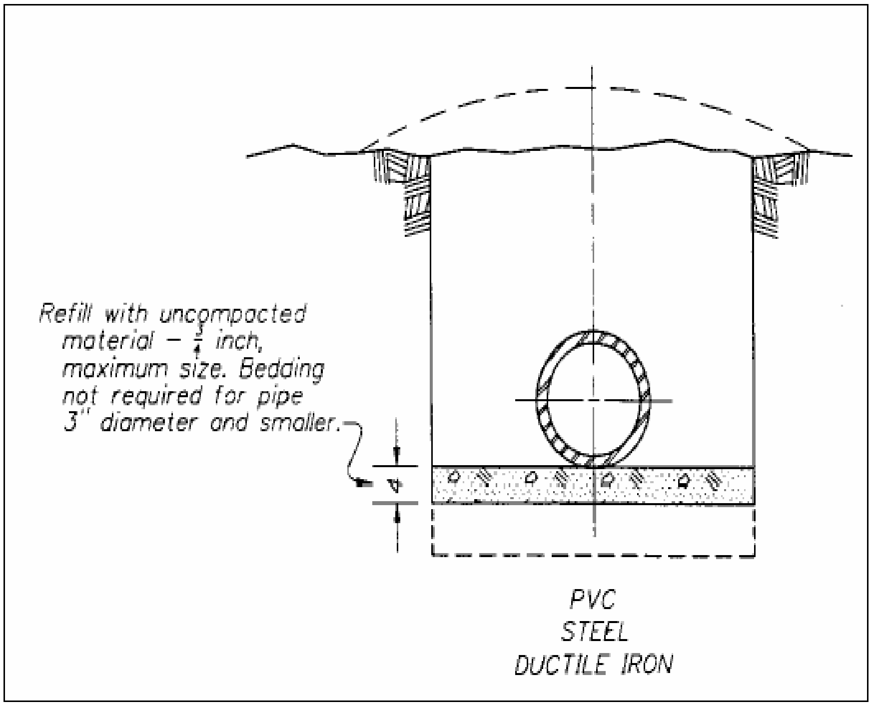


Figure 37, Installation requirements of pipe 10" diameter and smaller.

33. Specifications Provisions. Typical trench details for buried pipe are shown on figure 38. The specifications describe type of soil and compaction requirements for the foundation, bedding, embedment, and backfill.

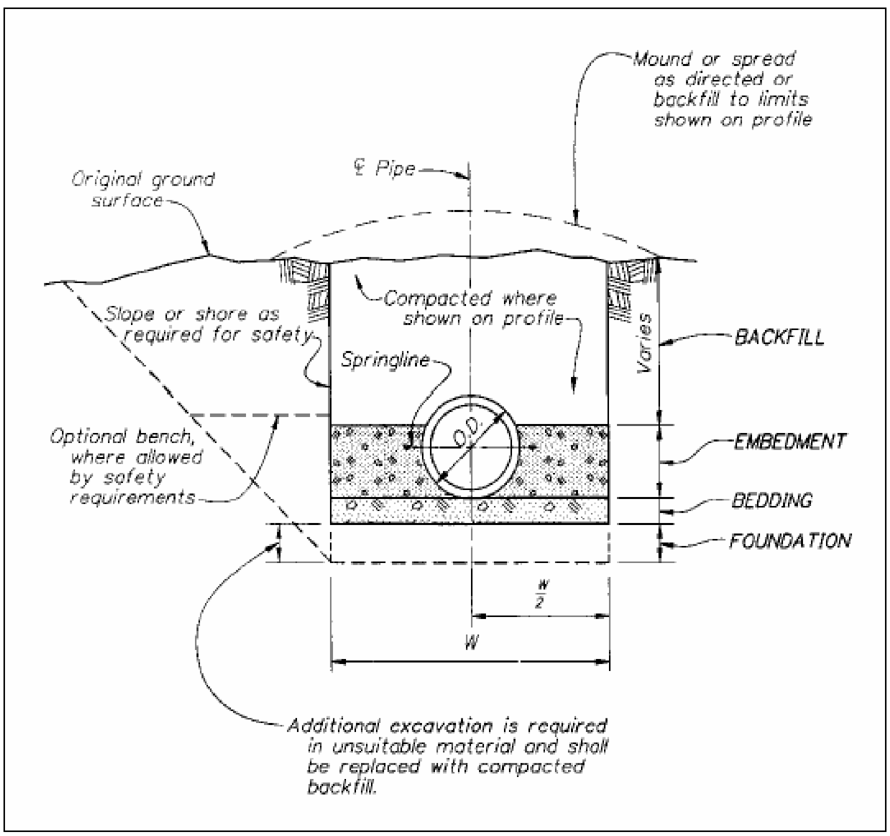


Figure 38, Typical trench details.

The foundation is the in-place material beneath the pipe. If the foundation is unsuitable, a minimum of 6”of material must be removed and replaced with compacted material. Removal and replacement of material to 3” to 6” depth or more may be necessary to attain a suitable foundation for the pipe.

Unsuitable foundations include potentially expansive material such as shale, mudstone, siltstone, claystone, or dry, dense fat clay (CH). These materials may expand when wetted. Uplift pressures created from expansion of these materials have been known to cause broken backs in rigid pipe.

Another type of unsuitable foundation includes soft, unstable soils such as very wet soils that flow into the excavation, low-density soils, peat or other organic material. A soft, unstable foundation may result in uneven settlement of the pipe. Low-density soils may collapse upon wetting. Very wet, unstable soils must be removed to provide a stable foundation that will maintain grade and provide uniform support for the pipe. Peat or other organic soils are highly compressible; significant settling of the pipe may occur if these soils are left in the foundation. Foundation materials disturbed during construction must be removed and replaced with material that is compacted.

Material is placed and compacted to replace the excavated foundation. Fat clays (CH) must not be used because moisture changes can cause significant volume increase or decrease. Elastic silts (MH), peat, or other organic material must not be used because they are highly compressible; nor should frozen soils be used. Material that allows migration of fines should not be used for a replaced foundation; for example, a crushed rock or gravel material containing significant voids should not be placed next to a fine-grained material. The fine-grained material can migrate into the voids and result in the rock particles floating in a matrix of fine-grained material. This would cause loss of support for the pipe and result in uneven settlement. Any method of compacting the replaced foundation may be used; however, specified density requirements must be satisfied.

For both rigid and flexible pipe, the bedding is a layer of uncompacted select material placed over the foundation (or the replaced foundation). The pipe is laid on the uncompacted bedding. For pipe diameters between 12” and 54”, bedding is 4”thick.

For pipe diameters 60”and over, bedding is 6”thick. The surface of the bedding shall be fine graded so that the final grade of the pipe does not exceed specified departure from grade. Settlement of the pipe into the uncompacted bedding soil must be taken into account. For bell-and-spigot jointed pipe, "bell holes" must be excavated in the bedding to provide a space between the bottom of the bell and any soil. This may require that material beneath the bedding also be excavated. This bell hole prevents a point loading condition on the bell end of the pipe.

After laying the pipe on the bedding, embedment soil is compacted underneath and beside the pipe up to the specified height. For rigid pipe, embedment soil is placed and compacted to a height of 0.37 units of the outside diameter of the pipe. Embedment soil must be a select material and must be compacted to a relative density not less than 70 percent. It is important that soil placed in the haunch area of the pipe receive sufficient compactive effort to meet the 70 percent relative density requirement. Compaction by saturation and internal vibration is an effective method of working the select material into the haunch area and of obtaining the required density. For flexible pipe, embedment soil is placed to a height of 0.7 units of the outside diameter of the pipe.

The select material used for pipe embedment must be cohesionless and free-draining (5 percent fines or less), have a maximum size of ¾”, and not more than 25 percent of the soil shall pass a No. 50 sieve.

Where the pipeline grade exceeds 0.3 units, silty or clayey material may be used instead of the specified select material for bedding and embedment. This change is allowed because of the difficulty encountered in compacting by saturation and vibration on steep slopes. If cohesive soil is used for bedding or embedment, density requirements must still be satisfied; the inspector must be sure that soil placed in the haunch areas is compacted as specified.

Most soil types may be used for backfill over the pipe, except there are maximum particle size restrictions. A limit is placed on the maximum particle size (limitation depends on type of pipe) in a zone 12”around the pipe. Outside this zone, any rock particle with any dimension greater than 18”is not allowed in the backfill. Frozen soils shall not be used. Where backfill is to be compacted to the ground surface, such as at road crossings, peat or other organic materials shall not be used. Local requirements for compacted backfill under roads must be satisfied. Backfill material must not be dropped on the pipe.

