Chapter 7
Embankment Design

7-1. Embankment Materials

a. Earth-fill materials.

(1) While most soils can be used for earth-fill construction as long as they are insoluble and substantially inorganic, typical rock flours and clays with liquid limits above 80 should generally be avoided. The term "soil" as used herein includes such materials as soft sandstone or other rocks that break down into soil during handling and compaction.

(2) If a fine-grained soil can be brought readily within the range of water contents suitable for compaction and for operation of construction equipment, it can be used for embankment construction. Some slow-drying impervious soils may be unusable as embankment fill because of excessive moisture, and the reduction of moisture content would be impracticable in some climatic areas because of anticipated rainfall during construction. In other cases, soils may require additional water to approach optimum water content for compaction. Even ponding or sprinkling in borrow areas may be necessary. The use of fine-grained soils having high water contents may cause high porewater pressures to develop in the embankment under its own weight. Moisture penetration into dry hard borrow material can be aided by ripping or plowing prior to sprinkling or ponding operations.

(3) As it is generally difficult to reduce substantially the water content of impervious soils, borrow areas containing impervious soils more than about 2 to 5 percent wet of optimum water content (depending upon their plasticity characteristics) may be difficult to use in an embankment. However, this depends upon local climatic conditions and the size and layout of the work, and must be assessed for each project on an individual basis. The cost of using drier material requiring a longer haul should be compared with the cost of using wetter materials and flatter slopes. Other factors being equal, and if a choice is possible, soils having a wide range of grain sizes (well-graded) are preferable to soils having relatively uniform particle sizes, since the former usually are stronger, less susceptible to piping, erosion, and liquefaction, and less compressible. Cobbles and boulders in soils may add to the cost of construction since stone with maximum dimensions greater than the thickness of the compacted layer must be removed to permit proper compaction. Embankment soils that undergo considerable shrinkage upon drying should be protected by adequate thicknesses of nonshrinking fine-grained soils to reduce evaporation. Clay soils should not be used as backfill in contact with concrete or masonry structures, except in the impervious zone of an embankment.

(4) Most earth materials suitable for the impervious zone of an earth dam are also suitable for the impervious zone of a rock-fill dam. When water loss must be kept to a minimum (i.e., when the reservoir is used for long-term storage), and fine-grained material is in short supply, resulting in a thin zone, the material used in the core should have a low permeability. Where seepage loss is less important, as in some flood control dams not used for storage, less impervious material may be used in the impervious zone.

b. Rock-fill materials.

(1) Sound rock is ideal for compacted rock-fill, and some weathered or weak rocks may be suitable, including sandstones and cemented shales (but not clay shales). Rocks that break down to fine sizes during excavation, placement, or compaction are unsuitable as rock-fill, and such materials should be treated as soils. Processing by passing rock-fill materials over a grizzly may be required to remove excess fine sizes or oversize material. If splitting/processing is required, processing should be limited to the minimum amount that will

achieve required results. For guidance in producing satisfactory rock-fill material and for test quarrying, reference should be made to EM 1110-2-3800 and EM 1110-2-2302.

(2) In climates where deep frost penetration occurs, a more durable rock is required in the outer layers than in milder climates. Rock is unsuitable if it splits easily, crushes, or shatters into dust and small fragments. The suitability of rock may be judged by examination of the effects of weathering action in outcrops. Rock-fill composed of a relatively wide gradation of angular, bulk fragment settles less than if composed of flat, elongated fragments that tend to bridge and then break under stresses imposed by overlying fill. If rounded cobbles and boulders are scattered throughout the mass, they need not be picked out and placed in separate zones.

7-2. Zoning

The embankment should be zoned to use as much material as possible from required excavation and from borrow areas with the shortest haul distances and the least waste. Embankment zoning should provide an adequate impervious zone, transition zones between the core and the shells, seepage control, and stability. Gradation of the materials in the transition zones should meet the filter criteria presented in Appendix B.

a. Earth dams.

(1) In a common type of earth fill embankment, a central impervious core is flanked by much more pervious shells that support the core (Figures 2-1b and 2-1c). The upstream shell affords stability against end of construction, rapid drawdown, earthquake, and other loading conditions. The downstream shell acts as a drain that controls the line of seepage and provides stability under high reservoir levels and during earthquakes. For the most effective control of through seepage and seepage during reservoir drawdown, the permeability should increase progressively from the core out toward each slope. Frequently suitable materials are not available for pervious downstream shells. In this event, control of seepage through the embankment is provided by internal drains as discussed in paragraph 6-2*a*(3).

(2) The core width for a central impervious core-type embankment should be established using seepage and piping considerations, types of material available for the core and shells, the filter design, and seismic considerations. In general, the width of the core at the base or cutoff should be equal to or greater than 25 percent of the difference between the maximum reservoir and minimum tailwater elevations. The greater the width of the contact area between the impervious fill and rock, the less likely that a leak will develop along this contact surface. Where a thin embankment core is selected, it is good engineering to increase the width of the core at the rock juncture, to produce a wider core contact area. Where the contact between the impervious core and rock is relatively narrow, the downstream filter zone becomes more important. A core top width of 10 ft is considered to be the minimum for construction equipment. The maximum core width will usually be controlled by stability and availability of impervious materials.

(3) A dam with a core of moderate width and strong, adequate pervious outer shells may have relatively steep outer slopes, limited primarily by the strength of the foundation and by maintenance considerations.

(4) Where considerable freezing takes place and soils are susceptible to frost action, it is desirable to terminate the core at or slightly below the bottom of the frost zone to avoid damage to the top of the dam. Methods for determination of depths of freeze and thaw in soils are given in TM 5-852-6. For design of road pavements on the top of the dam under conditions of frost action in the underlying core, see TM 5-822-5.

