GROUND ENGINEER'S REFERENCE BOOK, Edited by F.G. Bell Butterworths,
ISBN 0-408-01173-4. (1987)

44 Embankments

V F B de Mello
Consultant, São Paulo, Brazil

Contents

44.1 Functions of embankments. Embankment dams

An embankment is a trapezoidal section of fill made of earth materials for establishing a crest of moderate width and grade higher than the surrounding terrain, and is contained by stable side slopes. It performs certain engineering functions. Common functions of embankments include raising highway and railroad grades above levels of a certain statistical recurrence of flooding, and so also for approaches to bridges, viaducts, flyovers, etc. In all such cases the design engineer estimates the maximum height up to which the use of an embankment is, technically and economically, the most desirable solution, being thereafter supplanted by structural bridging.

An embankment dam is a fill as described above that, besides serving the functions required of all embankments, impounds water on one side, sustaining a differential hydraulic head between upstream and downstream. Thus embankment dams must dissipate this differential head under optimized conditions of stability and of seepage.

All embankments have common problems and solutions, but embankment dams face in addition the recognized worst source of problems in soil engineering, namely, the effects of water, through pore water pressures and seepage gradients. Therefore it is proposed to discuss embankment dams in this chapter which will also cover the basic points for other embankments.

44.2 Types of embankment dam

Embankments and embankment dams date back as far as recorded history. Early civilizations developed adjacent to water and there was frequent use of homogeneous fills on swampy and flooded flat ground. Embankments were built with local materials, transported, spread, and often lightly compacted by manpower or by animals. Alternatively, materials were hydraulically transported and sedimented, generating the embankment mounds with lateral beach slopes. Either of these primitive embankment types may be visualized as having evolved from a schematically homogeneous mound.

In more recent times a distinct general trend is attributed to the intensification of gold-mining in the gravel beds of temporary rivers. Two trends developed; on the one hand the availability of sufficient impervious soil, and construction time unhindered by hydrology, favoured the use of the central earthcore dam with gravel shells. Symmetrical cross-sections prevailed in such early zoned embankment dams. At the other extreme, that of hydrological hindrances and lack of impervious soils, a parallel type of dam developed wherein a coarse pervious non-erodible embankment provided the stable geometry, and a thin non-erodible upstream impervious facing impounded the water.

For an understanding of the principal types of embankment dam cross-sections it is helpful to visualize an idealized more or less historical development which excludes foundation problems by assuming a firm, impervious, homogeneous foundation. Conditions affected by foundation problems must be faced quite separately because of the strong dominance of geology, and the fact that no two cases are sufficiently alike. One might thus start from three conceptually different basic types, as summarized in *Table 44.1* and illustrated in *Figure 44.1*.

Recommendations for the design of embankment dams must take account of, firstly, the selection of the physical model judged most preferable and, secondly, the dimensions derived by geotechnical computation. Obviously there is a continual interaction between the two. The distinction is emphasized, however, because basic physical principles can be of much greater consequence than a subsequent computational adjustment based on nominally accepted practices.

44.2.1 Reservations regarding visual principles of imitative designs

A critical scrutiny reveals that preliminary designs of embankment dams have been primarily influenced by visual imitations of cross-sections of existing dams in lieu of assumed physical principles of behaviour. Two principal components have been symmetry and geometrical similitude. So many dams have used symmetrical sections, with acceptable behaviour, that often such sections are taken as establishing a design precedent (see de Mello[1]). However, it must be emphasized that both symmetry and geometrical similitude as bases for precedent are devoid of sense. The first must be rejected since the functions required of an embankment are too blatantly asymmetrical. The second retains some acceptability under the proviso of small incremental variations from one case to the next. Nonetheless, geometrical similitude implies linear variations of behaviour and degrees of acceptance, which seldom, if ever, represent reality.

44.2.2 Idealized interpretation of basic principles behind design evolution

The homogeneous earth mound impounding water was progressively made to incorporate important details, mostly because of lessons drawn from failures. A dam with a highly impounded voluminous reservoir should be guaranteed against failure. There should not be a risk probability, however small, computationally attached or attachable to such catastrophic events. Such singular conditions, classed below as type (a) cases, can only be associated with so-called 'statistics of extreme values', for which calculations, even probabilistic, are illusory. The solution demands a change of physical model. Other failure and misbehaviour conditions are sufficiently repetitive, type (b) cases, to permit establishing 'laws', thus inducing theorization for calculations using statistics and probabilities of averages, within a constant physical model.

(a) Design changes dictated by exceptional failures (extremes)

Absolute priority is assigned to this group, wherein the designer must avert eventual failure by 'choice of change of statistical universe', that is, by a physical change of section and/or materials.

Exaggerated examples help to elucidate the concept. For instance, in constructing an embankment with clean compacted

Table 44.1 Types of embankment dam cross-section

1. Homogeneous section, semi-pervious to impervious { spread, and compacted / hydraulic fill	Symmetrical at start	Preliminary designs mostly by visual imitations, that is, by geometrical similitude
2. Zoned earthcore and rockfill-shell section, placed and compacted	Symmetrical at start	
3. Upstream impervious deck, on gravel or rockfill embankment section		

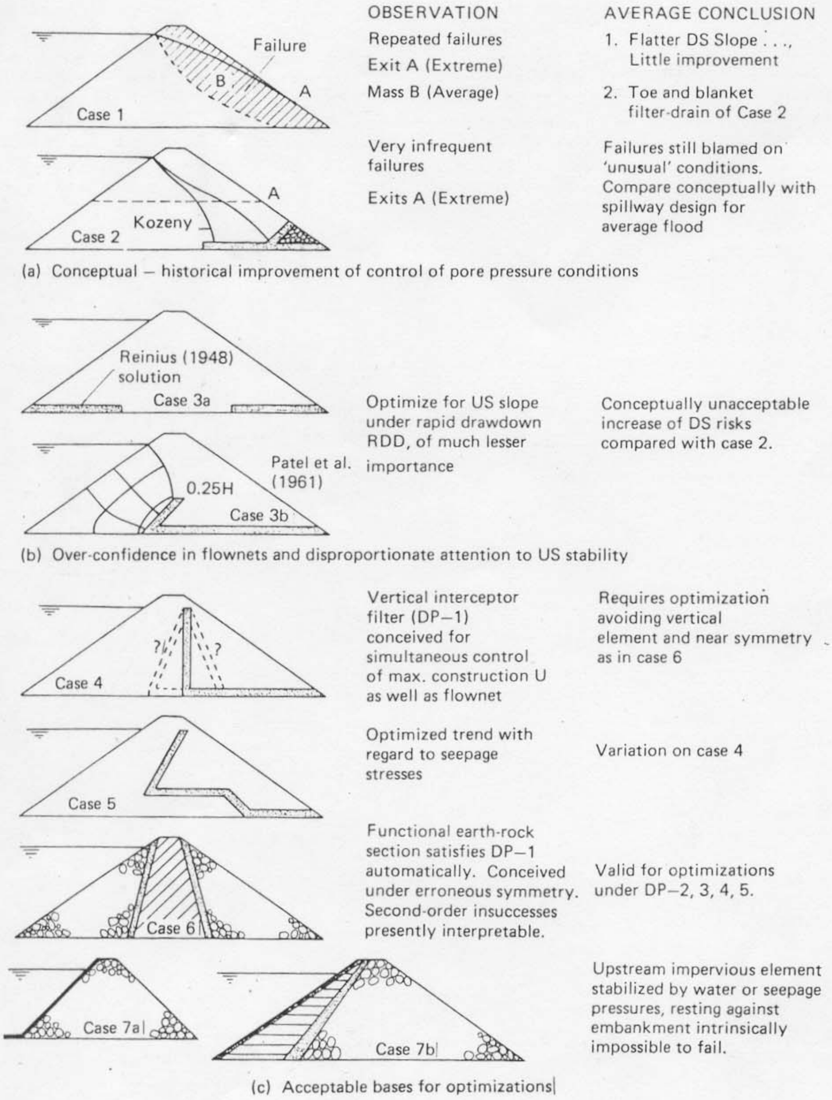


Figure 44.1 Rationale of conceptual evolution of types of embankment dam and cross-sections

rockfill any problem of construction-period compressibility pore water pressures (Sections 44.3.1 and 44.6.3) and corresponding slope instability is excluded. Therefore, one may claim, because of the physical change of material, that the mathematical probability risk of construction-period instability of clean rockfill due to pore water pressures is truly and deterministically zero (see de Mello[2]).

The really serious concerns regarding embankment dams are associated with stability under extreme conditions of reservoir operation, and these may be subdivided into:

(i) mass instability problems, of both slopes, type (b) cases;
(ii) eventual problems of internal erosion (piping), transverse tensile cracking, and other situations of extreme-value statistics, type (a) cases.

Any singular condition of failure, such as that of tensile cracking or internal erosion (piping), which, if undetected, leads to overall ruin, requires a judicious change of cross-section to preclude the development of such conditions. For example, a change in the cross-section of a dam which would guarantee behaviour under compressive stresses, precluding tension, represents an acceptable solution. Such basic design choices of physical models and materials obviously take precedence over optimization by computation.

(b) Design optimizations promoted by theorized computations

Within the second category of design developments, derived from failures and misbehaviour, fall all the adjustments employing present computations on seepage flownets, mass sliding instability of slopes and corresponding computed factors of safety, compressibility settlements and other deformation estimates, and so forth. It is emphasized that the priority design preferences, presently often incalculable, represent the starting point not of design but of design optimization.

The improved control of pore water pressures and exiting seepage stresses through filter-drains is interpreted as having been principally predicated as in (a) above, for averting piping. But under average conditions the accompanying benefits to mass slope stability are automatic. Any impervious or drainage zone automatically affects both types (a) and (b) of instability. Widely different conceptual effectiveness is attached to each, but the very fact that type (b) conditions are amenable to the geotechnical computation rather than the art of embankment dam engineering, leads to a disproportionate occurrence of the latter in publications.

Computational methods and results on such mass geotechnical problems have contributed much to the evolution of design. Seismic (dynamic) instability analyses have become an additional important consideration but have not yet clearly contributed to the incorporation of specific optimization features in the more commonly used design sections.

44.2.3 Schematic rationale of a conceptual evolution of three basic design sections

The three basic sections, namely, the homogeneous embankment section (with filter-drainage features), the central core earth-rock section, and the upstream impervious deck rockfill section, serve as a basis for discussing conceptual design developments and help elucidate the comparative importance of extreme versus average conditions.