34. Construction Control. Soil placed against the pipe must be in firm, complete contact with the pipe. Compacting soil in the haunch area of a pipe is the most difficult part of pipeline construction; this compaction must be carefully monitored. A test pit should be dug at regular intervals to inspect the haunch area and to determine density of soil in the haunch area. When using saturation and vibration, the area under the bottom of the pipe must be inspected to ensure the pipe did not "float" during construction. "Bell holes" for bell-and-spigot pipe, sling holes for large diameter pipe, and spaces left for joint treatment, for other than bell-and-spigot pipe, must be filled after the pipe is laid. Embedment soil must be compacted the complete width of the trench. Embedment soil shall be placed to about the same elevation on both sides of the pipe to prevent unequal loading and displacement of the pipe. Where trenches have been left open at pipe-structure junctions, requirements (as previously stated) for pipe embedment must be continued to the structure after it is constructed but before the excavation is backfilled.

For silty or clayey soils, the thickness of each horizontal layer after compaction shall not exceed 6”. For cohesionless, free-draining material such as sands and gravels, the thickness of the horizontal layer, after compaction, shall not exceed:

- 6” if compaction is performed by tampers or rollers
- More than 6” if compaction is performed by treads of crawler-type tractors, surface vibrators, or similar equipment
- More than the penetrating depth of the vibrator if compaction is performed by internal vibrators

Backfill over the pipe should be placed to a minimum depth of 3’or one-half pipe diameter (whichever is greater) above the top of the pipe before power-operated hauling or rolling equipment is allowed over the pipe. In addition, limitations may be imposed on weight of equipment traveling over the pipeline. Equipment crossings, detours, or haul roads crossing the pipeline must be approved prior to use.

Pipe 10”diameter and smaller can withstand the weight of backfill without special soil support. Compacted soil around this pipe is used only on the outside of horizontal curves or where the pipe crosses under another conduit, and under a roadway, canal, ditch, or structure. Previous comments on adequacy of a foundation, however, still pertain to 10”pipe.
For pipe 3” to 10”in diameter, 4”of uncompacted material are required as bedding between the pipe and the foundation. Maximum particle size of the bedding material is ¾”. Peat or other organic material shall not be used as bedding. The uncompacted material can be imported or the bottom of the excavation can be loosened by scarifying if the in-place material is suitable. The main requirement for pipe 10”and smaller is suitability of the foundation.

35. Construction Control Requirements. Trench dimensions, minimum installation width, slope of the trench walls, trench depth, and side clearance of flexible pipe must always be carefully checked.

A minimum installation width, *W,* is specified to ensure a minimum distance between the pipe and the trench wall. Enough clearance must be allowed to inspect pipe joints and to compact the embedment. This clearance is particularly critical for vertical trench walls. Minimum installation width is measured at the top of the foundation. For trench slopes, the most recent Office of Safety and Health Administration (OSHA) requirements should be consulted. However, State or local regulations may override these requirements. The trench wall slope must begin at the bottom of the excavation, which includes any additional excavation of foundation material.

For flexible pipe, side clearance between the pipe at the springline and the trench wall must be checked for all conditions, including shored trenches, sloping trench walls, and vertical walls. This clearance is required in addition to the minimum installation width and may require, in some cases, a wider trench bottom than the minimum installation width.

Performance of flexible pipe depends on soil at the sides of the pipe for support. This side soil support results from the combination of embedment and trench wall soil. The width of the trench depends on the relative firmness of embedment and trench wall material. If the trench walls are firmer than the embedment, embedment soil is used to fill the space between the pipe and the trench walls. If the trench walls are soft and easily compressible, most of the side soil support must come from the embedment. As shown on figure 39, three types of trenches are specified which state the minimum clearance between the pipe and the trench wall measured at springline of the pipe.

Trench type 1 is where trench wall material is stronger or firmer than the embedment. Typical trench wall materials would be rock; materials described as:

- Claystone, mudstone, or siltstone
- Strongly cemented soils even though of low density
- Cohesionless soils with in-place relative densities over 70 percent
- Silty or clayey material with in-place densities over 95 percent compaction

For trench type 1 installation, minimum side clearance is 10”for pipe 18”and smaller, and 18”for pipe larger than 18”diameter.

Trench type 2 is where trench wall soil has a strength or firmness equivalent to the embedment. These soils include:

- Silty or clayey material with in-place densities less than 95 percent compaction, but over 85 percent compaction
- Cohesionless soils with in-place relative densities between 40 and 70 percent

Minimum side clearance for trench type 2 installation is equal to the outside diameter of the pipe.

Trench type 3 is where trench walls are much softer than the embedment. Soils in this category include:

- Peat or other organic soils
- Elastic silts (MH)
- Low-density silty or clayey material (below 85 percent compaction)
- Low-density cohesionless soils (below 40 percent relative density)

Minimum side clearance for trench type 3 is equal to two pipe outside diameters, making a total trench width of five pipe diameters measured at pipe springline. Generally, this is impractical so, in trench type 3 areas, rigid pipe may be specified. During construction, if unexpected areas of trench type 3 conditions are encountered, changes to the type of pipe or method of construction must be made to obtain a proper installation.

During investigations where types 2 or 3 trench may be required, areas along the pipeline must be identified. Particular attention should be paid to stream crossings, loessal deposits, talus slopes, and landfills.

The specifications requirements pertain to one method of pipeline construction acceptable to the design engineer. Other acceptable installation techniques may be used. Sometimes, pipe design may have to be changed to be compatible with the installation method. It is not uncommon to provide a stiffer pipe than specified, so the density requirement for embedment may be lowered.

36. Soil-Cement Slurry Pipe Bedding. When conditions warrant, the contractor may choose to install pipe using soil-cement slurry as an alternate to the standard compacted soil embedment. The trench is trimmed to a semicircular cross section (or other approved shape) slightly larger than the pipe (as shown on figure 40). A soil-cement slurry is used to fill the gap between the pipe and the in place soil. Since soil-cement transfers the load from the pipe to the trench material, the native soil must be able to provide the necessary support for the pipe. Typically, in-place materials that categorize as trench type 1 would qualify.

SIDE CLEARANCE TABLE

TRENCH TYPE	MINIMUM SIDE CLEARANCE (INCHES)
1	10 INCHES FOR 12" THRU 18" I.D. 18 INCHES FOR OVER 18" I.D.
2	ONE O.D.
3	TWO O.D.

For location of Trench Types, see Specifications.

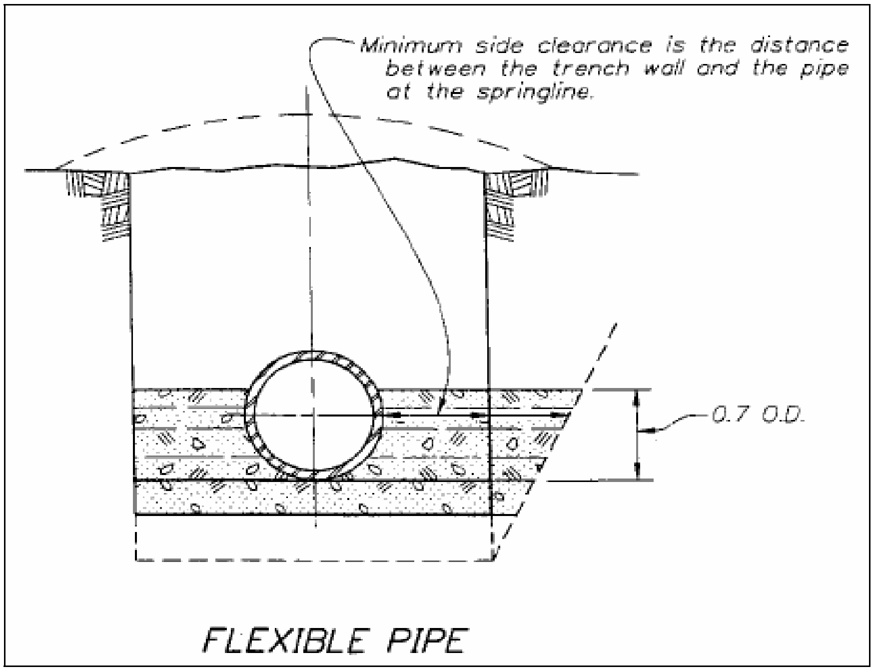


Figure 39, Side clearance requirements for flexible pipe.