(5) Considerable volumes of soils of a random nature or intermediate permeability are usually obtained from required excavations and in excavating select impervious or pervious soils from borrow areas. It is generally economical to design sections in which these materials can be utilized, preferably without stockpiling. Where random zones are large, vertical (or inclined) and horizontal drainage layers within the downstream portion of the embankment can be used to control seepage and to isolate the downstream zone from effects of through

seepage. Random zones may need to be separated from pervious or impervious zones by suitable transition zones. Homogeneous embankment sections are considered satisfactory only when internal vertical (or inclined) and horizontal drainage layers are provided to control through seepage. Such embankments are appropriate where available fill materials are predominantly of one soil type or where available materials are so variable it is not feasible to separate them as to soil type for placement in specific zones and when the height of the dam is relatively low. However, even though the embankment is unzoned, the specifications should require that more pervious material be routed to the outer portions of the embankment.

b. Examples of earth dams.

(1) Examples of embankment sections of earth dams constructed by the Corps of Engineers are shown in Figures 7-1. Prompton Dam, a flood control project (Figure 7-1a), illustrates an unzoned embankment, except for interior inclined and horizontal drainage layers to control through seepage.

(2) Figure 7-1b, Alamo Dam, shows a zoned embankment with an inclined core of sandy clay and outer pervious shells of gravelly sand. The core extends through the gravelly sand alluvium to the top of rock, and the core trench is flanked on the downstream side by a transition layer of silty sand and a pervious layer of gravelly sand.

(3) Where several distinctively different materials are obtained from required excavation and borrow areas, more complex embankment zones are used, as illustrated by Figure 7-2a, Milford Dam, and Figure 7-2b, W. Kerr Scott Dam. The embankment for Milford Dam consists of a central impervious core connected to an upstream impervious blanket, an upstream shell of shale and limestone from required excavation, an inclined and horizontal sand drainage layer downstream of the core, and downstream random fill zone consisting of sand, silty sand, and clay. The embankment of W. Kerr Scott Dam consists of an impervious zone of low plasticity silt, sloping upstream from the centerline and flanked by zones of random material (silty sands and gravels). Inclined and horizontal drainage layers are provided in the downstream random zone. Since impervious materials are generally weaker than the more pervious and less cohesive soils used in other zones, their location in a central core flanked by stronger material permits steeper embankment slopes than would be possible with an upstream sloping impervious zone. An inclined core near the upstream face may permit construction of pervious downstream zones during wet weather with later construction of the sloping impervious zone during dry weather. This location often ensures a better seepage pattern within the downstream portion of the embankment and permits a steeper downstream slope than would a central core.

c. Rock-fill dams. Impervious zones, whether inclined or central, should have sufficient thickness to control through seepage, permit efficient placement with normal hauling and compacting equipment, and minimize effect of differential settlement and possible cracking. The minimum horizontal thickness of core, filter, or transition zones should be 10 ft. For design considerations where earthquakes are a factor, see paragraphs 4-6 and 6-8.

d. Examples of rock-fill dams. Embankment sections of four Corps of Engineers rock-fill dams are shown in Figures 7-3 and 7-4. Variations of the two principal types of embankment zoning (central impervious core and upstream inclined impervious zone) are illustrated in these figures.

7-3. Cracking

a. General. Cracking develops within zones of tensile stresses within earth dams due to differential settlement, filling of the reservoir, and seismic action. Since cracking can not be prevented, the design must include provisions to minimize adverse effects. Cracks are of four general types: transverse, horizontal, longitudinal, and shrinkage. Shrinkage cracks are generally shallow and can be treated from the surface by removing the cracked material and backfilling (Walker 1984, Singh and Sharma 1976, Jansen 1988).