Some points illustrated in *Figure 44.1* call for additional comment on principles. For case 1 (and similar) if there is adequate time for downstream treatment under slow reservoir filling, the most effective and economical way of controlling seepages as they appear is to apply localized filter-drains. However, a good remedy is not necessarily a good preventive measure. Design should resemble a fully effective vaccination: one seeks protection against any trouble developing at an inopportune moment and/or in more virulent conditions.

The use of seepage flow nets led to a series of design suggestions (case 2, 3a, 3b) which are questionable as intrinsically undesirable, even though good behaviour has been obtained in many cases. The intrinsic failing arises from the disregard of extreme conditions (infrequent, but not precluded) whereby a single path of preferential seepage may cross the dam with negligible head loss and consequent high risk of piping failure. In short, the design should require some fully intercepting (inclined or subvertical) filter-drainage, such as introduced in cases 4 and 5, and such as existed in principle (merely requiring detail optimization) in cases 6 and 7.

The zoned earth-rock sections of case 6 and upstream deck sections of cases 7a, 7b, basically optimized the use of very stable, fully drained rockfill (or gravels, etc.) for the zones designed to guarantee mass stability, and separated the zone required to provide imperviousness. The important problems thus are shifted to providing adequate transitioning between such highly differentiated zones and materials, such transitioning being compatible with filter-drainage and deformations. Once again, what has been dominant in design of these intrinsically well-conceived sections, is the attention to the extreme-value problems of cracking and of potential piping.

The upstream-deck dams using non-erodible impervious membranes (for example, lightly reinforced concrete slab) will, in principle, be recognized as free from risk of 'failure'. The problems of upstream-deck rockfill dams thus has shifted to the area of minimizing 'misbehaviour' of higher water losses, due either to imperfect sealing of foundation or slab joints, or to the cracking of slabs because of incompatibility of slab flexibility with deformation of its supporting embankment.

Failures and the analyses of failure modes have furnished the principal evidence for development of optimized sections and corresponding materials. Thereupon, within chosen sections, methods of testing and computation, and somewhat arbitrary factors of safety, and acceptance criteria, have been established on the basis of precedents, and intuitive revisions thereof. Embankment dams may be built of almost any available disintegrated materials, even organic and sanitary fills, and depending on the material in question may be executed by many different construction procedures, provided that each gives the necessary guarantee of safety and adequate behaviour.

44.2.4 Checklist of basic design principles

A checklist of design principles to be used is shown in *Figure 44.2*. It provides a framework both for analysis of case histories and for sketching of new preliminary design proposals (see de Mello[1]).

Two basic points are emphasized. Firstly, a *sine qua non* Design Principle 1 (DP1) is proposed, to exclude extreme value conditions by choice of appropriate physical model (statistical universe). Secondly, in the subsequent use of idealizations and computations for design optimizations one must compensate by emphasizing the use of statistical variabilities within the computation routines.

All repetitive behaviour is subject to dispersion, establishing a histogram of a continuum that is only acceptably quantifiable within the band of mean plus or minus standard deviations. Therefore design adjustments, here discussed under four additional Design Principles (DP2 through DP5, *Figure 44.2*), must satisfactorily cover foreseeable problems to be deduced from parametric variations of consequence in the computations.

Design Principle 2 (DP2) is explained by some common examples. The basic desire is to use an overriding conditioning element so as to make the engineering behaviour essentially independent of the statistical variabilities. If a subsoil presents questionable conditions for footing foundations, the engineering solution will be to cut across the region of uncertainty of the upper strata. In similar manner, strong preference is given to a chimney filter-drainage interceptor over the full height of the dam not only for the physical exclusion of piping (extreme-value) under DP1, but also for subsequent optimizations according to DP2.

Another important Design Principle, DP3, for selecting the best physical design, recommends that the total contribution of the medium traversed be drawn in. This principle seeks to ensure the behaviour of averages, with resultant homogeneity, prudence and economy. In the example of piled foundations, preference would fall on pile types that foster frictional resistance, to complement point resistance. In the case of pervious foundations of dams the principle suggests lengthening the seepage path to the utmost, without encroaching on other design aims. Whenever a larger mass is brought into play, the risks of the behaviour being controlled by a localized unfavourable condition are smaller.

Design Principle 4 (DP4) emphasizes that preloading or prestressing under controlled conditions, subject to adequate observation, is an ideal design solution. The proportion of failures in geotechnical structures increases significantly as the ratio of incremental to existing stress increases, and as the time rates of such changes become higher. For instance, in foundations the worst failures have occurred with silos, tanks, etc., where the ratio of live load, rapidly applied, to structural dead load, is very high. Thus a useful design expedient favours prestressing as much as possible, under slow controlled conditions, in comparison with stresses to be faced under future uncontrolled critical conditions.

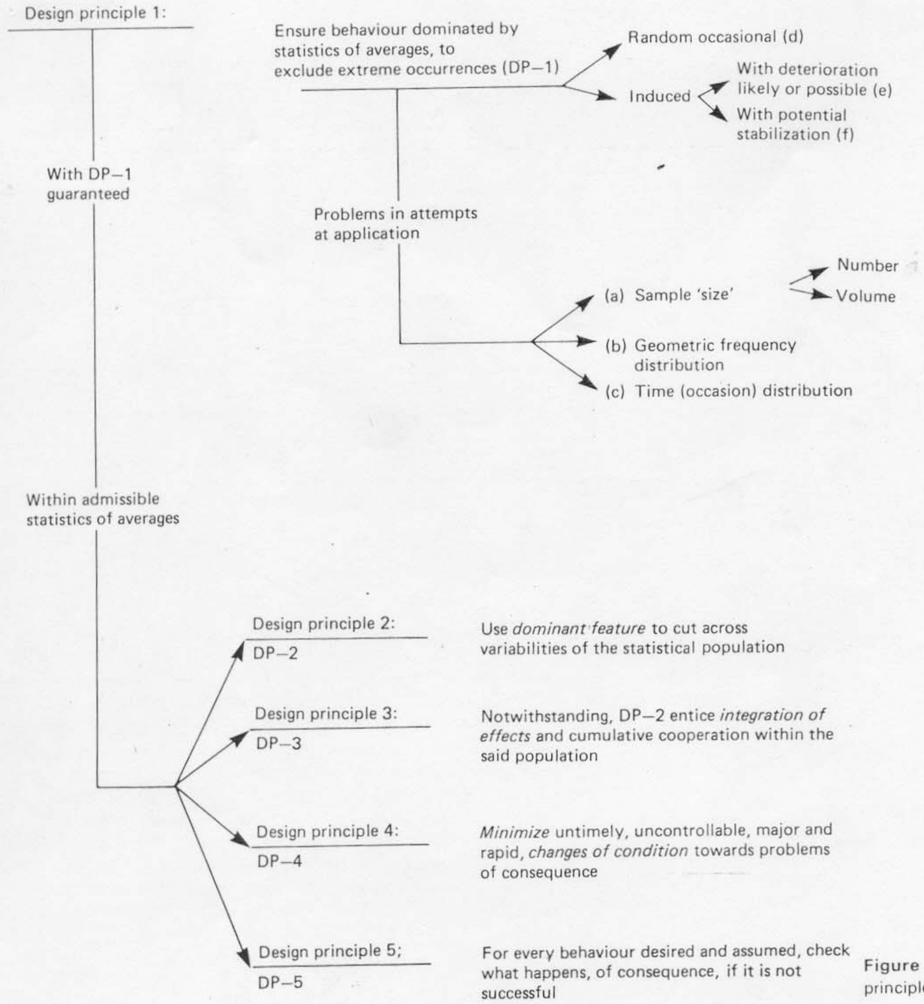


Figure 44.2 Checklist of design principles (de Mello[1])

Finally, in Design Principle 5 (DP5) engineers are reminded that it is mostly on the adequacy of the hypotheses, and not on refinements of computations, that the safety and satisfactory performance of a dam are critically dependent. Every design decision, and especially every computation, presupposes some functional hypotheses. In the face of each assumption and/or prediction, one should question and check what would happen, and what could still be done to accommodate surprises, if results turn out significantly worse than anticipated.

Hypotheses and parameters initially taken at judicious average values should be rechecked under variabilities dictated by statistical probabilities. For instance, what would happen to an assumed flow net due to full reservoir if the filter-drainage system proved insufficient or clogged? What corrective treatment could be used, with what rapidity and efficiency, to return the pore water pressures and embankment stability to the desired levels? Prudent design treats with greatest respect the parametric variations that provoke the greatest rate of change of effect with rate of change of variation of hypothesis.

In summary, experienced design starts from the choice of adequate physical model, free from unquantifiable damage phenomena of extreme-value statistics, and then uses judicious design computations, with respective parametric variations. The resulting computed variations would establish a continuum of the histogram of predicted behaviour. As shown in *Figure 44.3*, the decision of design is a Yes–No (accept–reject) discontinuity applied to truncate this predicted continuum on the basis of acceptance criteria derived from previous projects. Thus, the ultimate need for design computations is to establish acceptance criteria within the prefailure 'small-misbehaviour range' that permits collecting sufficient repetitive data.

Priority attention to catastrophic failures has understandably contributed to advances under DP1. But the comparative indifference to varying degrees of small-misbehaviour histograms of cause–effect, and the corresponding acceptance criteria, has been the deterrent to progress in the link between rapidly advancing theory and computations, and to the confident exercise of professional activity of design and

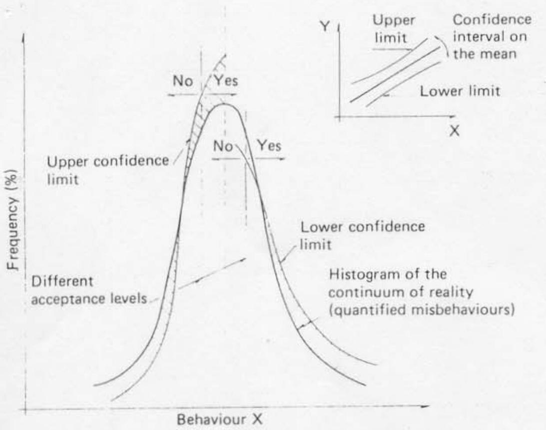


Figure 44.3 Histogram of the continuum of reality, and discontinuity of yes–no design decision of engineering (de Mello[1])

construction, assisted by Design Principles 2 through 5.