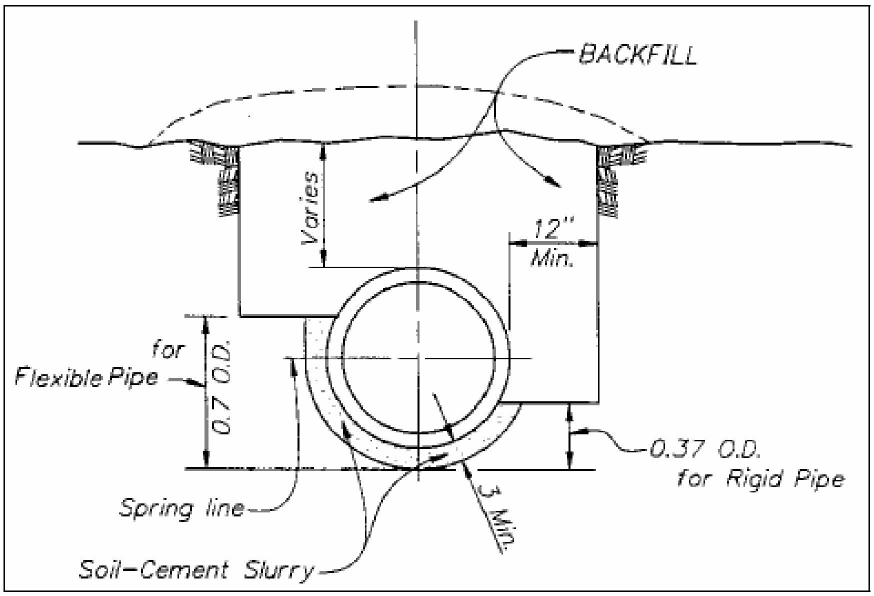


Figure 40, Soil-cement slurry installation.

The trimmed shape is advantageous because it reduces excavation quantities and handling of excavated soil. Placing soil-cement slurry is quicker than replacing and compacting soil beneath and beside the pipe. This faster installation is a distinct advantage where construction work space is limited or where time of installation is critical.
Soil-cement slurry is a combination of soil, portland cement, and enough water so the mixture has the consistency of a thick liquid. In this form, the slurry flows readily into openings and provides a hardened material with strength greater than the in place soil.

Allowable ingredients and specific placement requirements are given in the construction specifications. Typically, soil-cement slurry contains about 6 to 10 percent portland cement by dry mass of soil. Some cementitious fly ashes have been successfully used in place of cement.

One distinct advantage in using soil-cement slurry is that soil-cement may be produced using local soils. The soil can contain up to 30 percent nonplastic or slightly plastic fines. Although clean, concrete sands have been used, the presence of fines helps keep the sand-size particles in suspension. This allows the mixture to flow easier and helps prevent segregation. The allowable maximum particle size, limit of organic material, and other requirements are usually specified.

Soil-cement slurry typically has a water-cement ratio of about 2.5 to 3.5 to attain required flow properties. Generally, internal vibrators are used to ensure that the slurry completely fills the annular space around the pipe.

Central batch plants, with the slurry delivered in ready-mix trucks, or portable batch plants have been used to mix soil-cement with the latter normally used when soil comes from trench excavation. Materials to be used and method of mixing depend on availability of local materials and equipment.

Construction control of soil-cement slurry involves only two factors: (1) proper proportioning of the mixture and (2) ensuring that the slurry completely fills the annulus between pipe and trench walls. Proper mixture is checked by determining the unconfined compressive strength at 7 days for minimum and maximum strength requirements of 100 and 200 lbf/in^2. Higher strength is not desirable because structural characteristics of the pipe may be altered.

G. Miscellaneous Construction Features

37. Highways and Railroads. During construction, public utilities, communication systems, or transportation systems (including highways and railroads) may need to be relocated. Normally, a relocation agreement is entered into with the owner of the facility. Provisions of the relocation agreement will require the contractor to construct the relocation of facilities according to design and construction standards agreed upon.

a. Design Features. The design of a highway or railroad consists of selecting and laying out the roadway or roadbed alignment, designing and detailing required structures, and preparing specifications for constructing the facility. As a basis for agreement on the design and construction of relocated highways, the design engineer should use American Association of State Highway and Transportation Officials standards and manuals, the standards of the state highway departments, and general highway construction practice. For the design and construction of relocated and access railroads, the design engineer should use American Railway Engineering Association manuals, the railroad' s own standards, and general railroad construction practice.

Access roads for use by the contractor should be designed and constructed in accordance with current acceptable standards. These standards are based on local proven methods of road construction which permit using local materials and services of local contractors experienced with construction equipment designed for such work.

b. Earthwork Specifications Provisions. Specifications for constructing a railroad or highway will contain a paragraph on roadway excavation. This paragraph will require the contractor to construct the roadway to dimensions shown on drawings or as staked and to prescribed lines and grades. Drawings include profiles of gradelines either to finished grade or to subgrade. The term "subgrade" refers to the top of embankments and to the bottom of excavations ready to receive the roadway surface or the railroad ballast. In rock excavation, the bottom of the cut is excavated to 6” below subgrade; and, in common excavation, all loose rock fragments, cobbles, and boulders are removed or excavated to a depth of not less than 6”below subgrade.

Excavation below subgrade is refilled to subgrade with material obtained from excavation for the roadway or from borrow pits; the refilled material is compacted equivalent to the compaction required for embankments. Provisions are made in the paragraph on roadway excavation for dealing with slopes shattered or loosened by blasting and slides extending beyond the established slope lines or below subgrade. Provisions for constructing side drains and for excavating around trees, poles, or other objects that are to remain within the right-of-way for the roadway are also given in the paragraph on roadway excavation.

Provisions are included in paragraphs on preparation of surfaces under roadway embankments for surface treatment and for removal of unsuitable materials. Surfaces of sloping ground underneath embankments must provide bond with the embankments and to prevent slipping. Where embankments placed on smooth, firm surfaces and where low embankments are placed, the original ground surface is thoroughly plowed or stepped to ensure proper bonding of new and existing material. When embankments are placed on steep rock slopes, trenches are blasted into the rock surface for "keying" the embankment to the rock surface. The contractor must strip the area under an embankment of all unsuitable material to depths as directed.

Roadway embankments are constructed to established lines and grades and increased by such heights and widths as necessary to allow for settlement. Brush, roots, stumps, sod, or other unsuitable materials are not permitted in embankments. Hard material or hard lumps of earth more than 12”in size must be broken down, or removed, and boulders and cobbles more than 6”in diameter are not permitted in the upper 24”of embankment. Given a choice of materials, the best are used in the upper 12”of the embankment. Material is not permitted in embankments when either the material or the surface on which it is to be placed is frozen. Good distribution of cobbles and gravel within the embankment is required when such material is being used. Rockfill embankments are constructed so large spaces are not unfilled. When directed, rock fragments are deposited in the outer portions of embankments to protect the slopes against erosion.

The contractor’s combined excavation and placing operations should ensure that material in the embankment will be blended sufficiently to secure the best possible degree of compaction and stability. The required thickness of layers is 12”after compaction for earth fill and not more than 24”after compaction for rockfill. Layers are constructed across the entire width of embankments and must be built to the required slopes rather than widened with loose material dumped from the top. On sidehill fills, where the width is too narrow to accommodate hauling equipment, end dumping is used until the width of the embankment becomes great enough to permit using hauling equipment. The contractor is required to route hauling equipment over the layers already in place and to direct travel evenly over the entire width of the embankment so as to obtain the maximum amount of compaction possible.

Moistening and compacting roadway embankments may be necessary where normal construction techniques will not result in proper densification. Where sufficient moisture is not available in the material, additional water must be provided by sprinkling as the layers of material are placed on the embankments. Moistening must be uniform; harrowing, disking, or other working may be necessary to produce a uniform moisture content. Material containing excess moisture must be permitted to dry to the extent required before being compacted.

The amount of embankment rolling depends on the nature of the material compacted and the extent of compaction required; rolling will be specified for each embankment or portion of an embankment. The design engineer usually follows State highway specifications when embankments are built for a State highway department. Generally, highway department specifications are of the performance type where only the end result is specified. For this type of specification, the contractor usually selects the equipment to obtain the end result. If highway and railroad embankments are not built according to State specifications, the contractor should build them using an end-result type of specification requiring the fill to be compacted to 95 percent laboratory maximum dry density.