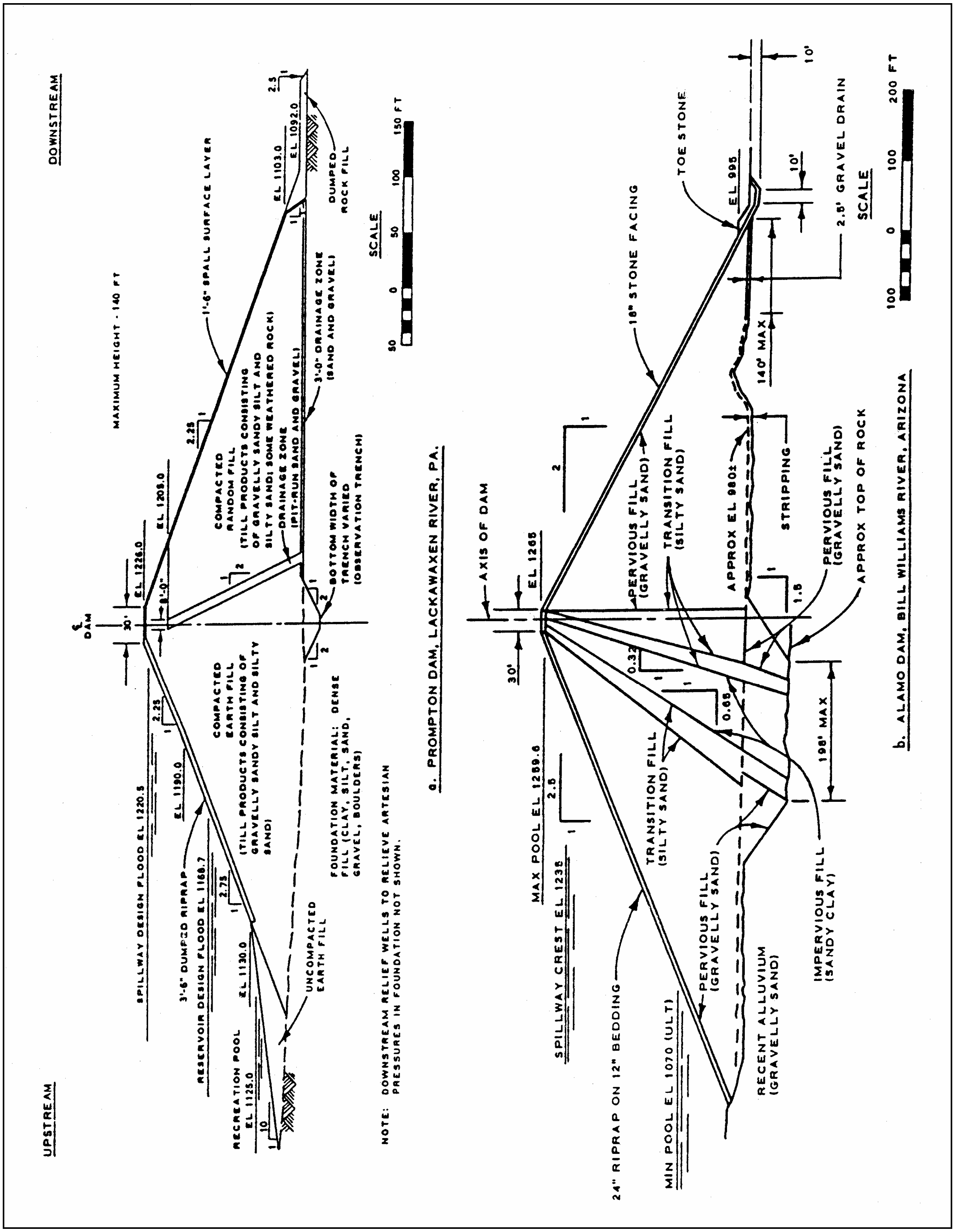

Figure 7-1. Typical embankment sections, earth dams (Prompton and Alamo Dams)

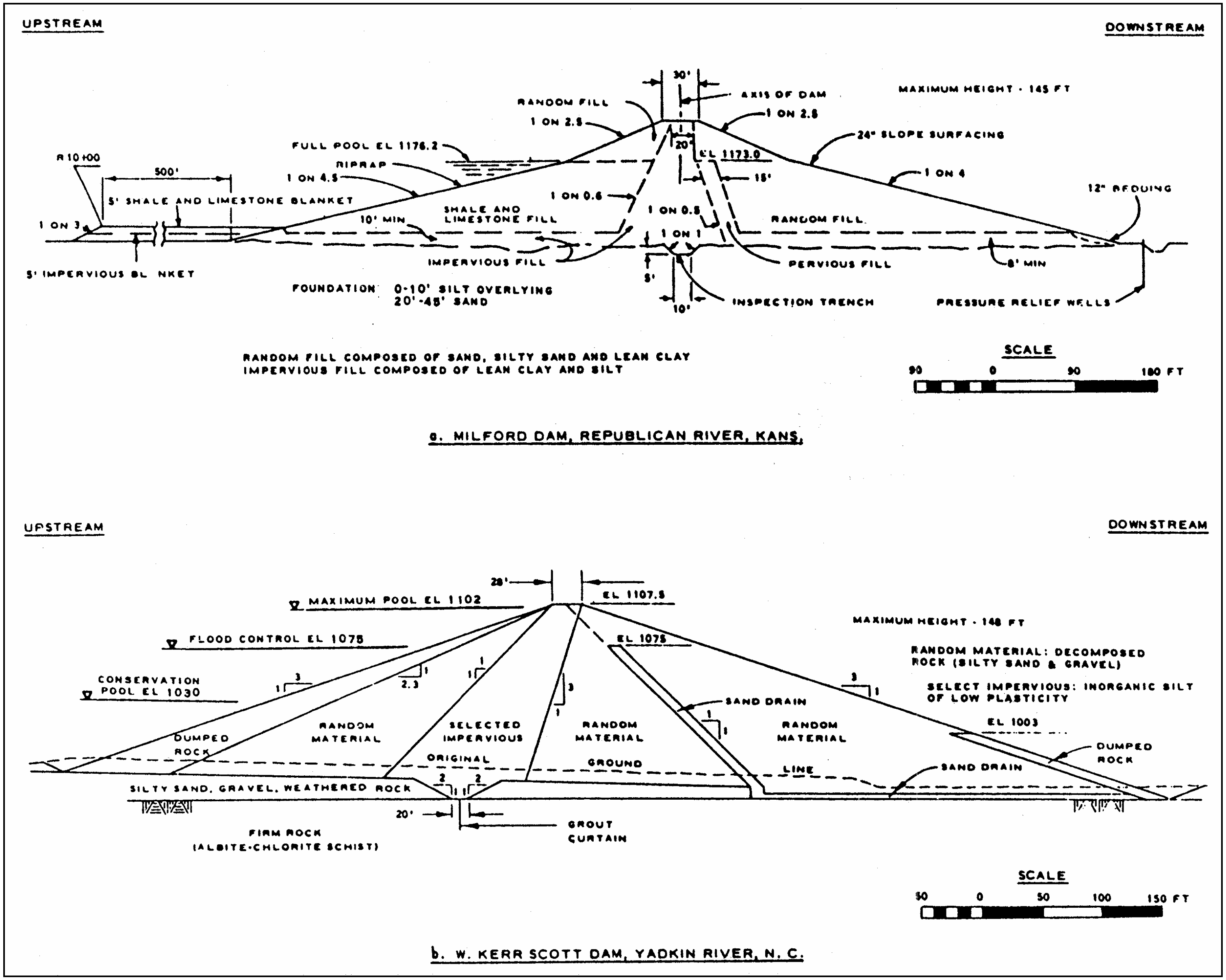

Figure 7-2. Typical embankment sections, earth dams (Milford and W. Kerr Scott Dams)