44.3 Principal modes of failure and misbehaviour, and behaviour parameters

An embankment dam is a very complex structure obliged to synthesize information from many areas (see Thomas[3]; Salas[4]; Sherard[5]).

(*a*) *Hydrology*. It is well established that hydrological factors have been responsible for the greatest number of failures, due to insufficient spillway capacity, or misoperation of critical hydraulic features. An important point to be made for embankment dams is that a so-called probable maximum flood (PMF), which has no probability attached to it but may be considered as a 'once-never flood' to be used for design, cannot be taken as a fixed value for a site, but requires adjustment. Some factors imposing such adjustment include rate of rise of reservoir water level with incident PMF; proportion of PMF flood volume (not merely peak discharges) to volume of reservoir surcharge; guarantee and rate of control of operative gates; and consequences to downstream upon discharge of the PMF or variations thereof, controlled or uncontrolled.

(*b*) *Geology*. The geology, both of the reservoir, and of the site, is recognized as responsible for a great number of cases of misbehaviour and of total failure. Rivers are the geomorphological expression of weaker geological zones which have been preferentially attacked, and sites selected for dam construction tend to have topographical and geological conditions that singled them out. Moreover, the higher the dam the greater tend to be the problems, and therefore the foundation requirements are more demanding. One simple design-construction dictum is that the functional competence of foundation zones should measure up to being equivalent to or better than that of the immediately overlying embankment zone. The abutments call for special attention. The point to emphasize is the immensely different degrees of confidence in comparing a material constructed as designed and inspected, and the underlying geological material, accepted under simplified hypotheses and mere spot investigations, however sophisticated, and therefore much more subject to erratic and unforeseeable local variations.

The principal facets of misbehaviour or failure facing the geotechnical engineer include mass instability of slopes, unfavourable seepage effects, and differential deformation detrimental to the behaviour of the embankment.

44.3.1 Construction period stability

There have been many cases of slope instability during construction, especially in cases of compressible impervious materials placed under near-saturated conditions in which the added weight of fill generates significant pore water pressures. The more rapid the rise of the fill, the worse is the problem. The slope stability computation is routine, and is handled by two alternative procedures as regards stresses and strength envelopes. It is recommended that both analyses be used. One is based on total stresses and the apparent strength envelope of quick, Q, or unconsolidated-undrained, UU, triaxial tests and gives satisfactory first estimates, less subject to errors in laboratory and field pore water pressure determinations. The other, based on the effective stress strength envelope requires concomitant determinations (or, at the least, appropriate estimates) of the pore water pressures that would develop under representative 'undrained' loading conditions.

Since sliding failure can be averted with relative ease and, if it did occur, would not generally be of great consequence, it has been often proposed that a very low factor of safety, FS (even as low as about 1.1), could be accepted. Any such generalization must be denounced as wrong and dangerous, since many idealized hypotheses are hidden in the standard stability analyses and oversimplified concepts of factor of safety. Judicious analyses may well show different cases at $FS = 1.5\ (\pm 0.6)$ to have analogous behaviours. The questions of slope stability analysis and choice of appropriate factor of safety are further discussed in Section 44.6.1.

Besides the above slope instability hypothesis for embankments on competent foundations, there are cases of instability on soft foundations. The cases of embankments on soft clays reflect a special condition of foundation bearing capacity, and represent an ever-growing chapter in geotechnical engineering. The most common solutions include the use of weight berms (an efficient way of achieving the effect akin to a flatter overall slope) or the use of various ground improvement techniques such as vertical drains for accelerating clay consolidation, compaction techniques for loose sands (vibroflotation, compaction piles, deep dynamic compaction, etc.), and so forth (*Figure 44.4*).

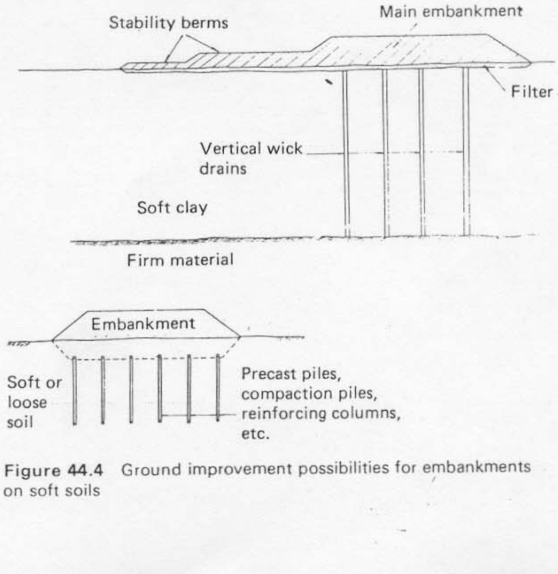


Figure 44.4 Ground improvement possibilities for embankments on soft soils

44.3.2 Sliding instability of the downstream slope (DS) with reservoir filled

Such failures have occurred much less frequently after the principles of flow nets and of slope stability analyses by effective stresses became more widely used. This type of catastrophic failure must be avoided. Since greatly varying hypotheses regarding both pore water pressures and nominal strength equations still prevail, one concludes that the exclusion of risk of such failures may not be guaranteed merely through improved design computations. The author emphasizes a design preference for sections and materials that would exclude perceptible and rapid changes of pore water pressures affecting stability of critical downstream zones, in changing from end-of-construction to full reservoir conditions. In such situations fully intercepting filter-drains are used, and great confidence is attached to calculations of incremental stability (ΔFS referred to the proven end-of-construction situation).

44.3.3 Upstream slope (US) sliding under rapid drawdown (RDD)

Lowering of reservoir level is critical for upstream slope stability analysis. The destabilizing action due to the rapid removal of the confining reservoir water pressure comes about because there is no time for an equivalent relief of pore water pressures within the soil mass. Once again, the necessary parameters and hypotheses concern seepage pore water pressures, transient pore water pressure increments due to compressibility, and appropriate strength equations.

Once again, analyses prove more meaningful when conducted via incremental factors of safety, ΔFS, on proven prior conditions of the slope, at end-of-construction, and under full reservoir. Damage is far from catastrophic, as failure would occur immediately after major depletion; thus lower computed factor of safety values are accepted.

44.3.4 Seepage through embankment and foundation

Inadequate control of seepage through the embankment and its foundation has been one of the principal sources of failure and of different degrees of misbehaviour. Indeed, it continues to be the principal source of troubles, in some cases catastrophic (as was the failure of the Teton Dam, USA, 1976). To the ordinary observer the simplest evidence of some degree of malfunction is the occurrence of perceptible flows appearing downstream. In fact, however, high uplift pressures, often associated with small exiting flow, are more dangerous, and cases of high seepage flows may not, of themselves, constitute a danger to the dam if their exit conditions, from embankment or foundation, are controlled.

High leakages obviously reflect poor upstream sealing, but when considered excessive downstream must be judged quite distinctly, either as affecting safety, or with regard to the economics of water loss. Seepage control for safety must relieve uplift pressures, despite increased water loss by the facilitated drainage, and must also limit seepage stresses within the mass, and principally at flow exits (exit gradients). Internal erosion and piping have been the principal cause of failures, leading to the development of filter-drainage designs as the single most effective measure for embankment dam safety. Piping is well evidenced as the erosion that starts at an exit point and works retrogressively along the flow path in an accelerating fashion. Internal erosion, redistributing fines from one zone into adjacent bigger voids, follows similar physical laws of behaviour, though more difficult to analyse, and often remains undetected until its bigger cumulative effect surfaces as upstream sinkholes.

As shown in *Figure 44.5*, soil mechanics and dam engineering received a big impulse towards solution of seepage problems

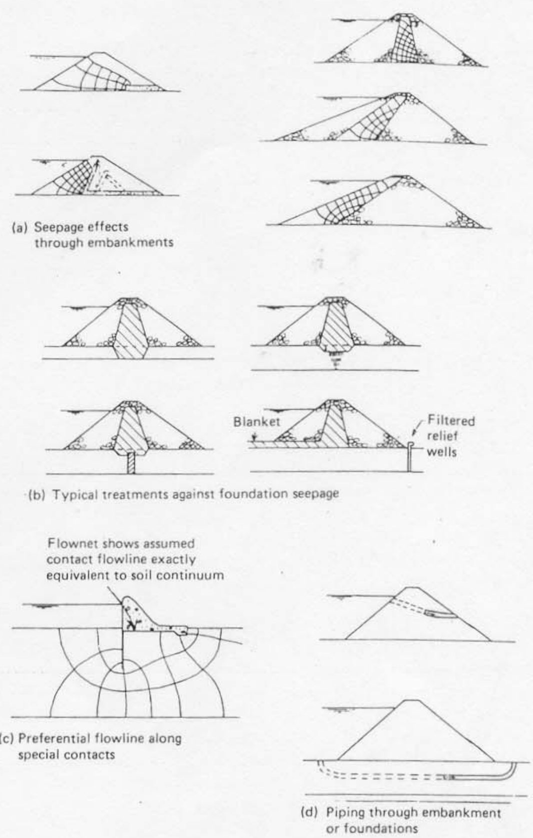


Figure 44.5 Schematic indication of seepage effects through embankments and their foundations, and typical problems of preferential path and of piping

through the use of flow nets. It must be emphasized, however, that the reliance on idealized flow nets may have gone to an undesirable extreme, and that some adjustments are often necessary, to correct for oversimplified hypotheses.

One such exception preceded the introduction of flow net analysis for design, and merits renewed consideration. One refers to the empirical recommendations on 'creep ratios' for the preferential boundary flowline of contact between a rigid concrete block (*Figure 44.5c*) and the foundation. Preferential flowlines (especially when they establish boundary conditions for flow nets), heterogeneous subsoil conditions, and the very changes wrought on pore sizes of soils by the seepage effective stresses, are among the important factors that require adjustments when basing design computation on routine simplified flow nets.

In conclusion, the principal parameters to be considered with regard to the design of embankments against seepage problems are:

1. Coefficients of permeability, and anisotropy of the 'impervious' zones, in order to establish probable flow nets.
2. Full consideration of extreme variations of flow net conditions around average hypotheses.
3. Careful attention to comparative filtering qualities and drainage capabilities of the materials into which the flow exits from the adjacent impervious section.