Additional material for construction of embankments and for excavation refills in rock cuts is obtained by widening the cuts, flattening the slopes on either or both sides of the centerline, or from borrow pits. If additional material is taken from borrow pits adjacent to the roadway, a berm of original unbroken ground, not less than 10’wide, is required between the outside toe of the embankment and the edge of the borrow pits, with provisions for a side slope of 1.5h:1v to the bottom of the borrow pit. Where directed for drainage or other requirements, the berm between the toe of the embankment and the borrow-pit slope should be no less than 40’wide. Provisions are made for connecting pit to pit with waterways, unless otherwise directed, and to provide ample drainage and leave no stagnant pools. Unless otherwise approved, the bottoms of pits near bridge and culvert openings are not permitted to be excavated below the culvert or bridge. The borrow pits should be left in a reasonably smooth and even condition.

c. Control Techniques. Many of the control techniques discussed in preceding paragraphs are applicable to highway and railroad embankments. Moisture content-density relationship of soils and related field control methods are fundamental for any earthwork construction where compaction of soil is required. However, the same degree of control may not be practical nor required for highway and railroad embankments.

Control requirements vary depending on standards adopted by the design engineer and any relocation agreement. The inspector should first become familiar with the appropriate requirements and standards and adapt the techniques discussed for embankment dam embankments to the requirement. The principal differences in control are:

1. Moisture, if required, for the most part will be added to material on the fill. Usually, moisture content should be as near as possible to the optimum required for maximum compaction.
2. Density requirements are usually specified as percentages of laboratory maximum dry density and vary according to the maximum dry density of soil.
3. The frequency of testing required to maintain control on these embankments is not standardized. For each fill location, the quantity of water and the amount of rolling required must be determined rapidly so the information may be used when

constructing the fill. Since construction of a fill may be a relatively short operation, moisture and rolling requirements must be established quickly. A rapid method of construction control can be used for compacted highway and railroad embankments. Where compaction is required by the specifications, one in-place density test should be made for each 2,000 yd^3 of Conformance with certain specifications provisions can be determined by visual inspection, such as preparation of foundation soils, and deposition of material. Selection of borrow materials required in embankment dam construction may not be required for highway or railroad work.

4. Thickness of layers, whether they are to be compacted by roller or equipment, is normally specified; control of layer thickness is the inspector' s responsibility. Where embankment is to be compacted by equipment travel, distribution of travel over the embankment should be directed to obtain the maximum amount of compaction.

38. Miscellaneous Structures.

a. General. In addition to large earth structures such as dams and embankments, many earthwork features relate to smaller structures built. These structures include:

- Pumping plants
- Powerplants
- Bridges
- Substations
- Warehouses
- Residences

Various types of canal structures such as:

- Checks
- Drops
- Turnouts
- Siphons
- Pipelines
- Overchutes.

Although foundation loadings for most of these structures are normally light, moderately heavy loadings on soils are not uncommon for larger units. Even for small structures soft, highly compressible, or expansive soils, at natural moisture content or in a future wetted state, must be examined carefully. Care must be exercised when compacting embankment and compacting backfill about these smaller structures.

b. Structure Foundations on Soil or Rock. The contractor is required to use methods of foundation soil preparation, including moistening and tamping if necessary, which will provide appropriate foundation soil conditions. Excavation is made to lines and grades shown on drawings or as directed when subsurface conditions become known. Additional excavation is refilled and compacted, as specified.

Loosened or disturbed natural material must be either compacted or removed and replaced with suitably compacted material. When excavation for concrete structures is in rock, rock should be excavated so there will be more than 150 mm (6 in) between rock points and the bottom of the concrete slab. This space is filled with compacted soil. For other structures, concrete is often placed directly on rock which has been previously cleaned and dampened. Large unit loadings may be used in this case.
Construction of small miscellaneous structures usually involves a variety of earthwork problems because of the numerous structures involved, the extent of the work, and the many different soils encountered in foundation soils and backfill materials. For larger structures (i.e., powerplants and pumping plants), where loadings are moderately high and settlements must be small, explorations are normally conducted before designs are prepared to determine competency of foundation soils. Measures to improve the competency of foundation soils, if needed, are usually known ahead of time and are included in plans and specifications. Often, it is impractical to perform adequate predesign explorations, so provisions for drilling exploratory holes or performing tests, after the excavation has been completed, are contained in the specifications. Usually, it is impractical to perform detailed investigations of foundation soils for the numerous small structures built. Regardless of the amount of prior exploration, the inspector' s responsibility is to make sure that soil conditions, which can be observed upon completion of excavation to grade, are compatible with design assumptions. If any doubt exists concerning adequacy of foundation soils, this fact should be brought to the attention of the supervisor.

Several methods can be used to check properties of foundation soils. When sands are involved, the field penetration test, with a split-tube sampler, provides a rapid method for determining the relative density of soil. The relationship of blow counts to relative density was shown on figure 1-15. Unless otherwise specified, 70 percent relative density of sands can normally be considered adequate for all but very heavy loadings. For very fine sands, 80 percent relative density is often used. Gravelly soils containing at least 50 percent gravel and graded up to coarse sizes can be considered to have adequate shear strength and satisfactory consolidation properties for most structures if the in-place relative density is 60 percent and the index unit weight tests can be used to determine the relative density of sand, sand-gravel, and gravel soils.

The relative consistency of saturated fine-grained soils can be estimated using data from the standard penetration test. Cone penetration tests and pressuremeter tests (ASTM D 4719)may be used to evaluate the adequacy of foundation soils. Both tests may be performed in cohesive or cohesionless soil deposits.

Hard, saturated fine-grained soils normally will support moderately heavy to heavy loads. Very firm soils usually are adequate for light loadings. When fine-grained cohesive soils are not saturated, data from most *in situ* test methods are misleading and should not be used for evaluating foundation soils for hydraulic structures because properties of the soils in a later saturated condition cannot be determined. In-place density tests provide a good index as to the supporting capacity of saturated or unsaturated fine-grained soils.

Fat clays (CH), lean clays with liquid limits above 40 percent, or high plasticity clay shales should be checked to determine whether they are sufficiently expansive to cause undesirable uplift of a structure. This can be accomplished by determining the gradation, plasticity index, and shrinkage limit of the soil. Any such soils falling into the "low expansion" grouping of table 2 are satisfactory.

When qualitative tests indicate doubtful support more detailed testing is necessary. Usually, this is accomplished by securing undisturbed soil samples and testing them in the laboratory.

When bottom grade of a structure is known to be below the water table, provisions are usually contained in the specifications for dewatering foundation soils and unwatering the excavation. When water is encountered, the inspector must ensure that foundation soils are not disturbed by removal of the water. This may be accomplished by beginning the dewatering operations as soon as ground water is encountered or at an elevation at least 5'above bottom grade elevation. During dewatering operations, the water level should be maintained continuously at least 12"below base grade until 48 hours after sufficient concrete has been placed to overcome uplift pressure. Dewatering may be performed by well points, deep wells, trenches, or sumps, or by other methods which will prevent upward movement of water through soils within the area of the structure foundation.

After excavation has proceeded to grade, care should be taken to ensure that foundation soils are not disturbed by equipment working in the area. Excessive drying or wetting may damage certain types of soils. Shales may require a cover of pneumatically applied mortar, soil, or a thin cover of concrete to prevent slaking; however, placement immediately after final cleanup is usually sufficient.

When a structure is designed for placement on impervious soils, the inspector should ascertain that the material is adequately impervious and that strata or lenses of permeable soil do not exist which might cause subsequent piping. Cutoff walls should be constructed to the depth shown on the drawings and into impermeable soils as required. Construction of cutoff walls into soil must be such that piping cannot occur around the walls. Normally, specifications require either that concrete for transitions and cutoff walls be placed against the sides of the excavation without use of intervening forms or that compacted backfill of silty or clayey soils be used between the concrete and natural ground after the forms are removed.