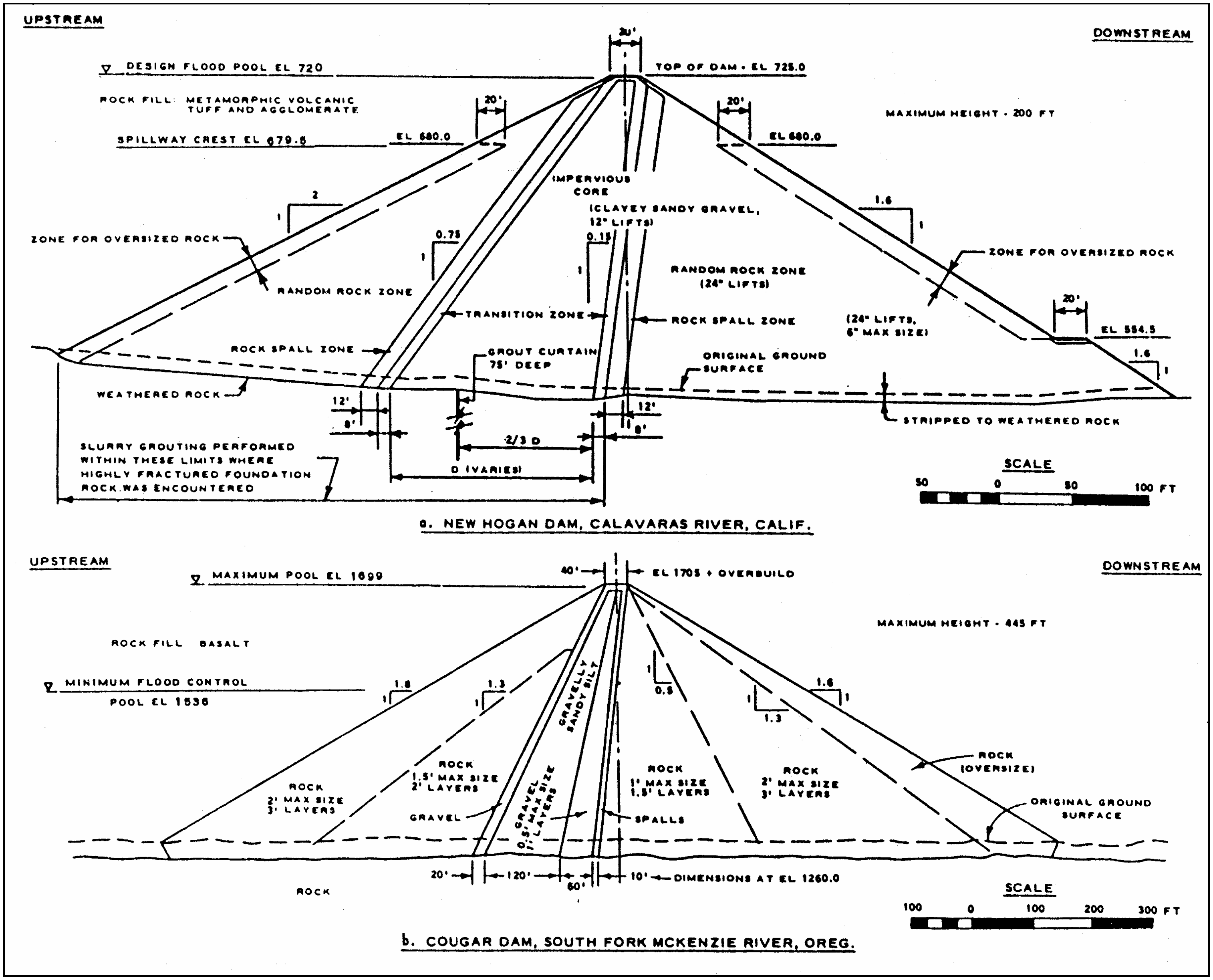

Figure 7-3. Embankment sections, rock-fill dams (New Hogan and Cougar Dams)