4. Careful consideration of erratically critical combinations of conditions (2) and (3), because eventual development of piping belongs to the phenomena of extreme-value statistics.
5. Judicious appreciation of changes that might occur, either gradual or abrupt, due to the very action of seepage or other phenomena, during the long operative life of the impoundment.

44.3.5 Deformations and consequences

In the early days of design and construction of embankment dams it was automatically assumed that they were sufficiently flexible and plastic to adjust to foundation deformation without cracking. Thus the routine recommendation was to resort to embankment dams whenever foundation conditions were expected to lead to moderate total and differential settlements. However, several factors that made this assumption valid initially later suffered gradual changes either invalidating, or at least qualifying, it. Thus compressibilities, both absolute and relative, of foundation and embankment materials have called for special considerations over the past forty years because of the number of failures and misbehaviour associated with cracking due to incompatible deformation.

There has been a gradual recognition of the importance of settlement. In the early days compaction was not heavy and construction often involved well-moistened soils. If the dams were of hydraulic fill, or employed 'puddled cores' (British technique) their plastic deformability was quite assured during construction and for a long initial period of operation before thixotropic brittleness set in. Moreover, early dams were smaller, and generally selected favourable foundations, avoiding strata of questionable supporting capacity.

The development of heavy earthmoving and compaction equipment, and the concern (somewhat misplaced) with regard to construction pore water pressures, coincided, in the 1940s and 1950s with the promotion of denser compaction on the dry side of standard compaction optimum water contents. As a consequence compacted clayey fills became far more rigid than had been suspected. Simultaneously there were rapid advances in the heights of dams constructed, and in the acceptance of less favourable foundation conditions. A notable increase occurred in the incidence of visible cracking. This highlighted the problem of differential settlements that generate cracking. The differences between the types of cracks that endanger dams (transverse tension cracks near the tops of dams, and tendencies to hydraulic fracturing of cores), and innocuous longitudinal cracks and shear displacement planes at depth which are generally tight, has to be emphasized.

In summary, parameters of significance for embankment dam design include compressibility both of foundation and of the compacted embankment itself. Such behaviour must be modelled carefully to permit computing the increments of differential deformation that would generate transverse tensile cracking across the top 15–20 m of the impervious core of a dam, especially if such tendencies to tensile stressing are rapid and do not permit the clayey cores to swell or heal.

Significant differential settlement, both in longitudinal and transverse direction, may occur within high dams. Such deformation can give rise to cracking (*Figure 44.6*). The important transverse tensile cracks are related to differential settlements along the longitudinal direction. *Figure 44.6c* indicates that critical conditions for tensile cracking, as a function of abutment inclination, tend to occur on slopes of intermediate steepness because along much steeper slopes the tendency is for shear displacement to tighten the contact along the sheared face (shear remoulding generally decreases permeability). Other critical conditions arise from abrupt changes of abutment slope, notably where it is concave downward.

44.4 Construction materials and methods. Anticipated geotechnical behaviour

44.4.1 Types of materials and construction methods

Embankment dams have successfully employed the widest imaginable range of materials, and it is likely that this range will be further widened. Wherever possible local materials are used, especially those derived from the necessary excavation on site. The design of the dams and construction specifications are adjusted accordingly, rather than seeking specific borrow materials from greater distances. Such a trend, no doubt, will increase as the volumes handled, and hauling costs increase.

Moreover, testing and experience widen the range of materials of acceptable behaviour, even if restricted to special zones within the cross-section. For instance, soft rocks, and even those subject to some dissolution, may be used in a central random zone (*Figure 44.7*) downstream of a fully intercepting filter drain, that excludes contact with water. Similarly some basalts that contain nontronite and disintegrate within a few days, have been successfully used when protected by a minimum thickness of about 3–5 m of durable rock.

Construction methods in the past included end-dumped rockfills and hydraulic fills. Both methods were successful when used up to heights of the order of 100 m. Dumped rockfills (using clean sound angular rock) have been virtually abandoned in the past twenty-five years, except for construction of cofferdams for river closure. In such conditions end-dumping is used from a platform only slightly higher than the water level, and the resultant fills have been satisfactorily used up to heights of more than 50 m. Any large deformation does not impair the self-healing behaviour of the suitably graded upstream impervious soil (as shown in case 7b, *Figure 44.1*) to adjust to deformation and in fact to be improved by seepage stresses.

Hydraulic fills involve hydraulic transport of soil–water mixtures and deposition directly where desired. Originally, broadly graded clayey-sands were used and selective deposition of core and shells took place (generally called the American method). Variations of such techniques are used as the basis for construction of tailings dams, often resorting to cyclones to separate the more pervious and stable sandy material from the slimes.

Such tailings dams (*Figure 44.8*) are being used to heights of 200 m, comparable with those of the highest compacted fill dams. But some risk of catastrophic failure has to be recognized under dynamic loading by earthquakes. This is due to the low density of the hydraulically deposited sands. As far as conventional dams are concerned, hydraulic fill construction continues to be used with success in major dams in Russia, where local conditions have led to the use of what are termed essentially 'homogeneous' fills of fine sands (*Figure 44.8*).

No further mention will be made of dumped rockfills or hydraulic fills, attention being concentrated on compacted materials. In accordance with the dominant functions of the zones within a compacted earth–rock section, the materials will be separately considered as:

(i) Core materials.
(ii) Shell materials of rocks or gravels.
(iii) Sand and fine gravel-size filters and transitions.
(iv) Well-graded materials that are presently much debated regarding conditions of use and acceptability criteria, thus meriting special consideration.

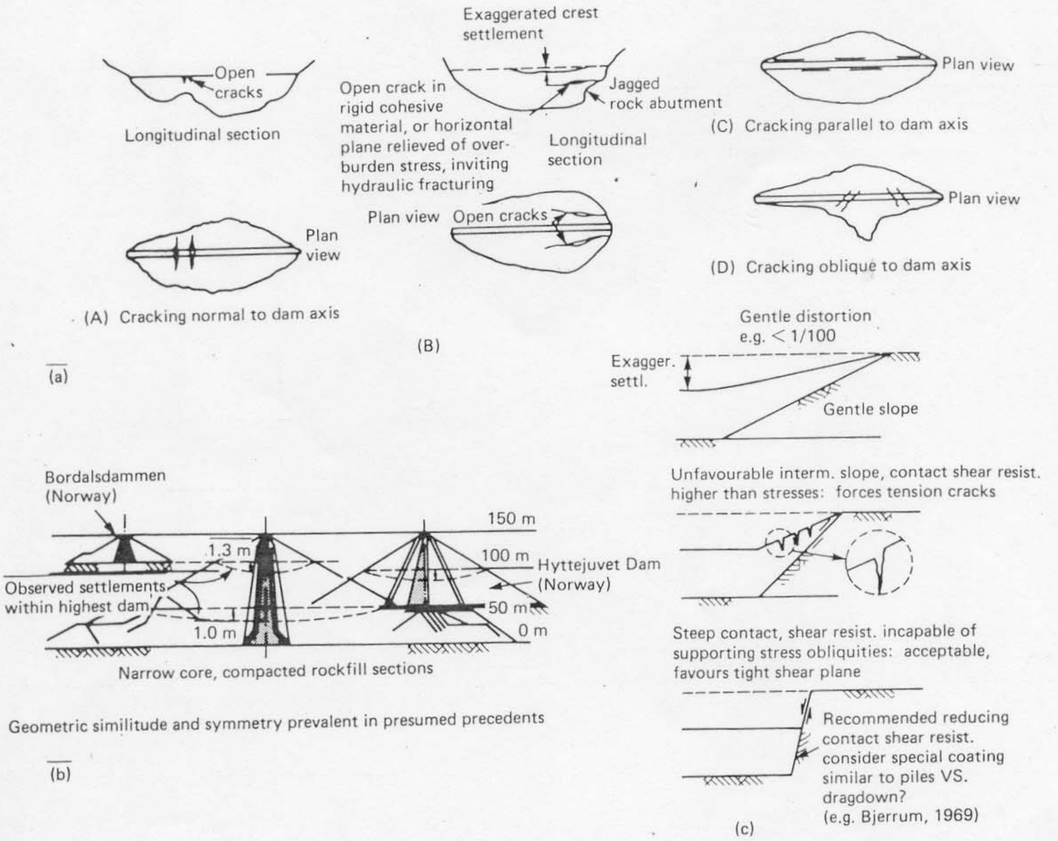


Figure 44.6 Importance of differential deformations for transverse tensile cracking

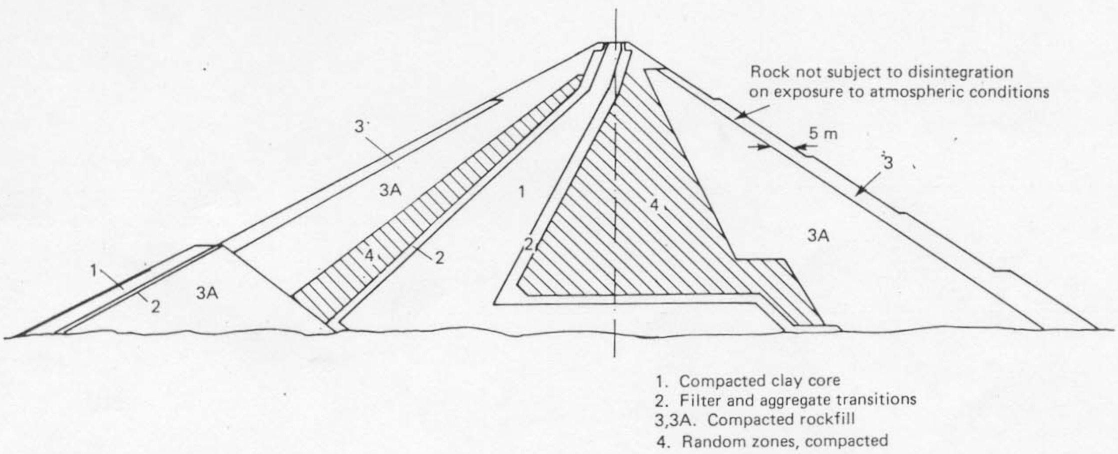


Figure 44.7 Typical detailed cross-section of zoned earth-rock dam indicating zones of selective materials (see *Table 44.2*)

44.4.2 Core materials

The main function of the core is to be sufficiently impervious. Minimum core widths comfortably accepted according to experience tend to be of the order of $0.25H$ to $0.5H$, where H is the hydraulic head. *Table 44.2* indicates the core widths employed in some important projects in the world, under conditions recognized to be unfavourable to narrow cores, which include the vertical core, high dams, and seismic regions.