When the foundation area consists of more than one type of soil, care should be taken to prevent appreciable differential settlements. Often, pockets of soft or loose materials must be removed and be replaced with suitable compacted soil to provide uniform bearing over the entire foundation area. When a foundation area is partially on soil and partially on rock, sometimes it is necessary to remove the soil to rock and refill with lean concrete. At times, excavating additional rock and replacing it with compacted soil is desirable. The type of treatment depends upon the type of structure, structural loadings, and strength and compressibility of the soil.

c. Pile and Drilled Shaft Foundations. When piles are required to provide desired bearing capacity or to minimize settlement, specifications contain requirements as to the type of piles, the cross-sectional area, and, in some cases, the length of the pile. Frequently, minimum depth of penetration is specified. A pile must never have a minimum penetration less than that shown on the drawings. Piles are of such length that the top of the pile will be at the specified elevation even after any damaged or battered portion of the top of the pile is cut off. Piles are required to be driven accurately to required lines and depths with either drop (gravity) hammers, single acting or double acting diesel hammers, or vibratory or hydraulic drivers. To reach the minimum penetration specified on the drawings, preboring or jetting prior to driving may be required. A usual requirement is that the number of jets and the volume of water, with adequate pressure, be available to freely erode material adjacent to the pile. When the minimum penetration called for is reached, the jets shall be removed and the pile finally set to grade using normal driving techniques. When holes are prebored, the diameter of the prebore holes shall be slightly smaller than the diameter of the pile, and the depth of the holes shall be such that the piles will reach the specified minimum penetration.

The pile driving specifications contain detailed provisions for:

- Shoes
- Cutoff
- Preboring or jetting
- Cutting
- Splicing
- Pile materials
- Preservative materials
- Preservative treatment
- Miscellaneous accessories

The inspector is required to ensure that all piles are located properly and are driven to specified lines, grades, and tolerances. The inspector must ensure that piles are not damaged during driving and that required penetration is obtained. Damaged piles or piles not driven to correct line and grade are ordered removed. When piles are in place in freezing weather, the top elevation should be checked before placing of concrete. If heaving has occurred, the piles should be redriven to grade.

Sometimes, static pile load tests are specified for determining bearing capacity or tensile capacity of single piles or pile groups. When pile load tests are required and, unless otherwise specified, they are in accordance with:

- ASTM D 1143: Piles Under Static Axial Compressive Load
- ASTM D 3689: Individual Piles Under Static Compressive Load
- ASTM D 3966: Piles Under Lateral Loads

In special cases, other tests may be required; special instruction will be provided for those tests.
When pile load tests are performed, the inspector is responsible to verify that:

- Test piles are placed to specified line, grade, and length
- Driving records are secured
- Test loadings are applied in proper manner, amount, sequence, and period of time
- Data are obtained and recorded as required

Static load capacity of individual piles also may be determined using dynamic testing methods. When using dynamic test methods, transducers are mounted to the top of a pile to obtain force-time or velocity-time data from individual hammer blows. The data can be analyzed using wave equation theory to determine:

- Pile integrity
- Static load capacity (bearing capacity)
- Hammer performance
- Stresses in the pile during driving

Dynamic testing should be performed in accordance with ASTM D 4945: High-Strain Dynamic Testing of Piles.
Drilled shaft foundations (also called caisson, drilled pier, drilled caisson, or bored pile foundations) sometimes are used in lieu of pile foundations when it is necessary to transfer heavy foundation loads to deep, competent bearing strata. Drilled shaft foundations may be of constant diameter, tapered, or belled-out at the bottom. Belled-out shafts (piles) have been used in expansive clay as anchors to prevent structure uplift. Usually, shafts are drilled with large auger boring equipment, belled if needed, and cleaned; then, reinforcing steel and concrete are placed as specified. The inspector is responsible to ensure that:

- All loose soil is removed from the bottom of the shaft

- Soil at the bottom is competent for design loadings
- The bells, if required, are properly excavated in undisturbed material

Shafts may be cased or uncased, depending on soil and ground-water conditions. If water is encountered, the hole must be dewatered by approved methods until some specified time after the concrete is placed (usually 6 hours), unless tremie concrete is allowed in the specifications.

d. Backfill. Backfill is specified in specifications variously as:

- Backfill about structures may be specified simply as backfill for which compaction is not required, or as compacted backfill for which the compacted dry density is specified.
- Backfill may be further classified on the basis of material required, for example, "cohesionless free-draining materials of high permeability, commonly called pervious material," or "clayey and silty materials of low permeability, commonly called impervious material."
- Compacted backfill is normally specified if backfill under structures is required.
- Compacted backfill may be specified as compacted clayey or silty materials for which compaction is done by tamping rollers, power tampers, or other similar suitable equipment.
- Compacted backfill may be specified as compacted cohesionless free-draining materials for which compaction is accomplished by thorough wetting accompanied by operation of surface vibrator, internal vibrator, tractor, or other similar suitable equipment.

Backfill, or compacted backfill is placed as shown on the drawings or where prescribed by the contracting officer. Type of material used, amount, and manner of depositing backfill are subject to approval of the contracting officer. Insofar as possible, backfill materials are normally secured from required excavation. When suitable materials required for a specific structure are not available from required excavation, the materials are secured from approved borrow sources. Distribution of material must be uniform and such that compacted backfill is free from lenses, pockets, streaks, or other imperfections.

When silty and clayey soils are placed as compacted backfill, required dry density of soil is 95 percent of laboratory maximum dry density unless otherwise specified.[13] Prior to compaction, these soils normally are required to have a uniform moisture content. When cohesionless, free-draining soils are placed as compacted backfill, required dry density of soil is that equivalent to 70 percent relative density unless otherwise specified. Higher density requirements may be specified for fine sands for large structure loadings, when structure vibrations are severe, or when only very small settlements can be tolerated. After compaction, the depth of layers, unless otherwise specified, is 6”when rollers are used, or not more than 12”when vibrators or surface tractors are used. A normal requirement is that these fine sands be thoroughly wetted during compaction.

To provide adequate protection for compacted backfill about structures where backfill adjoins embankment, specifications often provide that the contracting officer can direct the contractor to place a sufficient amount of embankment material over the compacted backfill within a specified length of time (often 72 hours) after compaction of backfill has been completed. This provision is included so the backfill will not dry and shrink and cause cracking and pulling away from the structure.
Sand and gravel backfill material is often specified for placement:

- Under and about structures
- Under canal linings
- For filters
- At bridge approaches
- At weep holes, or against retaining walls
- For other purposes

Various gradations of material may be specified for different structural purposes. Laboratory tests should be made at frequent intervals to ensure that materials meet specified gradation limits. Unless otherwise specified, maximum particle size for backfill materials is 3”; and, the amount of material finer than the No. 200 sieve size for sand-gravel backfill is limited to 5 percent or less.
Generally, control techniques given for compacted earth embankments and compacted linings apply to backfill. Control of compacted backfill consists of:

- Inspecting materials
- Checking amount and uniformity of soil moisture
- Maintaining thickness of layers being placed
- Determining the percentage of laboratory maximum dry density or relative density obtained by compacting operations

Also, determination should confirm that the soils are fully compacted to specified lines and grades.

e. Filters. Protective filters for canals and miscellaneous structures consist of one or more layers of free-draining soil placed on less pervious soil. The soil to be protected by the filter is commonly referred to as the base soil or the base material. The purposes of a protective filter are to:

- Safely carry off seepage from the base soil
- Prevent erosion or piping of the base soil
- Prevent damage to overlying structures from uplift pressure

The base soil can be protected by filter materials that have a certain range of gradation. Gradation of the filter material bears a definite relationship to the gradation of the base material. Filters may consist of a single layer or several layers, each with a different gradation. Multiple layer filters are known as zoned filters.
Material selected for the protective filter must satisfy four main requirements:

1. Filter material should be much more pervious than base material. This requirement prevents excess hydraulic pressures from building up in either the filter material or the base material.
2. Voiding the compacted filter material must be small enough to prevent base material from penetrating the filter. Penetration of the filter can cause the filter to clog with base material or piping of the base soil through the filter. Either condition can result in failure of the filter system.
3. The filter must be thick enough to provide good distribution of particle sizes throughout the filter to provide adequate hydraulic capacity for the volume of water flowing out of the base soil.
4. Filter material must be prevented from moving into drainage pipes by providing sufficiently small slot openings or perforations, or by additional coarser filter zones if necessary.