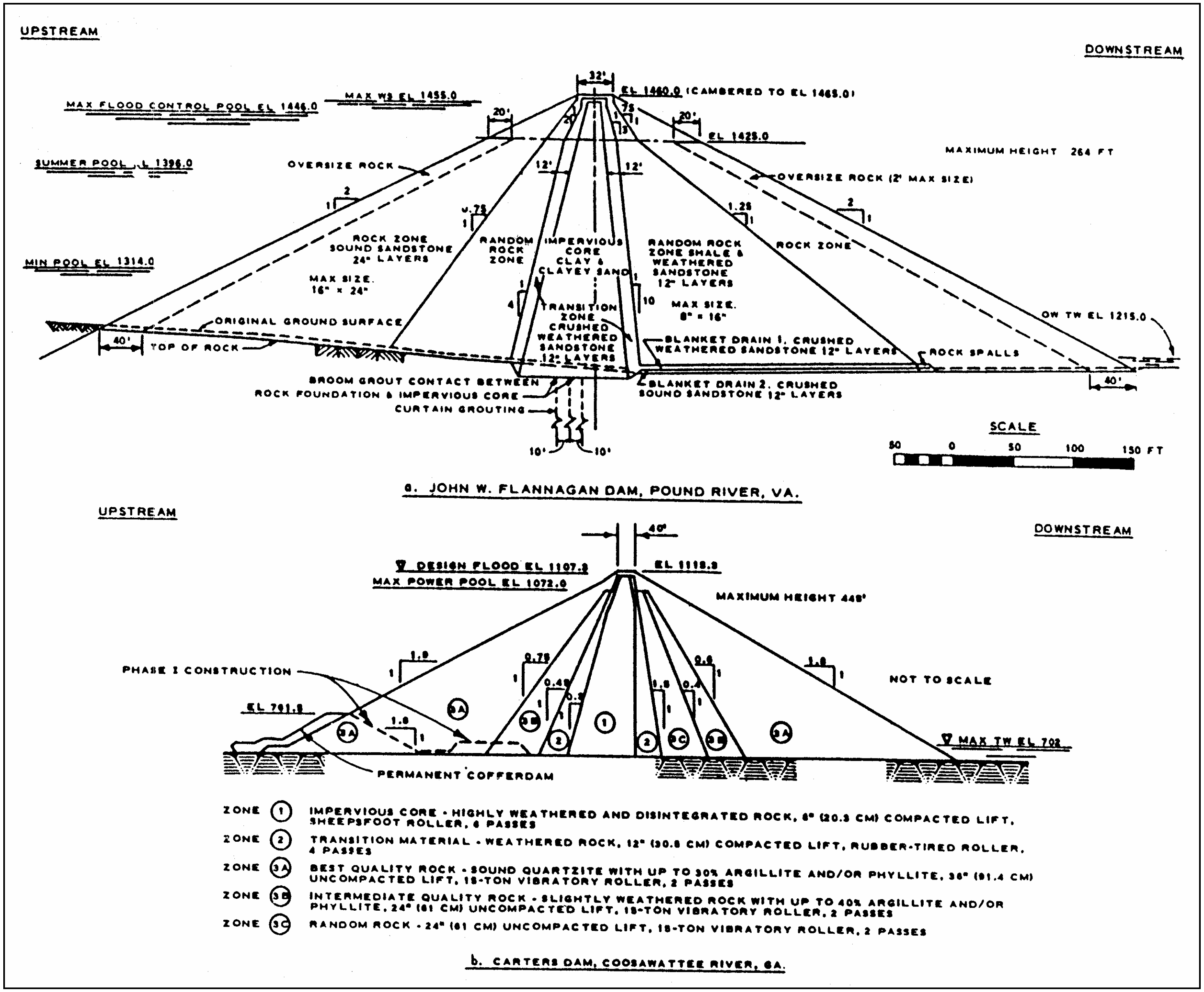

Figure 7-4. Embankment sections, rock-fill dams (John W. Flannagan and Carters Dams)

b. Transverse cracking. Transverse cracking of the impervious core is of primary concern because it creates flow paths for concentrated seepage through the embankment. Transverse cracking may be caused by tensile stresses related to differential embankment and/or foundation settlement. Differential settlement may occur at steep abutments, at the junction of a closure section, at adjoining structures where compaction is difficult, or over old stream channels or meanders filled with compressible soils.

c. Horizontal cracking. Horizontal cracking of the impervious core may occur when the core material is much more compressible than the adjacent transition or shell material so that the core material tends to arch across the less compressible adjacent zones resulting in a reduction of the vertical stress in the core. The lower portion of the core may separate out, resulting in a horizontal crack. Arching may also occur if the core rests on highly compressible foundation material. Horizontal cracking is not visible from the outside and may result in damage to the dam before it is detected.

d. Longitudinal cracking. Longitudinal cracking may result from settlement of upstream transition zone or shell due to initial saturation by the reservoir or due to rapid drawdown. It may also be due to differential settlement in adjacent materials or seismic action. Longitudinal cracks do not provide continuous open seepage paths across the core of the dam, as do transverse and horizontal cracks, and therefore pose no threat with regard to piping through the embankment. However, longitudinal cracks may reduce the overall embankment stability leading to slope failure, particularly if the cracks fill with water.

e. Defensive measures. The primary line of defense against a concentrated leak through the dam core is the downstream filter (filter design is covered in Appendix B). Since prevention of cracks cannot be ensured, an adequate downstream filter must be provided (Sherard 1984). Other design measures to reduce the susceptibility to cracking are of secondary importance. The susceptibility to cracking can be reduced by shaping the foundation and structural interfaces to reduce differential settlement, densely compacting the upstream shell to reduce settlement from saturation, compacting core materials at water contents sufficiently high so that stress-strain behavior is relative plastic, i.e., low deformation moduli, and shear strength, so that cracks cannot remain open (pore pressure and stability must be considered), and staged construction to lessen the effects of settlement of the foundation and the lower parts of the embankment.

7-4. Filter Design

The filter design for the drainage layers and internal zoning of a dam is a critical part of the embankment design. It is essential that the individual particles in the foundation and embankment are held in place and do not move as a result of seepage forces. This is accomplished by ensuring that the zones of material meet "filter criteria" with respect to adjacent materials. The criteria for a filter design is presented in Appendix B. In a zoned embankment the coarseness between the fine and coarse zones may be such that an intermediate or transition section is required. Drainage layers should also meet these criteria to ensure free passage of water. All drainage or pervious zones should be well compacted. Where a large carrying capacity is required, a multilayer drain should be provided. Geotextiles (filter fabrics) should not be used in or on embankment dams.

7-5. Consolidation and Excess Porewater Pressures

a. Foundations.

(1) Foundation settlement should be considered in selecting a site since minimum foundation settlements are desirable. Overbuilding of the embankment and of the core is necessary to ensure a dependable freeboard. Stage construction or other measures may be required to dissipate high porewater pressures more rapidly. Wick drains should be considered except where installation would be detrimental to seepage characteristics of the structure and foundation. If a compressible foundation is encountered, consolidation tests should be performed on undisturbed samples to provide data from which settlement analyses can be made for use in comparing sites and

for final design. Procedures for making settlement and bearing capacity analyses are given in EM 1110-1-1904 and EM 1110-1-1905, respectively. Instrumentation required for control purposes is discussed in Chapter 10.

(2) The shear strength of a soil is affected by its consolidation characteristics. If a foundation consolidates slowly, relative to the rate of construction, a substantial portion of the applied load will be carried by the pore water, which has no shear strength, and the available shearing resistance is limited to the in situ shear strength as determined by undrained “Q” tests. Where the foundation shearing resistance is low, it may be necessary to flatten slopes, lengthen the time of construction, or accelerate consolidation by drainage layers or wick drains. Analyses of foundation porewater pressures are covered by Snyder (1968). Procedures for stability analyses are discussed in EM 1110-2-1902 and Edris (1992).