Seepage losses across cores are computed from flow nets, and are quite small when permeability coefficients are lower than 10^{-7} m/s. With core widths varying between $0.25H$ and about

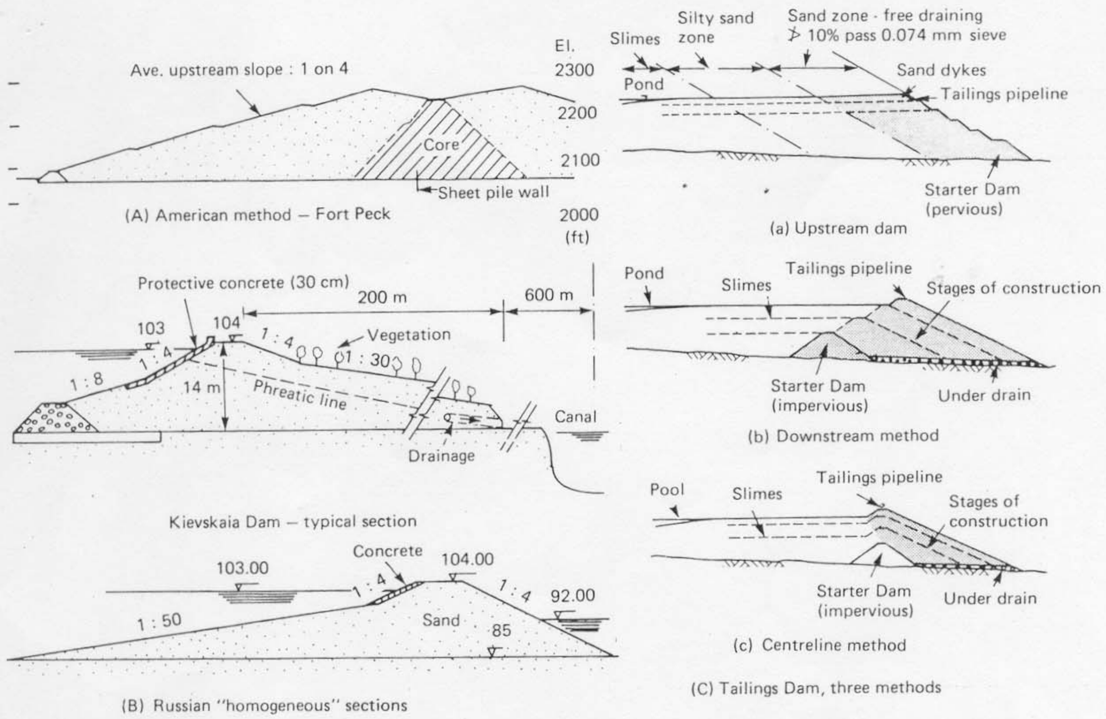


Figure 44.8 Various conditions of use of hydraulic fills

Table 44.2 Minimum core widths successfully used in difficult conditions (see *Figure 44.7*)

Dam	*Country*	*Height*	*Core-width*	*Year*
Oroville	USA	235 m	0.3 H	1968
El Infernillo	Mexico	149 m	0.2 H	1963
La Angostura	Mexico	144 m	0.3 H	1974
Misakubo	Japan	105 m	0.2 H	1969
Gepatsch	Austria	153 m	0.30 H	1966
Netzahualcoyotl	Mexico	138 m	0.4 H	1964
Goschenen	Switzerland	155 m	0.3 H	1960
Messaure	Sweden	101 m	0.35 H	1963
Belmeken	Bulgaria	94 m	0.25 H	
Geehi	Australia	90 m	0.4 H	1966
Outardes	Canada	122 m	0.3 H	1968
Vicente Guerreiro	Mexico	96 m	0.4 H	1968
Basilio Badillo	Mexico	93 m	0.4 H	1973
Miho	Japan	95 m	0.45 H	
Chicoasen	Mexico	200 m	0.40 H	1980
Srinagarind	Thailand	140 m	0.35 H	1977
Frauenau	Germany	86 m	0.3 H	
Sarsang Skaya	USSR	125 m	0.4 H	
Binga	Philippines	103 m	0.25 H	
Kenney	Canada	97 m	0.2 H	
Kajakai	Afghanistan	100 m	0.4 H	
Brownlee	USA	122 m	0.2 H	1958
Seitevare	Sweden	105 m	0.30 H	1968

$1.2H$ (maximum desirable, for avoiding significant interference of the core shear strength on the stability of the upstream and downstream slopes of the dam) seepage gradients and losses vary roughly in inverse proportion to the width. Therefore this design dimension is of secondary importance for water losses. The principal contributions to higher seepage losses do not occur across a well-constructed core, but across the foundation (most frequently through the abutments), or are caused by crucial secondary defects that impair the behaviour of the core.

Traditionally, low permeabilities were automatically associated with fine grained materials and with clayey soils. However, theoretical reasoning and experimentation have demonstrated that the conventional procedures of predicting permeabilities from routine identification tests (grain size and plasticity characteristics of the soil) are too indirect and often lead to surprises. What really matters is the pore size distribution, which can be well inferred and, if necessary, determined. Moreover, field experience with clayey borrow materials has shown that, firstly, most such materials (necessarily unsaturated so as to be compactable, and thus subject to soil suction and micro-cementation above the water table) generally occur as clusters and crumbs, which are misrepresented by the conventional tests, of fully dispersed grain size distributions and fully remoulded Atterberg limits. Secondly, depending on the type of excavation, spreading and compaction equipment used, in modern dam construction with heavy equipment, the lifts are frequently built-up of clay chunks pressed together.

Therefore, for required imperviousness a very wide range of material proves perfectly acceptable, from high plasticity residual red clays, through sandy-clays, clayey-sands, to well-graded gravel-sand-silt-clay soils (even when the clay-size particles are rock flour material rather than clay minerals).

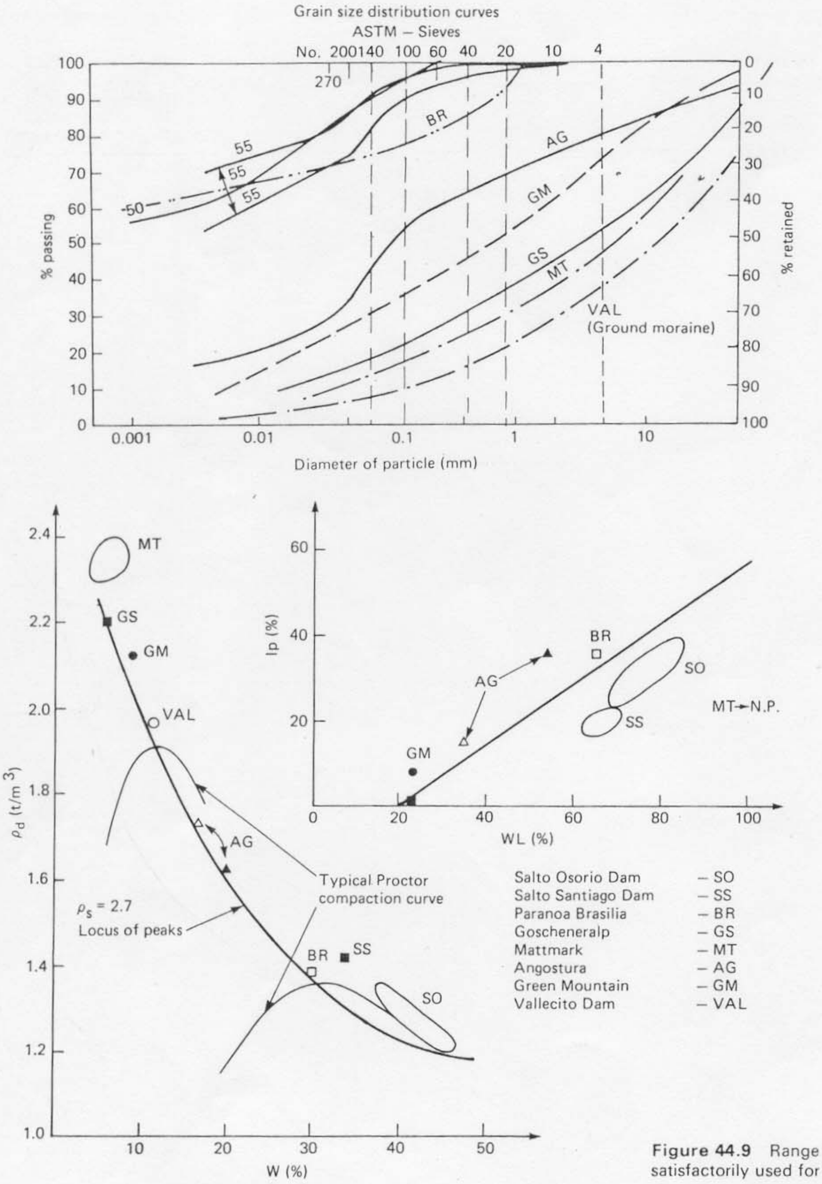


Figure 44.9 Range of identification test data of materials satisfactorily used for compacted cores

Figure 44.9 shows the range of core materials that have given satisfactory behaviour (see de Mello[6]).

The best overall identification test for compacted embankment materials is the compaction test peak point coordinates (maximum dry density and optimum water content). High dry density signifies low total volume porosity. Porosity decreases considerably in well-graded materials (Section 44.4.5) with a broad range of grain sizes. Moreover, the number of grains and of pores increases very greatly in soils wherein the representative grains are smaller. Both conditions yield exponentially smaller average permeability coefficients which relate directly to representative pore diameters and areas.

Apart from permeability, the suitability of material for narrow cores depends very much on important details of construction heterogeneities. For example, there is the systematic variability which occurs during placement and compaction. Segregations of broadly graded materials is discussed in Section 44.4.5. In the case of finer soils, the most important problem may be the compaction gradient across the lift, which leads to high anisotropy ($k_h/k_v \gg 1$), but in addition may lead to collapsible behaviour and erosion of the bottom sublayer of each lift. The latter effects are more pronounced the sandier the clayey soil, and in dry compaction compared with wet compaction (*Figure 44.10*) (see de Mello[7]).