Filter design based on gradation criteria is generally credited to G.E. Bertram, with the assistance of Karl Terzaghi and Arthur Casagrande. Over the years, studies by the U.S. Army Corps of Engineers, and the Natural Resources Conservation Service have produced various, and sometimes conflicting, filter criteria. Stability criteria are the oldest criteria and are the only criteria having successfully been demonstrated in laboratory studies; they are the only criteria that seem to be universally accepted. Stability criteria (also referred to as the stability ratio) state that the ratio of the D_{15} size of the filter soil to the D_{85} of the base soil must be equal to or less than 5. The stability ratio has been shown to be conservative but effective in preventing penetration of the base soil into or through the filter.

Permeability of filter material should be at least 25 times larger than permeability of the base material. Generally, permeability criteria are satisfied if the D_{15} size of the filter material is equal to or greater than 5 times the D_{15} size of the base material.

1. Filter material should be clean, cohesionless sands or gravels with C_u (coefficient of uniformity) between 1.5 and 8. Filter material should pass the 1-1/2"sieve to minimize particle segregation and bridging during placement. Smaller maximum particle sizes may be specified. Filters must have less than 5 percent minus No. 200 sieve size material to prevent clogging caused from excessive movement of fines in the filter and drainage pipes and to maximize permeability of the filter.
2. Filter material adjacent to drainage pipes should have sufficient coarse sizes to prevent movement of filter material into the drainage pipes. The maximum size of perforations or joint openings of drainage pipe is selected as one-half of the D_{85} grain size of the filter material.

The specifications will normally describe the following requirements for filter construction:

1. Before filter placement, the base soil should be firm and, if necessary, be lightly tamped or rolled.
2. Clean filter material should be protected from contamination during and after placement; the placement method should minimize segregation in the filter.
3. Filters are compacted to 70 percent relative density unless otherwise specified, in a manner similar to free-draining sand-gravel backfill to prevent settlement.
4. Filter layers are often specified at a 6"minimum thickness. However, for extreme conditions such as high heads, variations in base material, or filter gradations that are near the extreme coarse limit, a minimum thickness of 8"may be specified. For zoned or graded filters these minimum thicknesses may be specified and are maintained for each layer.
5. Where drainage pipe is used in a filter system, hydraulic capacity of the pipe should be sufficient to collect the expected volume of seepage water and to convey it to a place for discharge.
6. While drainage pipe is being laid, the openings are often protected from inflow of fines from the filter material.

Figure 41 shows an example using filter criteria. Often ASTM C-33 concrete sand can serve as a protective filter material for many materials.

Recently, geosynthetic materials (geotextiles, geonets, and geocomposites) have gained wide acceptance for use as filters and drains in civil engineering works. These specialized materials have been used successfully in:

- Landfill liner systems
- Highway edge drains
- Retaining walls and basement walls drains
- Highway subgrade drains
- Strip or wick drains

Design methods for these materials, while similar to the design of soil filters, require specialized testing to determine required properties of the geosynthetics. Design methods and applications for these special materials is discussed in reference.

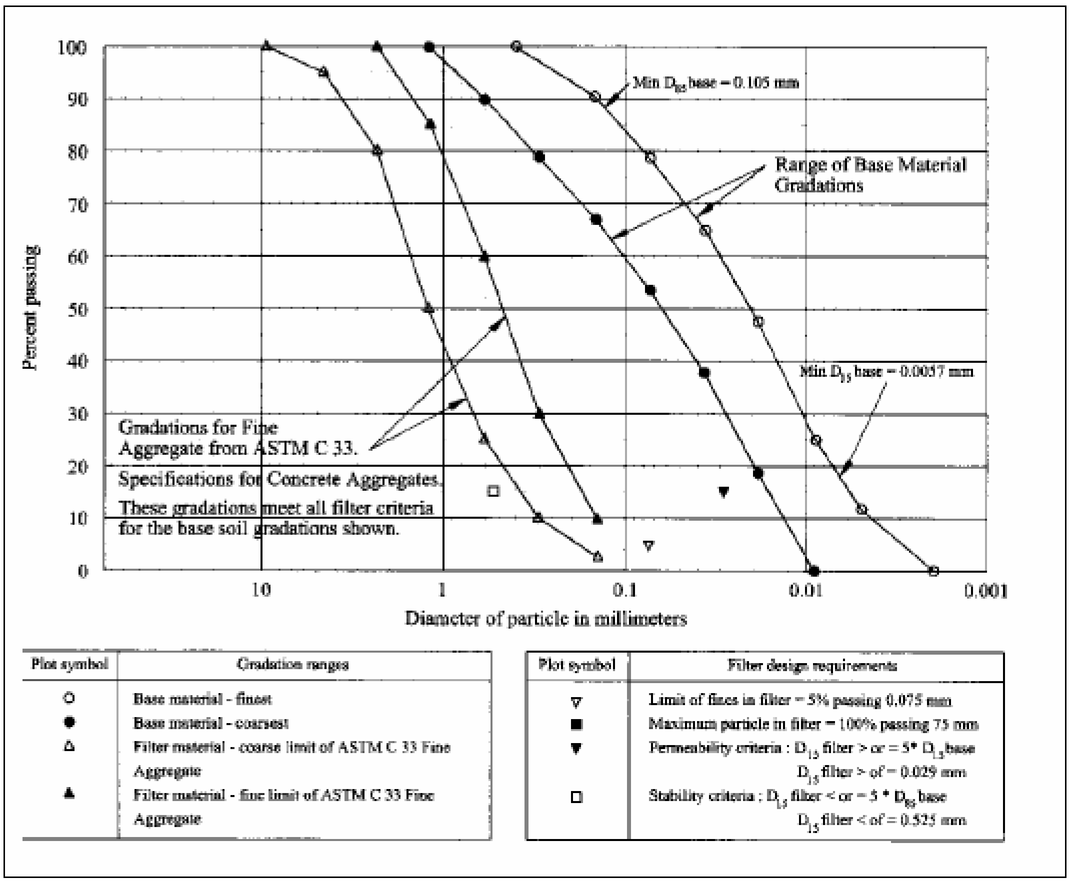


Figure 41, Filter gradations.

H. Stabilized Soils

39. General. Soil stabilization is the chemical or mechanical treatment of soil to improve its engineering properties. Chemically stabilized soils consist of soil and a small amount of additive such as cement, fly-ash, or lime. The additive is mixed with the soil, and the mixture is used in compacted fills, linings, or blankets. Quality and uniformity of the admixture and the uniformity of moisture are closely controlled to produce a high quality end product. Therefore, processing equipment and procedures are specified to ensure that the relatively small amount of additive is uniformly distributed throughout the soil mixture before placement and compaction. Uniformity of soil in the mixture is a major factor in controlling desired uniformity of the final product; and soil gradation, plasticity, and moisture content should be controlled prior to mixing with the additive, or prior to adding the stabilizer for in-place mixing.

A mixing plant must be calibrated over the range of soil gradation stated in the specifications. Adjustments to the mixing plant should then be made during construction to accommodate variation in soil gradation or for other variable conditions. These adjustments are based on mixing plant calibrations obtained before and during construction.

Sometimes, soils are stabilized to deeper depths by grouting or by injection methods to solve particular foundation problems (see sec. 19). These methods for stabilization require specialized knowledge and understanding of the materials being used, and quality control procedures are specifically developed for the particular situation.

40. Compacted Soil-Cement.

a. Design Considerations. Compacted soil-cement has been used as upstream slope protection for embankment dams and as blankets, linings, or other applications. Soil-cement used as upstream slope protection is placed in successive horizontal layers ranging from 6” to 12”in compacted thickness to protect the slope from wave action. The layers are placed successively up the slope, and the outer edges form a stairstep pattern. When soil-cement is placed as a blanket or lining it is usually placed in layers up to 2’thick with the layers parallel to the slope. Although roads are a minor part of the contactors work, compacted soil-cement has been used extensively for construction of road bases by others. Figure 42 show soil-cement facing constructed on the upstream slope of a structure.

To satisfy design requirements for slope protection, a layer must be:

1. Formed into a homogeneous, dense, permanently cemented mass that fulfills the requirements for compressive strength
2. In intimate contact with earth slopes, abutments, or concrete structures
3. Durable and resistant to "wetting and drying" and "freezing and thawing" action of water
4. Stable with respect to the structure and of sufficient thickness (mass) to resist displacement and uplift

Performance of soil-cement facings on government dams has generally been excellent. However, inspections of these facings have revealed that the bond between lifts is a weak point in the facing. Test results on cores taken from these faces show that the bond is much weaker than the remainder of the soil-cement. Since the layers are not well bonded, they perform as a series of nearly horizontal slabs on the slope of a dam. Each slab is offset from the previous one by a distance equal to the layer thickness multiplied by the slope of the dam. If the layers were well bonded, the entire facing would act as a massive unit instead of as individual layers, and damage by wave action would be greatly minimized.