(3) Although excess porewater pressures developed in pervious materials dissipate much more rapidly than those in impervious soils, their effect on stability is similar. Excess pore pressures may temporarily build up, especially under earthquake loadings, and effective stresses contributing to shearing resistance may be reduced to low values. In liquefaction of sand masses, the shearing resistance may temporarily drop to a fraction of its normal value.

b. Embankments. Factors affecting development of excess porewater pressures in embankments during construction include placement water contents, weight of overlying fill, length of drainage path, rate of construction (including stoppages), characteristics of the core and other fill materials, and drainage features such as inclined and horizontal drainage layers, and pervious shells. Analyses of porewater pressures in embankments are presented by Clough and Snyder (1966). Spaced vertical sand drains within the embankment should not be used in lieu of continuous drainage layers because of the greater danger of clogging by fines during construction.

7-6. Embankment Slopes and Berms

a. Stability. The stability of an embankment depends on the characteristics of foundation and fill materials and also on the geometry of the embankment section. Basic design considerations and procedures relating to embankment stability are discussed in detail in EM 1110-2-1902 and Edris (1992).

b. Unrelated factors. Several factors not related to embankment stability influence selection of embankment slopes. Flatter upstream slopes may be used at elevations where pool elevations are frequent (usually ±4 ft of conservation pool). In areas where mowing is required, the steepest slope should be 1 vertical on 3 horizontal to ensure the safety of maintenance personnel. Horizontal berms, once frequently used on the downstream slope, have been found undesirable because they tend to trap and concentrate runoff from upper slope surfaces. The water often cannot be disposed of adequately, whereupon it spills over the berm and erodes the lower slopes. A horizontal upstream berm at the base of the principal riprap protection has been found useful in placing and maintaining riprap.

c. Waste berms. Where required excavation or borrow area stripping produces material unsuitable for use in the embankment, waste berms can be used for upstream slope protection, or to contribute to the stability of upstream and downstream embankment slopes. Care must be taken, however, not to block drainage in the downstream area by placing unsuitable material, which is often impervious, over natural drainage features. The waste berm must be stable against erosion or it will erode and expose the upstream slope.

7-7. Embankment Reinforcement

The use of geosynthetics (geotextiles, geogrids, geonets, geomembranes, geocomposites, etc.) in civil engineering has been increasing since the 1970's. However, their use in dam construction or repairs, especially in the United States, has been limited (Roth and Schneider 1991; Giroud 1989a, 1989b; Giroud 1990, Giroud 1992a, 1992b). The Corps of Engineers pioneered the use of geotextiles to reinforce very soft foundation soils

(Fowler and Koerner 1987, Napolitano 1991). The Huntington District of the Corps of Engineers used a welded wire fabric geogrid for reconstruction of Mohicanville Dike No. 2 (Fowler et al. 1986; Franks, Duncan, and Collins 1991). The Bureau of Reclamation has used geogrid reinforcement to steepen the upper portion of the downstream slope of Davis Creek Dam, Nebraska (Engemoen and Hensley 1989, Dewey 1989).

7-8. Compaction Requirements

a. Impervious and semi-impervious fill.

(l) General considerations.

(a) The density, permeability, compressibility, and strength of impervious and semi-impervious fill materials are dependent upon water content at the time of compaction. Consequently, the design of an embankment is strongly influenced by the natural water content of borrow materials and by drying or wetting that may be practicable either before or after delivery to the fill. While natural water contents can be decreased to some extent, some borrow soils are so wet they cannot be used in an embankment unless slopes are flattened. However, water contents cannot be so high that hauling and compaction equipment cannot operate satisfactorily. The design and analysis of an embankment section require that shear strength and other engineering properties of fill material be determined at the densities and water contents that will be obtained during construction. In general, placement water contents for most projects will fall within the range of 2 percent dry to 3 percent wet of optimum water content as determined by the standard compaction test (EM 1110-2-1906). A narrower range will be required for soils having compaction curves with sharp peaks.

(b) While use of water contents that are practically obtainable is a principal construction requirement, the effect of water content on engineering properties of a compacted fill is of paramount design interest. Soils that are compacted wet of optimum water content exhibit a somewhat plastic type of stress-strain behavior (in the sense that deformation moduli are relatively low and stress-strain curves are rounded) and may develop low “Q” strengths and high porewater pressures during construction. Alternatively, soils that are compacted dry of optimum water content exhibit a more rigid stress-strain behavior (high deformation moduli), develop high “Q” strengths and low porewater pressures during construction, and consolidate less than soils compacted wet of optimum water content. However, soils compacted substantially dry of optimum water content may undergo undesirable settlements upon saturation. Cracks in an embankment would tend to be shallower and more self-healing if compacting is on the wet side of optimum water content than if on the dry side. This results from the lower shear strength, which cannot support deep open cracks, and from lower deformation moduli.

(c) Stability during construction is determined largely by “Q” strengths at compacted water contents and densities. Since “Q” strengths are a maximum for water contents dry of optimum and decrease with increasing water content, construction stability is determined (apart from foundation influences) by the water contents at which fill material is compacted. This is equivalent to saying that porewater pressures are a controlling factor on stability during construction. “Q” strengths, and pore-water pressures during construction are of more importance for high dams than for low dams.

(d) Stability during reservoir operating conditions is determined largely by “R” strengths for compacted material that has become saturated. Since “R” strengths are a maximum at about optimum water content, shear strengths for fill water contents both dry and wet of optimum must be established in determining the allowable range of placement water contents. In addition, the limiting water content on the dry side of optimum must be selected to avoid excessive settlement due to saturation. Preferably no settlement on saturation should occur.

(2) Dams on weak, compressible foundations. Where dams are constructed on weak, compressible foundations, the embankment and foundation materials should have stress-strain characteristics as nearly similar as possible. Embankments can be made more plastic and will adjust more readily to settlements if they are compacted wet of the optimum water content. Differences in the stress-strain characteristics of the embankment and

foundation may result in progressive failure. To prevent this from occurring, the embankment is designed so that neither the embankment nor the foundation will be strained beyond the peak strength so that the stage where progressive failure begins will not be reached. Strength reduction factors for the embankment and foundation are given in Figure 7-5 (Duncan and Buchignani 1975, Chirapuntu and Duncan 1976).