What is more, heavy compaction on wetter near-saturated soils leads to unfavourable anisotropy with the development of subhorizontal slickensides, which are particularly serious near the top of high cores where their healing by reconsolidation is

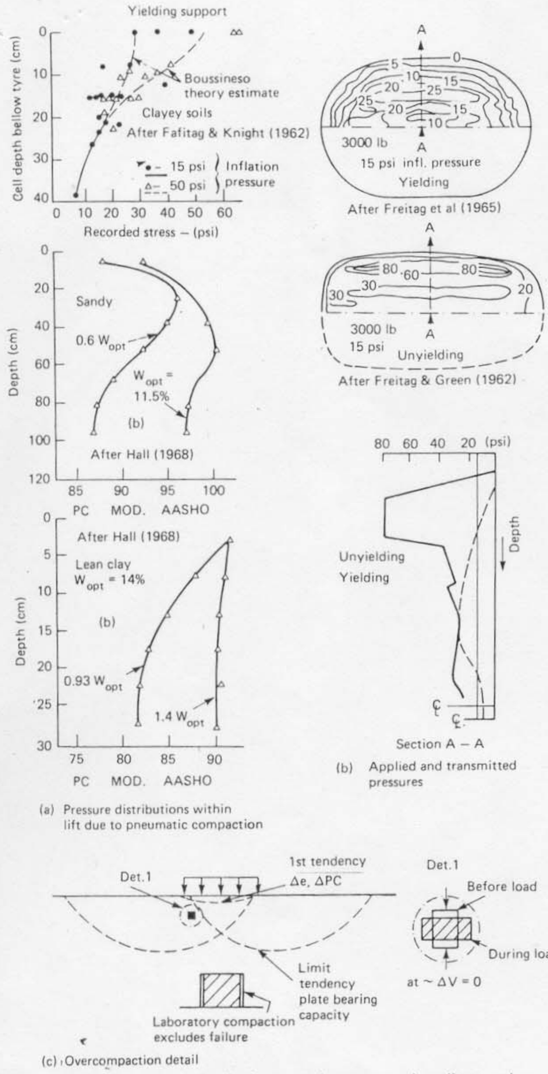


Figure 44.10 Stress distributions under pneumatic rollers and compaction gradients within lifts

precluded. They become more susceptible to hydraulic fracturing by reservoir pressure, if there is any tendency for core hang-up by less compressible contiguous filter-transitions and shells.

Experience suggests that clayey soils, say of standard Proctor optimum water content higher than about 15%, cannot absorb higher compaction energies. Therefore construction specifications referred to the Modified Proctor test should be avoided.

The second phase of behaviour of consequence involves compressibility settlement by construction period overburden loading. It is important to aim at the compacted core material being slightly less compressible than the contiguous materials, so that differential settlements may only lead to slight excess loading of the core by negative skin friction, and not the contrary (core hang-up). Thus design and specifications aim at materials and conditions that do not suffer retarded settlement, which would affect behaviour during and after reservoir filling. For minimizing core compressibility, preference falls on materials containing coarse fractions.

For the final phase of behaviour of the core, special attention is drawn to its upper part when subjected to reservoir water pressures and seepage stresses. There are risks of cracking if there has been an unfavourable redistribution of stresses, and the rate of rise of reservoir near the top is rapid (more than 150 to 300 mm per day). One view has been to minimize the tendency to cracking by use of plastic CH (that is, clay of high compressibility (plasticity)) materials, compacted in more flexible, plastic conditions, giving slightly higher per cent strains up to failure by tensile cracking. To this end, high plasticity index soils compacted wet of optimum have been favoured. Such preference for CH materials finds further support in the expected higher short-term erosion resistance and sealing of cracks by swelling, given time enough.

Professional practice has unquestioningly adopted this line of thinking. However, in prototype reality, within the domain of extreme value statistics that controls tensile failures, any weaker spot is sufficient to invalidate the idealized theoretical considerations and laboratory investigations. Using a material which is more cohesive means that tensile cracks tend to remain open to greater depths.

An alternative, and opposing, concept is based on accepting that tensile stresses and possible cracking are inevitable, and to limit the depth of open cracking while favouring tendencies of self-healing by erosion and clogging. As a consequence slightly clayey sands and clayey silts would be preferred to tough high plasticity index clays which sustain deeper open cracks. Moreover, for improved erosion and self-healing, materials with more broadly graded grain size are preferred. It must be noted, however, that the self-healing function is principally achieved by erodibility of an upstream cohesionless broadly graded transition, effectively retained by downstream filters.

Detailed considerations regarding cracking, erodibility, and self-healing of cores are among the principal debatable points in the present practice of embankment dam engineering. As a start, however, the good behaviour of a core lies in the relative compressibilities of the compacted zones of the embankment. Thus attention must be concentrated first on the compressibility, permeability, and shear strength of typical core materials.

The most fruitful model for interpreting the behaviour of compacted clayey soils is to liken compaction to compressing to a certain preconsolidation pressure, $\sigma'p$. Such a preconsolidation pressure should be directly related to the per cent compaction, PC%, and, presumably, within the narrow ranges of PC% in which a given project works. Beyond the $\sigma'p$ value, the nominal virgin compression index, C_c, would be essentially constant. This concept is illustrated in *Figure 44.11*, which also indicates that efficient compaction should be related to the energy absorption of the hysteresis between compression and decompression. Particle rearrangements in clays and crushed point contacts in sound uniform angular rockfills provide the most salient hysteresis of precompression.

Compressibilities of compacted core materials

The attempt to relate embankment behaviour to soil types as reflected in Atterberg limits has generally led to very pessimistic predictions of compressibilities and settlements. In comparison with the generally tenable simplified correlation of $C_c = f(LL)$ for clays (see Terzaghi and Peck[8]), *Figure 44.12a* would indicate that for borrow materials of $LL > 40\%$, there is:

1. A significantly lower virgin compressibility of compacted specimens.
2. A patently smaller compressibility of undisturbed block samples from the embankments in comparison with laboratory moulded specimens.

Laboratory tests are indispensable for design. Nonetheless,

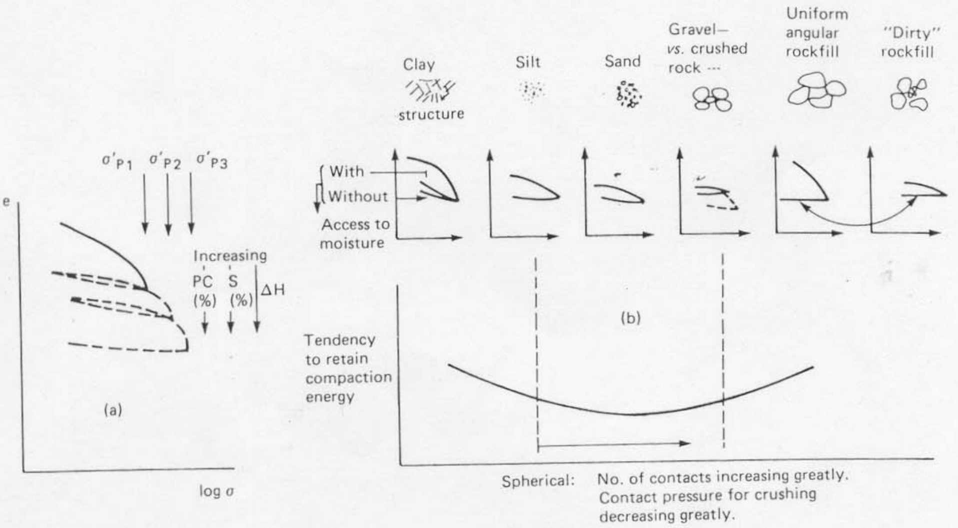


Figure 44.11 Schematic comparison of compaction grain structure and compressibility hysteresis

coefficients of adjustment must be used to correct laboratory test parameters in order to reflect field test parameters, and even to correct the latter to reflect prototype reality. This point needs emphasizing since the remarkable advances in laboratory testing in the past thirty years have insinuated that the test data may be used directly as valid.

For preliminary estimates, *Figures 44.12a* and *b* furnish proposed correlations of compressibility parameters from oedometer tests using undisturbed block samples, with index tests. Such empirical correlations were obtained from tests on unsaturated tropically weathered soils, both saprolites and laterites, as well as some aged sediments. Prudence should be adopted in extending their use beyond this range of experience.

In short, for the nominal virgin compression index, better statistical correlations are obtained with respect to maximum dry density, $\rho_{d\,max}$, derived from the standard Proctor compaction test than with regard to the liquid limit, even in very clayey soils. The final regression equation, $C_c \approx 0.23$ $(2.55 - \gamma_{d\,max})$, obtained, after adjusting all the test data to an average value of 2.67 for the relative density (specific gravity) of the soil grains, seems most appropriate both in its form and the coefficients obtained, since there is a direct association with porosity (see de Mello[7]).