Several studies have been performed to identify methods for enhancing bond between layers. Currently, the most promising method investigated is to apply a water-cement slurry to a layer just before placing the overlying layer. This technique was used at Davis Creek Dam in Nebraska to improve bonding between layers and to improve overall durability of the facing for an elevation of about 20”at normal reservoir operating level.

Figure 42, Soil-cement facing.

b. Construction Provisions.—The specifications describe the type and amount of cement, the quality and amount of water, and the borrow area for soil or aggregate. The permissible range of soil and aggregate gradation also is specified. If investigations show the deposit is variable, selective excavation and processing may be required to produce uniformity. Oversize particles and other objectionable materials must be removed.

A stationary mixing plant is usually required. Either a batch type or a continuous-feed pugmill type plant is acceptable. Control over mixing time; positive interlocking of cement and soil flow; and controls for accurately proportioning soil, cement, and water, all from appropriate storage, should be incorporated into the plant design.

Trucks for transporting the soil-cement mixture should have tight, clean, smooth beds and protective covers. The soil-cement spreader used for laying the soil-cement must produce a smooth uniform loose layer of required width and thickness. Usually, layers are placed horizontally; however, a slope toward the outer edge as steep as 8h: 1v is sometimes permitted to increase working width. The maximum time for hauling and spreading the soil-cement after mixing is usually specified as 30 minutes. A tractor and spreader box used for placing soil-cement is shown on figure 43.

Figure 43, Soil-cement placement with Tractor and Spreader Box.

Generally, compaction must be completed within 60 minutes after spreading with no more than 30 minutes between operations. Compaction is accomplished by several passes of a sheepsfoot tamping roller, followed by several passes of a pneumatic-tire roller, as shown on figure 44. The minimum number of passes by each roller should be determined by constructing a test section. The rollers should have provisions for ballast loading so the masses can be adjusted to provide optimum compaction. Vibratory, smooth steel drum rollers have been used to compact coarse grained soil-cement. A combination of vibratory and static mode may be used. The roller mass and the frequency and amplitude of vibration should be set for optimum compaction without damage to the soil-cement and to minimize localized shear failure in the upper part of a layer. A test section should always be constructed to determine: optimum equipment usage, roller mass, and number of passes, and vibratory characteristics if a vibratory roller is used. Rollers may be towed or self-propelled. After compaction, the compacted layer is cured by keeping the exposed surfaces continually moist using a fog spray until the overlying or adjacent layer is placed, or for a minimum of 7 days. A blanket of moist earth may be used for permanently exposed surfaces. The surface of a completed layer may require brushing to remove soil or other debris just before placing an overlying layer, as shown on figure 45.

Figure 44, Compaction of soil-cement.

Figure 45, Power brooming of surface prior to placement of the next lift.

c. Control Techniques. Compacted soil-cement is similar to compacted earthwork in that careful observations and additional control tests are required during the early stages to check planned control against results obtained under field conditions. These observations and tests are used toestablish placing conditions and to develop procedures for use in the remaining construction. Control should begin during excavation and stockpiling of the soil to ensure that the material is within gradation requirements and has a uniform moisture content. Gradation and moisture content are controlled by:

- Directing the excavation
- Mixing the stockpile by spreading and cross-dozing
- Sloping the stockpile surfaces to provide runoff without water catchments
- Sampling and testing during stockpiling

After the mixing plant is set up, the cement, soil, and water feeds must be calibrated individually to establish curves (or tables) of equipment settings versus quantity produced. The plant must be calibrated over the full range of anticipated production rates. A cement vane feeder and soil feed belt to a pugmill are shown on figure 46. Calibration is accomplished by timing and weighing quantities of moist soil, cement, and water to check feed and meter settings. To facilitate computation of dry soil production, moisture content of the wet soil should be determined at the plant by a quick method, a microwave oven, or the Moisture Teller device. The cement feed is perhaps the most crucial calibration because of variation in the amount of cement can critically affect the properties of soil-cement. The cement feed is calibrated over the range of required cement content by varying the feeder speed. The cement feed is usually quite consistent because cement is a fairly uniform product and is fed under reasonably uniform conditions in a well-designed plant. Cement feed calibration allows accurate adjustment if soil feed rates change during progress of the job. Plant performance and production is checked under full load by:

- Timing and weighing truckloads of the soil-cement mixture
- Checking mixing time
- Inspecting the mixture for:
  - Uniformity in texture
  - Moisture
  - Distribution of cement

The soil and soil-cement mixture is sampled to determine:

- Cement content
- Moisture content by both quick and ovendry methods
- Occurrence of "clay balls" (rounded balls of clayey fines and sand which do not break down during ordinary processing)

During construction, the inspector should check plant operation periodically. Inspection frequency will depend on performance of the plant and uniformity of soil gradation and moisture content. The soil feed rate should be checked at the beginning of each shift by timing and weighing a truckload of moist soil. By using a quick method to determine moisture content, the feed rate of dry soil can be computed, and the proper cement and water feed rates can be set. At regular intervals, the moisture content of the soil and soil-cement mixture should be determined by a quick method to provide a basis for making adjustments in the water feed if necessary. Also, the inspector does a check calibration each time a record control test is performed by timing the production and weighing a truckload of soil-cement. Based on water content and cement feed rate, the soil feed rate can be determined. The soil and soil-cement are sampled for moisture tests at the plant and for record tests in the laboratory.

The placement inspector should observe placing and compaction procedures and should verify that loose lift thickness, texture, and surface are uniform and that the lift is to specified dimensions.

Figure 46, Cement vane feeder and soil feed belt to pugmill.

Compaction begins with the roller starting at the outer edge. The number of passes and adequacy of overlap should be checked. When using a sheepsfoot roller, each pass over the entire width of the lift should be completed before the next pass begins, as the tamping feet tend to follow previous tracks if successive passes are made without overlap. Because of the sandy material, the sheepsfoot roller usually does not "walk out" completely but should begin to "walk out" on the last passes. Each layer is completed by compaction with a pneumatic-tire roller. This is in contrast to normal earthwork where successive layers are compacted together with a sheepsfoot roller. With the time limits used in soil-cement construction, it is usually not feasible to place successive layers quickly enough to compact them together; hence, each lift must be compacted individually.

The pneumatic-tire roller also starts compaction at the outer edge of the lift to minimize the amount of lateral spreading of the soil-cement. Smooth, even surfaces after compaction are desired. Rutting usually indicates a high placement water content. Thickness and width of the completed compacted lift should be checked. The placement inspector should be sure that the surface of the compacted layer is kept continuously moist until the overlying layer is placed, or for a minimum of 7 days. If required, the surface should be thoroughly cleaned by brooming (brushing) just before placing an overlying layer (fig. 45). Access ramps for trucks hauling soil-cement are built of soil on the soil-cement facing; these should also be checked to verify that they satisfy specified thickness requirements (2'). Ramps are constructed to adequately protect the edge of the top layers on which soil-cement is being placed. Inadequate ramps will result in damage to the outside edges of soil-cement layers and decrease durability of the finished product.

d. Control Testing. During construction, control testing is required for every 500 yd^3 of soil-cement placed, or a minimum of one test per shift. The control test consists of timing a sample through the process to ensure that specified time constraints are met. Times recorded are (1) when the truck leaves the plant, (2) when the load is spread, (3) when compaction is completed, and (4) when the control test is taken in the compacted material.

Control testing begins by timing the production of a truckload of soil-cement and obtaining a representative sample of soil from the soil feed. The mass of the timed truckload is determined, and a representative sample of soil-cement is obtained for laboratory testing. When the load is spread, the approximate center of the load is marked by the placement inspector. Moisture contents of the soil and soil-cement are determined by a quick method at the time of sampling, and the remainder of the samples are used for laboratory tests.

For the soil, the percentage of fines is determined, and the moisture content is obtained by the standard ovendrying method. The cement content of the soil can be determined by chemical titration. If the untreated soil contains a significant amount of calcium, a chemical analysis of the soil is performed. The presence of calcium should have been determined when the soil was being stockpiled. A complete gradation and specific gravity should be determined on specimens from every fourth control test.