(3) Dams on strong, incompressible foundations. Where the shear strength of the embankment is lower than that of the foundation, such as the case where there is a strong, relatively incompressible foundation, the strength of the fill controls the slope design. The “Q” strength of the fill will be increased by compacting it at water contents at or slightly below optimum water contents and the porewater pressures developed during construction will be reduced. Soils compacted slightly dry of optimum water content generally have higher permeability values and lower “R” strengths than those wet of optimum water content. Further, many soils will consolidate upon saturation if they are compacted dry of optimum water content. All of these factors must be considered in the selection of the range of allowable field compaction water contents.

(4) Abutment areas. In abutment areas, large differential settlements may take place within the embankment if the abutment slopes are steep or contain discontinuities such as benches or vertical faces. This may induce tension zones and cracking in the upper part of the embankment. It may be necessary to compact soils wet of optimum water content in the upper portion of embankment to eliminate cracking due to differential settlements. Again, shear strength must be taken into account.

(5) Field densities. Densities obtained from field compaction using conventional tamping or pneumatic rollers and the standard number of passes of lift thickness are about equal to or slightly less than maximum densities for the standard compaction test. This has established the practice of using a range of densities for performance of laboratory tests for design. Selection of design densities, while a matter of judgment, should be based on the results of test fills or past experience with similar soils and field compaction equipment. The usual assumption is that field densities will not exceed the maximum densities obtained from the standard compaction test nor be less than 95 percent of the maximum densities derived from this test.

(6) Design water contents and densities. A basic concept for both earth and rock-fill dams is that of a core surrounded by strong shells providing stability. This concept is obvious for rock-fill dams and can be applied even to internally drained homogeneous dams. In the latter case, the core may be compacted at or wet of optimum while the outer zones are compacted dry of optimum. The selection of design ranges of water contents and densities requires judgment and experience to balance the interaction of the many factors involved. These include:

(a) Borrow area water contents and the extent of drying or wetting that may be practicable.

(b) The relative significance on embankment design of “Q” versus “R” strengths (i.e., construction versus operating conditions).

(c) Climatic conditions.

(d) The relative importance of foundation strength on stability.

(e) The need to design for cracking and development of tension zones in the upper part of the embankment, especially in impervious zones.

(f) Settlement of compacted materials on saturation.

(g) The type and height of dam.

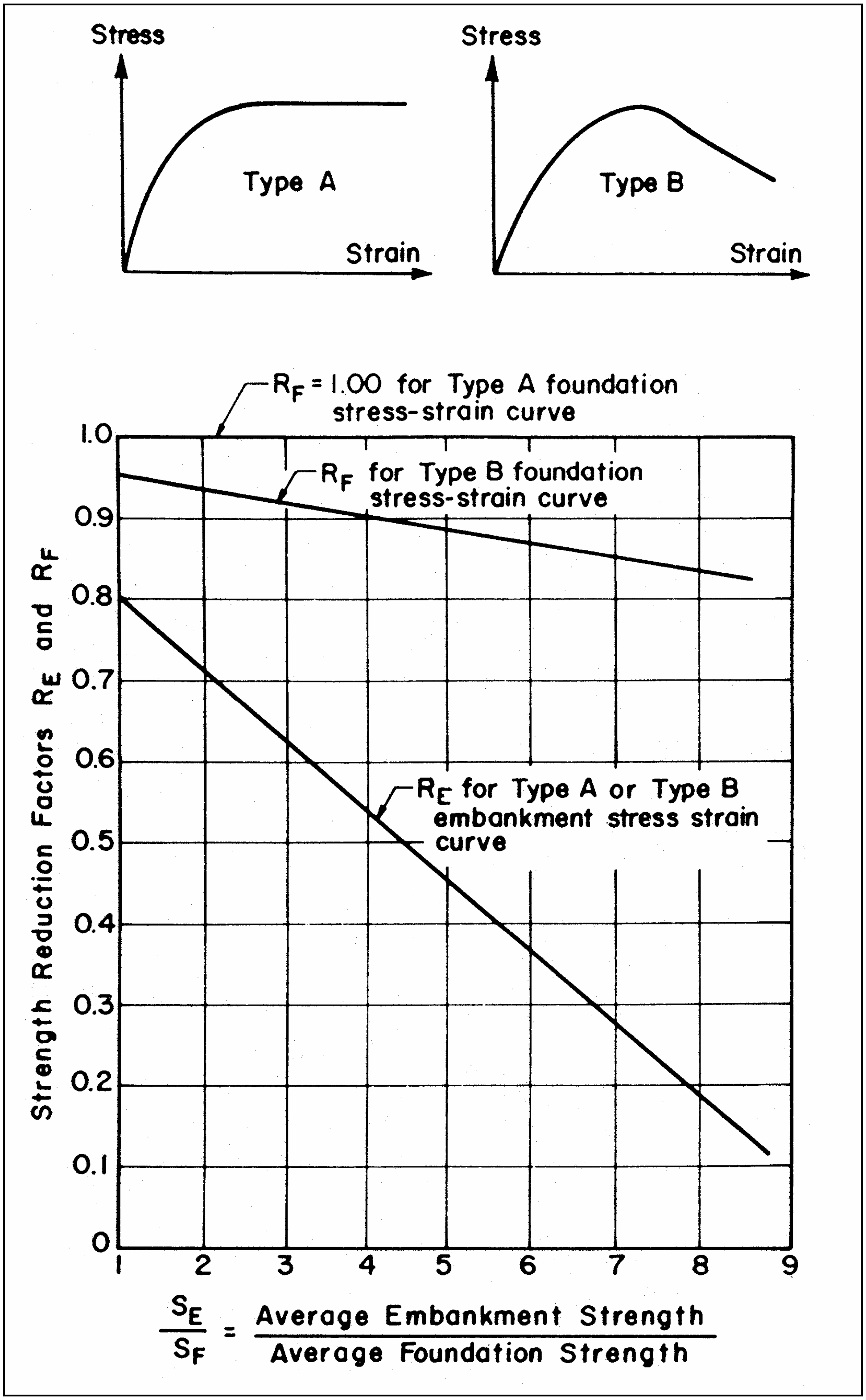

Figure 7-5. Peak strength correction factors for both embankment and foundation to prevent progressive failure in the foundation for embankments on soft clay foundations (Duncan and Buchignani 1975)

(h) The influence on construction cost of various ranges of design water contents and densities.