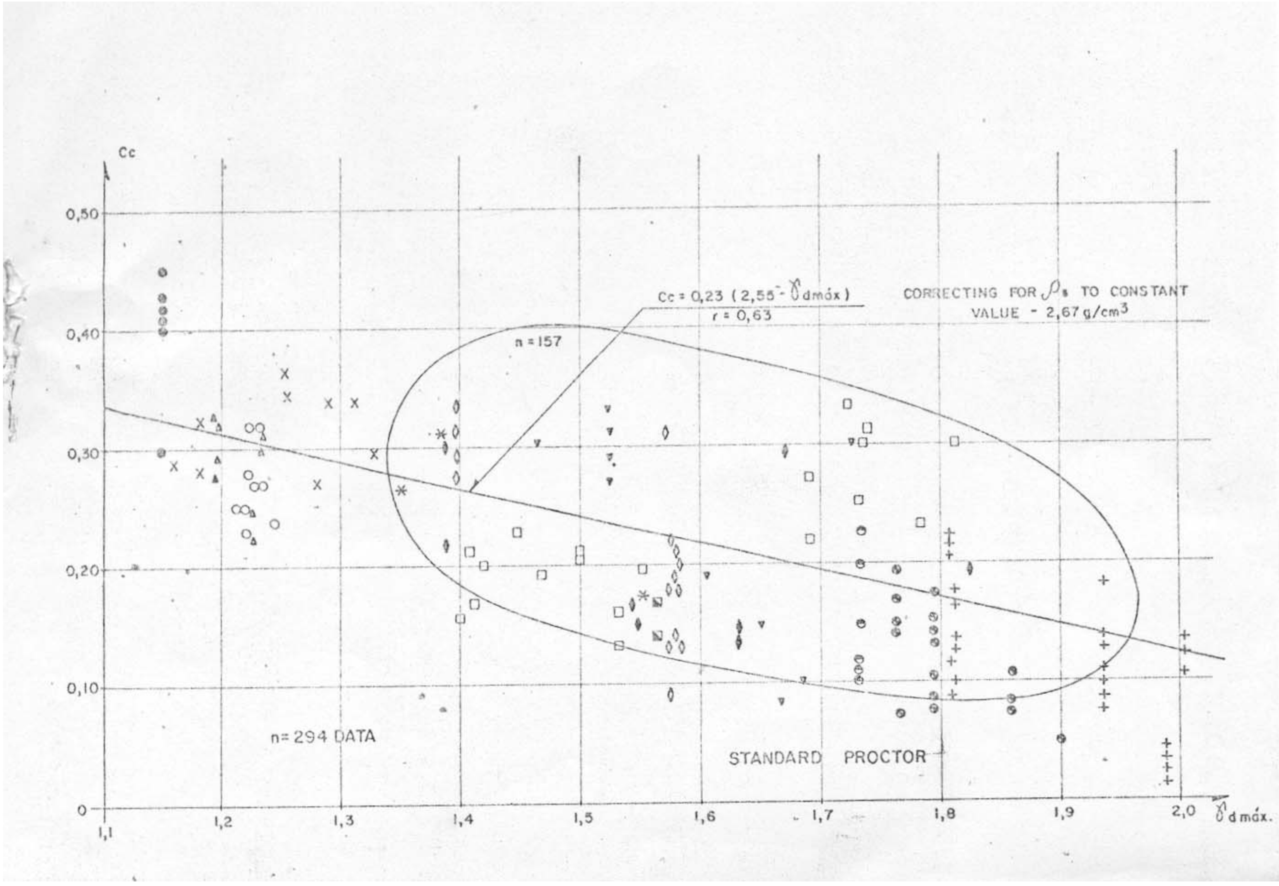
In cases such as that of compressibility settlements of porous granular material, individual results from small specimens reflect a much wider scatter, largely pertaining to the tests themselves, than obtains in larger masses of the prototype, wherein internally redistributed behaviour enforces the averaging effect. Average parameters may be used in conjunction with chosen confidence bands around the average (*Figure 44.3*). It is important to distinguish between confidence bands on single values and the narrower confidence bands on averages. In the present case it is the confidence bands on averages that apply. However, since there subsequently will be a major coefficient of adjustment between laboratory test-predicted settlements and prototype observations, it is preferable to employ the average equation, for systematic collation of the pertinent experience.

Two points merit emphasis. Firstly, best fit regressions of any broader spectrum of data must be based on some theoretical modelling in order to reach meaningful results. Secondly, for adequate design prediction the engineer must aim at an optimum point of judicious decision between too much credence given to a specific test result, and too much reliance on 'generalized experience'. The latter serves best for 'educated guesses', for feasibility estimates, or for questioning the validity of apparently unusual individual results. However, too sweeping a generalization loses practical interest to the engineer facing a given project, since the broad band of the correlation would permit estimates varying more than ten-fold for a given porosity. As an example of a very broad generalization of volumetric compressibility *Figure 44.13* is reproduced, as proposed by Janbu[9] for all mineral grains in a two-phase porous structure.

The data on compacted specimens as per *Figure 44.12b* are superposed, showing for clayey materials a behaviour somewhat more incompressible than anticipated, and the opposite in sandier materials. The latter result may be attributed to sampling, trimming and testing errors in more brittle specimens. Trends such as those illustrated in *Figure 44.13* are very useful to indicate the quantitative reduction of volumetric compressibility of a clayey soil when some gravel sizes are incorporated, thereby reducing average porosity. The The important proviso is that the percentage of gravel should not exceed about 30%, so that the matrix continues to be of clayey soil, fully enveloping the interspersed gravel.

For estimating the compaction preconsolidation pressures, $\sigma'p$, associated with different per cent compaction, PC%, the same oedometer tests have permitted the establishment of some statistical regressions such as shown in *Figure 44.14*. Since the capacity to retain precompression stresses depends on suction, it is necessary to separate the materials into groups of differing clay contents, as reflected by the $\gamma_{d\,max}$ values. *Figure 44.14a* shows for clayey materials ($\gamma_{d\,max} < 1.5$ t/m^3) and all data (specimens both moulded and trimmed from undisturbed blocks) a reasonable regression equation, $PC\% = 94.38 + 6.43 \log \sigma'p$ (kg/cm^2). Both the confidence bands on averages and on individual points are shown. The results appear reasonable, indicating a $\sigma'p$ value of about 4 kg/cm^2 (400 kN/m^2) for the 98% PC (standard Proctor) compatible with tyre pressures of heavy rollers.

Figure 44.14b shows an important difference between laboratory moulded tests and undisturbed block tests, the latter indicating perceptibly higher $\sigma'p$ values for a given PC. On repeating the regression analyses under different assumptions for the expansion index C_r, on stress release, and possibly reduced



Cc
0,50
0,40
0,30
0,20
0,10
0
1,1
1,2
1,3
1,4
1,5
1,6
1,7
1,8
1,9
2,0
γd máx.
Cc = 0,23 (2,55 - γd máx)
r = 0,63
n = 157
CORRECTING FOR ρs TO CONSTANT VALUE - 2,67 g/cm³
n = 294 DATA
STANDARD PROCTOR

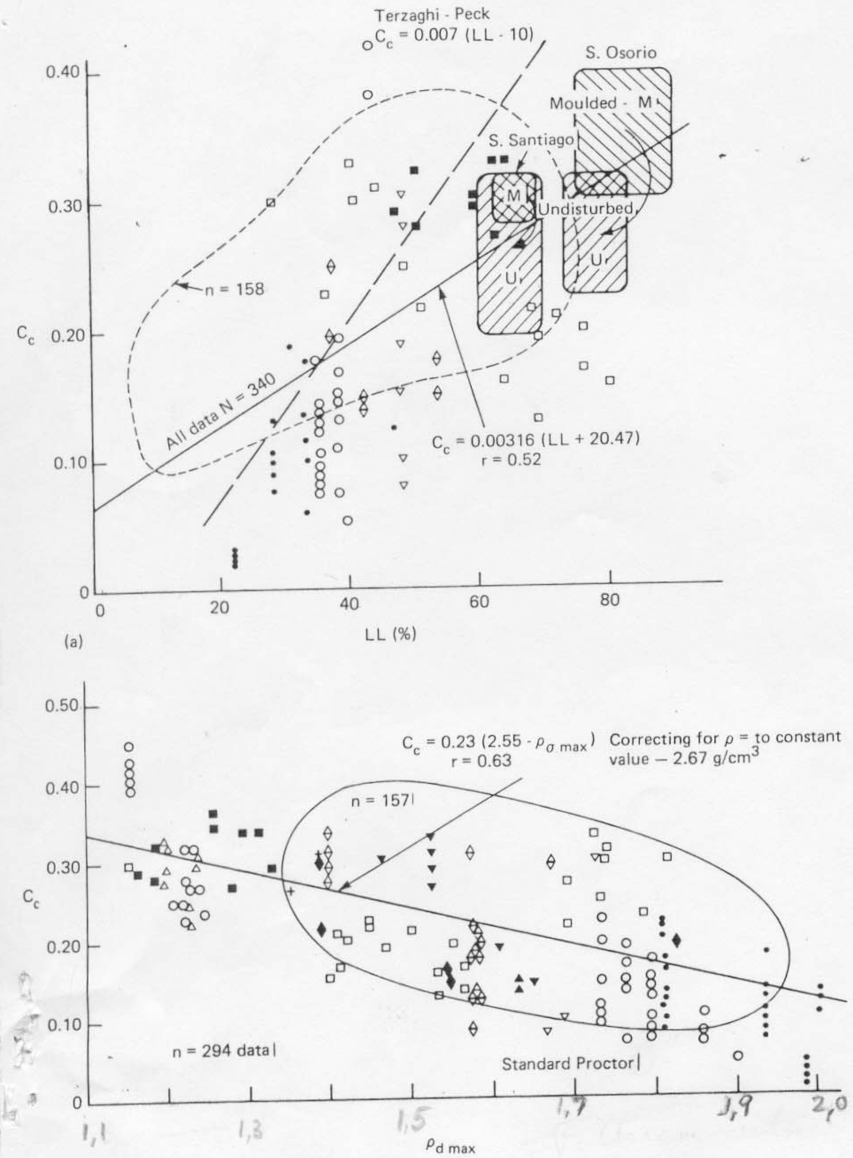


Figure 44.12 Virgin compressibilities of compacted clayey soils correlated with index tests

compression indices, C_c, associated with increasing PC and $\sigma'p$, it was concluded that the basic trends and estimated numerical values are not perceptibly affected.

Finally, despite there being insufficient data to establish a reliable correlation, *Figure 44.14c* shows a rough trend for somewhat sandier materials, $1.5 < \gamma_{d\ max} < 1.7\ t/m^3$, and for the most sandy, $\gamma_{d\ max} > 1.7\ t/m^3$.

44.4.2.2 Shear strength and permeability of compacted core materials

Historically compaction was considered as producing a homogeneous material of improved quality. Thus research investigations on stress-deformation and strength characteristics of compacted clays were programmed considering all specimens equivalent, defined by a constant initial density. The same reasoning dictated interpretation of parameters (c, ϕ, c', ϕ', E, r_u)* as constant or linearly varying.