For the soil-cement, the percentage of "clay balls" is determined, and the percentage of cement is determined by either the heat of neutralization method or the chemical titration method. A three-point compaction test is performed using the rapid method of construction control. The compaction test should be performed at the same time that material (from the sampled truck load) is being compacted. This is necessary to allow for time effects on compaction properties of soil-cement caused by hydration of cement.

The remainder of the soil-cement sample is used to prepare three or four compression test specimens to be tested at ages 7, 28, and 90 days. One 7-day and two 28-day specimens should be formed for each control test with an additional specimen for a 90-day test for every fourth control test. Specimens tested at 90-days are for design verification only and not for construction control. Specimens are placed at the density determined from the field density test of the compacted soil-cement layer. These specimens should be formed as soon as possible after the field density has been determined.

A field density test is performed at the point marked by the placement inspector when the timed load was spread. This test should be performed as soon as compaction is complete. Care should be taken so the test is not performed where roller overlap has occurred. When the test hole is complete, but before determining the volume, the depth of the lift is measured and recorded.

Control testing includes obtaining record cores from the compacted soil-cement at least 28, 90, and 360 days after completion. One hole should be drilled for each 5,000 yd^3 placed. Locations of core holes should be spaced to be representative of the area covered, with some cores near abutments or structures. In drilling, care should be exercised to obtain a continuous core; comments on bond strength should be included with other information accompanying the cores. The core holes should be carefully backfilled with cement grout and a reinforcing bar placed flush with the surface and located for future reference. Record cross sections of the compacted soil-cement are obtained at locations where 28-day core samples are taken. The compressive strength of a section of core from each of the holes should be determined in the field laboratory; this strength is compared to those of the record construction control cylinders representative of the immediate vicinity. The remainder of the cores should be sent to the laboratory for durability, direct shear, and compression tests. A summary of compressive strength and age of record core, and compressive strength of specimens of material in the immediate vicinity, should accompany the cores.

41. Compacted Soil-Lime.

a. General. Use of lime as a soil additive is the oldest known method of chemical stabilization; it was used by the Romans to construct the Appian Way. Soil-lime is a mixture of soil (usually clay), lime, and water which is compacted to form a dense mass. Experience has shown that mixtures of most clay soils, either quick or hydrated lime, and water will form cementitious products in a fairly short period of time.

b. Adding Lime to Highly Plastic Clay Soils. Adding lime to highly plastic clay soils produces several effects on physical properties:

1. The liquid limit decreases, and the plastic limit increases, radically decreasing the plasticity index (sometimes, by a factor of 4 or more)
2. The finer clay-size particles agglomerate to form larger particles, which makes the soil more friable and easier to work. By absorbing water, lime also assists in breaking up clay clods during mixing.
3. Lime dries the soil by absorbing water to hydrate the lime and makes wet soils easier to handle and compact.
4. Unconfined compressive strength increases many fold.

The governments Friant-Kern Canal, California, was constructed during 1945-51; about 54 mi of the canal traverse an area of expansive clay. Of these 54 mi, 23 mi are earth lined, and the remaining 31 mi are concrete lined. After 3 years of operation, portions of the canal traversing expansive clay soils began cracking, sloughing, and sliding with failures occurring in both the concrete-lined and earth-lined sections. Because these conditions caused continuing, expensive maintenance problems, in 1970, rehabilitation began for the worst failed areas. Lime stabilization was selected as the most effective method of treatment.

Riprap that had been dumped into slide areas was removed. Then, all material to be stabilized with lime and recompacted was removed by a benching operation. A series of long sloping benches or ramps were cut from the top of the bank down to the canal bottom, with the cut extending far enough into the slope to remove the entire depth of required excavation material. Two-percent quicklime was spread over the bench surface, and 1’of material from the bench was mixed with the quicklime, and the lime-clay mixture was pushed into the canal bottom. The material was spread on the canal bottom and an additional 2 percent lime was added. Water was added to at least 2 percentage points above optimum moisture, and about 0.3 m depth of material was mixed with dozers and graders. After about 6.6’of material had been mixed and cured for 24 hours, dozers began spreading the material on the slopes, which were then compacted with a sheepsfoot roller moving up and down the slope. The side slopes were constructed in three compacted lifts for a 3.6’compacted depth normal to the slope.

In subsequent rehabilitation work, the lime-treated soil was placed and compacted in successive horizontal layers stepped up the slope in the same manner in which soil-cement is placed on the face of an earth embankment. This placement method was found much more desirable than placing material parallel to the slope. Placing and compaction were much more efficient, and the finished product was of higher quality.

The amount of lime used was controlled by periodically placing a canvas on the ground where lime was to be spread, and after spreading, mass of lime was determined for a given area.

Four-percent lime by dry mass of soil was used during rehabilitation of Friant-Kern Canal; compressive strength increased to 20 times that of untreated soil. The rehabilitation has proved durable after about 20 years of additional service.

c. Adding Lime to Dispersive Clay Soils. Dispersive clay soils are those that erode in slow-moving or even quiet water by individual colloidal clay particles going into suspension and then being carried away by the flowing water. A concentrated leakage channel (crack) must be present for erosion to initiate in dispersive clay. This mechanism is totally different than that for piping where erosion begins at the discharge end of a leak and progresses upstream through the structure until it reaches the water source.

The design lime content for controlling dispersive clay soils is generally defined as the minimum lime content required to make the soil nondispersive. In addition, it may be desirable to increase the shrinkage limit to near optimum water content to prevent cracking from drying when using lime-treated soil in surface layers. In all known cases investigated to date, dispersive clay soils were made nondispersive by addition of 1 to 4 percent lime by dry mass of soil. However, in construction specifications, the design lime content is often increased 0.5 to 1.0 percent to account for losses, uneven distribution, incomplete mixing, etc.

Dispersive clay soils were identified throughout the borrow and foundation areas during investigations for McGee Creek Dam, Oklahoma; it was determined to be practical to stabilize these soils with lime during dam construction. Since dispersive clay soils were not concentrated in specific areas, lime treatment was considered more economical than attempting to identify the randomly occurring dispersive clays and selectively wasting them. Treating these clay soils with lime rendered them nondispersive and allowed their use in constructing the embankment-foundation contacts as erosion resistant material on the downstream slope of the embankments, and for placement as specially compacted backfill in areas of high piping potential such as along conduits through the embankment. Material was used directly from the borrow areas to construct the remainder of the dam and dike embankments. Specially designed filters were used to guard against any possible erosion of dispersive clay soils in untreated areas. Another benefit of using lime-treated soil was the improved workability of some of the highly plastic clay soil encountered.

d. Construction Procedures for Lime Treated Dispersive Clay Soils. The following general procedures have proved satisfactory for handling, mixing, and placing lime-treated soil.

Soil to be lime treated is pulverized in a high speed rotary mixer or with a disk harrow prior to applying lime, and the moisture content is brought to within 2 percent of optimum. Lime is uniformly spread on the pulverized soil to the specified percent lime by dry mass of soil. Lime is mixed with the soil using a rotary mixer, and additional water is added as necessary to again bring the mixture to within 2 percent of optimum (or other specified value). When mixing is completed, the soil-lime mixture is cured for at least 96 hours before placing and compacting. Exposed surfaces of the mixture are either lightly rolled to prevent moisture loss or the mixed material is stockpiled with the surface sealed.

Each section of the foundation is carefully prepared coincident with final mixing and pulverization of the lime-treated material. The soil-lime is mixed until 100 percent passes the 1”sieve and 60 percent passes the 1”and No. 4. Immediately after final mixing, the lime-treated earth fill is placed and compacted in horizontal lifts of no more than 6”after compaction. The material is compacted to no less than 95 percent laboratory maximum dry density, using a tamping roller followed by a pneumatic-tire roller. The top of each compacted lift is scarified or disked before the next lift is placed.
The following items should be monitored to ensure high quality earthwork construction control:

1. Soil pulverization (gradation)
2. Lime content
3. Soil dispersivity (before and after lime treatment)
4. Compacted density
5. Moisture content of both soil and soil-lime mixture

If embankment materials are placed on the compacted lime-treated earth fill within 36 hours, special curing provisions are not required. Otherwise, the exposed surface of the lime-treated earth fill is compacted with a neumatic-tire sealer to seal the surface, and it is sprinkled with water for 7 days or until embankment material is placed. Construction control testing is the same as that for other earthwork.

Test procedures discussed in this course can be obtained for the U.S. Department of the Interior, Bureau of Reclamation (USBR).