(7) Field compaction.

(a) While it is generally impracticable to consider possible differences between field and laboratory compaction when selecting design water contents and densities, such differences do exist and result in a different behavior from that predicted using procedures discussed in preceding paragraphs. Despite these limitations, the procedures described generally result in satisfactory embankments, but the designer must verify that this is true as early as possible during embankment construction. This can often be done by incorporating a test section within the embankment. When field test section investigations are performed, field compaction curves should be developed for the equipment used.

(b) Proper compaction at the contact between the embankment and the abutments is important. Sloping the fill surface up on a 10 percent grade toward a steep abutment facilitates compaction where heavy equipment is to be used. Where compaction equipment cannot be used against an abutment, thin lifts tamped with hand-operated powered tampers should be used, but tamping of soil under overhangs in lieu of removal or backfilling with concrete should not be permitted.

(c) Specific guidance on acceptable characteristics and operating procedures of tamping rollers, rubber-tired rollers, and vibratory rollers is given in guide specification UFGS-02330A, including dimensions, weights, and speed of rolling; also see EM 1110-2-1911.

b. Pervious materials (excluding rock-fill).

(l) The average in-place relative density of zones containing cohesionless soils should be at least 85 percent, and no portion of the fill should have a relative density less than 80 percent. This requirement applies to drainage and filter layers as well as to larger zones of pervious materials, but not to bedding layers beneath dumped riprap slope protection. The requirement also applies to filter layers and pervious backfill beneath and/or behind spillway structures. The relative density test is generally satisfactory for pervious materials containing only a few percent finer than the No. 200 sieve. For some materials, however, field compaction results equal to 100 percent or more of the standard compaction test maximum density can be readily obtained and may be higher than 85 percent relative density. If 98 percent of the maximum density from the standard compaction test is higher than 85 percent relative density, the standard compaction test should be used. The design should provide that clean, free-draining pervious materials be compacted in as nearly a saturated condition as possible. Otherwise compaction at bulking water contents might result in settlement upon saturation.

(2) It is possible to place pervious fill such as free-draining gravel or fine to coarse sand, into a lift 3 to 4 ft thick in shallow water and to obtain good compaction by rolling the emerged surface of the lift with heavy crawler tractors. However, less pervious soils cannot be compacted if placed in this manner or even on a wet subgrade. In general, sand containing more than 8 to 10 percent finer than the No. 200 sieve cannot be placed satisfactorily underwater, and well graded sand-gravel mixtures must contain even fewer fines. The ability to place pervious soils in shallow water after stripping simplifies construction and makes it possible to construct cofferdams of pervious material by adding a temporary impervious blanket on the outer face and thus permit unwatering for the impervious cutoff section. The cofferdams subsequently become part of the pervious shells of the embankment.

c. Rock-fill.

(l) It is often desirable, especially where rocks are soft, for procedures to be used in compacting rock-fill materials to be selected on the basis of test fills, in which lift thicknesses, numbers of passes, and types of compaction equipment (i.e., different vibratory rollers) are investigated (paragraph 3-1*k*). Many test fills have been constructed by the Corps of Engineers and other agencies, and the results should be reviewed for possible applicability before constructing test fills. Rock-fill should not be placed in layers thicker than 24 in. unless the results of test fills show that adequate compaction can be obtained using thicker lifts. As the maximum particle size of rockfill decreases, the lift thickness should be decreased. In no case should the maximum particle size exceed 0.9 of the lift thickness. Smooth-wheeled vibratory rollers having static weights of 10 to 15 tons are effective in achieving high densities for hard durable rock if the speed, cycles per minute, amplitude, and number of passes are correct. Quarry-run rock having an excess of fines can be passed over a grizzly, and the fines placed next to the core. Fine rock zones should be placed in 12- to 18-in. lift thicknesses.

(2) There is no need to scarify the surfaces of compacted lifts of hard rock-fill. Soft rocks, such as some sandstones and shales, often break down to fine materials on the surface of the lift. Other sandstones may be compacted in the same manner as other hard rocks. Scarifying has been used on soft sandstone layers to move fines down into the fill. If breaking down of the upper part of the layer cannot be prevented, it may be necessary to use very thin lifts to break the sandstone so that the larger particles are surrounded with sand. Ten-ton vibratory rollers and tracked equipment break the rock more than rubber-tired equipment. If soft material breaks down uniformly, vibratory or other equipment can be used, but the dam should be designed as an earth dam. Specifications should prohibit the practice often used by contractors of placing a cover of fine quarry waste on completed lifts of larger rock to facilitate hauling and to reduce tire wear. If such a cover of fines were extensive, it could have a detrimental effect on drainage and strength characteristics of the outer rock zones.

7-9. Slope Protection

Adequate slope protection must be provided for all earth and rock-fill dams to protect against wind and wave erosion, weathering, ice damage, and potential damage from floating debris. Methods of protecting slopes include dumped riprap, precast and cast-in-place concrete pavements, soil cement, bituminous soil stabilization, sodding, and planting. The type of protection provided is governed by available materials and economics. Slope protection should be designed in accordance with the procedures presented in Appendix C. Due to the high cost, the initial slope protection design should be accomplished during the survey studies to establish a reliable cost estimate. The final design should be presented in the appropriate feature design memorandum.