As a result of the model of a compacted material as preconsolidated, both the shear strength envelope and the permeability coefficient must be carefully reexamined in extrapolating from low-to-medium dams to really high dams. Within the precompressed range the drained strength equation includes a c' value dependent on the PC. Also, construction period pore water pressures are negative or negligible in the more clayey materials, at least up to overburden pressures close to the

* c = cohesion; ϕ = angle of shearing resistance (primes indicate effective conditions); E = Young's modulus; r_u = pore pressure ratio (pore water pressure/unit weight of overburden)

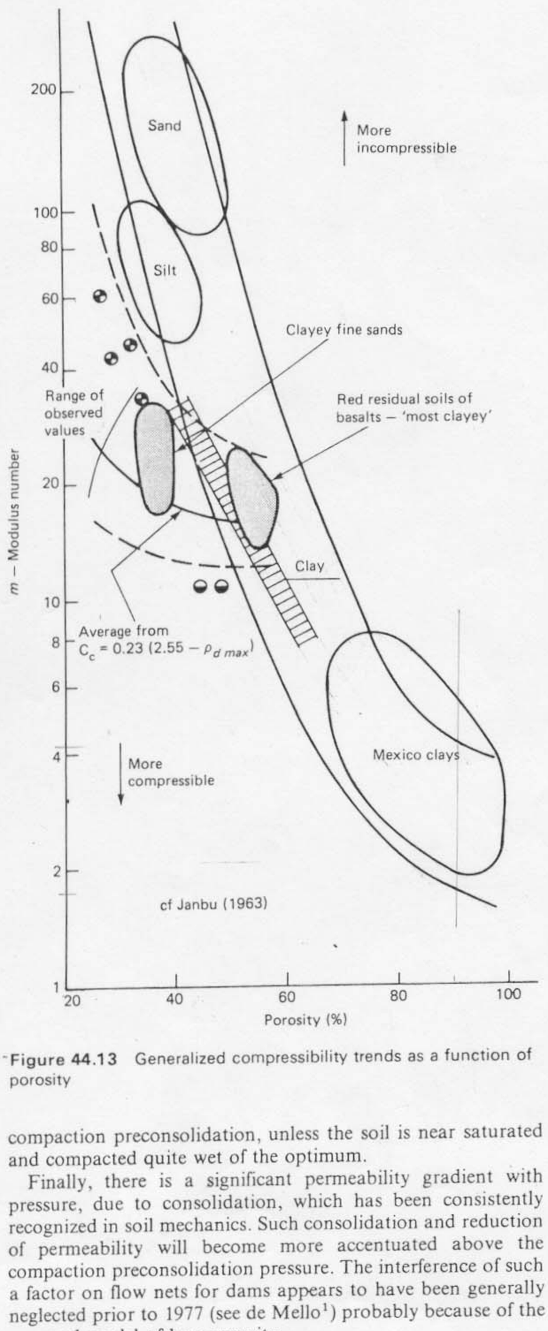


Figure 44.13 Generalized compressibility trends as a function of porosity

compaction preconsolidation, unless the soil is near saturated and compacted quite wet of the optimum.

Finally, there is a significant permeability gradient with pressure, due to consolidation, which has been consistently recognized in soil mechanics. Such consolidation and reduction of permeability will become more accentuated above the compaction preconsolidation pressure. The interference of such a factor on flow nets for dams appears to have been generally neglected prior to 1977 (see de Mello[1]) probably because of the assumed model of homogeneity.

A material constructed in a homogeneous condition but subjected to different stresses cannot remain homogeneous. The minimum permeability gradient in virgin compression that appears justifiable corresponds to somewhere between a 1.0 and 1.5 log cycle decrease of permeability for one log cycle of pressure increase. This has been interpreted as at least a one-hundred-fold variation in permeability from top to bottom on an idealized 100 m high dam cross-section studied for optimizations and comparisons (see de Mello[1]). *Figure 44.15* summarizes the principal indications on shear strength, permeability, and compressibility pore water pressures that are applicable to unsaturated clayey compacted materials.

44.4.3 Shell materials of rockfill and gravels

For the case of compacted rockfill, some of the principal points that have required revision of the original models regarding their nature are:

1. The layered heterogeneity, lift by lift, of the compacted rockfill.
2. The differentiation between compaction interlocking, crushing and compaction of the lift, and subsequent compression under overburden loading of the same lift.
3. The lack of analogy between compression observations in field compaction tests and in the construction period settlements.
4. The evidence that rockfill compaction also induces precompression effects, similar to the nominal preconsolidation pressures in compacted clayey fills.

In short, compaction of rockfills arose exclusively because of concern about deformation, but the concomitant benefits on strength and stability have not been adequately capitalized until recently.

Observations have clearly established that sound uniform angular quarried rock shells can be as compressible as the compacted clay core. It is only in broadly graded 'dirty' rockfill, in which point contacts are infinitely more numerous, so that high crushing stresses are not reached, that rockfills exhibit very low compressibility. Once again, the association with porosity and compaction precompression seems vindicated.

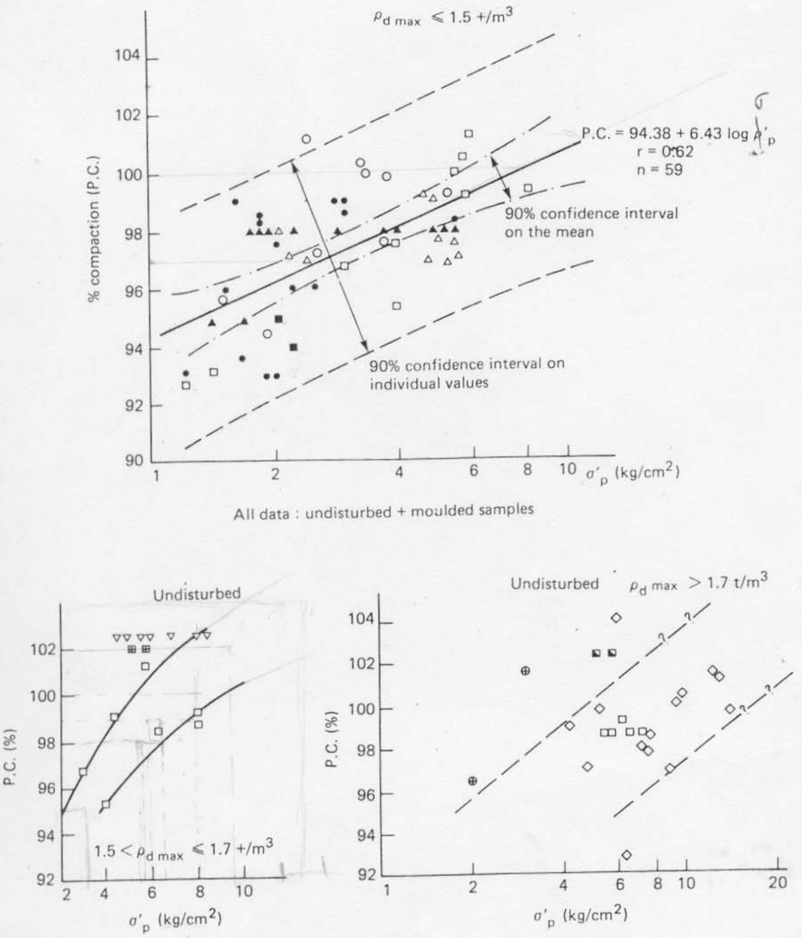
In comparing compressibilities of clean sound rockfill shells with those of alluvial or glacial gravels it is not surprising that the latter should be very much less compressible (exhibiting nominal elasticity moduli of the order of 5 to 10 times higher). *Figure 44.17* illustrates comparative magnitudes.

There are a number of reasons why rockfill is more compressible. Firstly, a cobble or gravel particle represents the toughest central core of a rock fragment, after all its weaknesses have been removed by surface attrition. Secondly, because of their rounded shape most particle-to-particle contacts are not point contacts, but contacts across moderate areas; and any minor deformation, by compression or crushing, causes a very rapid rate of increase of contact area with deformation. Finally, both because of smaller sizes and because of typical grain size distributions (well-graded and ideally concave upward cumulative curve, for minimal porosity), the number of solid contacts is so high that average stresses across them do not reach disproportionate ratios relative to overburden average stresses, as is the case with angular point contacts. Concomitantly, however, the load–unload hysteresis is minimal (as it is in sands) and the precompression effect is difficult to detect.

Both in rockfills and in highly incompressible gravels it must be noted that the use of dry density (or porosity) for an index of satisfactory compaction, in construction specifications, is far from adequate. A minute compression that fully absorbs compressibility up to the applied pressure will not cause any change of dry density, in comparison with the major dispersions that accompany changes of grain size distribution within the sample.

The permeabilities of gravels and rockfill need not be discussed. They are generally recognized as free-draining. In the case of well-graded materials (Section 44.4.5) the question merits closer examination.

The question of stability of rockfill slopes suffered greatly, in the period 1960–75 as a result of the oversimplified model of



(a)

Figure 44.14 Statistical regressions on compaction preconsolidation pressures as functions of per cent compaction, PC

conventional soil mechanics that considered rockfill as uniform, and rock fragments as analogous to large grains of sand. Early rockfill construction with dumped rock, and acceptance essentially of angles of repose, employed slopes generally steeper than subsequently began to be considered necessary in compacted rockfills. Yet there is no record of instability of such slopes. Such sound evidence of field performance suggests the need for re-examination of current routines relating to stability of rockfill slopes and factors of safety. *Figure 44.18* summarizes some data on representative well-proven external slopes of rockfill dams.

The following discussion, connected with crushability and precompression hysteresis, applies in decreasing degrees to rockfills of softer rock and to broadly graded 'dirty rocks', and least so in the case of cobbles and gravels.

The principal factor for assessment of stability of compacted rockfill slopes is the strength envelope, which in the period 1960–75 was extensively investigated by large-scale triaxial tests. One direct consequence of crushing was the more noticeable curvature of the strength envelopes. Since this could be the only factor capable of contributing to somewhat deeper sliding surfaces (assuming sound rock foundations) a reference study collated data available up to the end of 1976. *Figure 44.19* summarizes the information gathered.

Two points must be made. Firstly, that it is doubtful that rock-fragment interlocking and contact crushing during compaction such as occurs in the field could be reasonably imitated in the laboratory. It has been suggested that even minute differences in such factors may be of appreciable significance (at low stresses). *Figure 44.20* illustrates a method used in Spain (see Salas[10]) to test a compacted rockfill to failure by jacking in passive pressure conditions. The figure also shows a similar test which employs anchors to load a horizontal plate, thereby causing a sliding failure in the active condition. The latter could better represent a failure of an existing slope.

As regards large-scale triaxial tests, *Figure 44.18* provides an indication of the hysteresis benefits that would be expected (analogous to anisotropically consolidated preconsolidated clays) if some tests had been run imposing a failure by decrease of lateral stress.

44.4.4 Filter transition materials

The physical phenomena leading to localized concentrations of seepage stresses (analogous to stresses around a cavity), and localized internal erosion of particles to form contiguous voids are the result of a combination of situations subject to

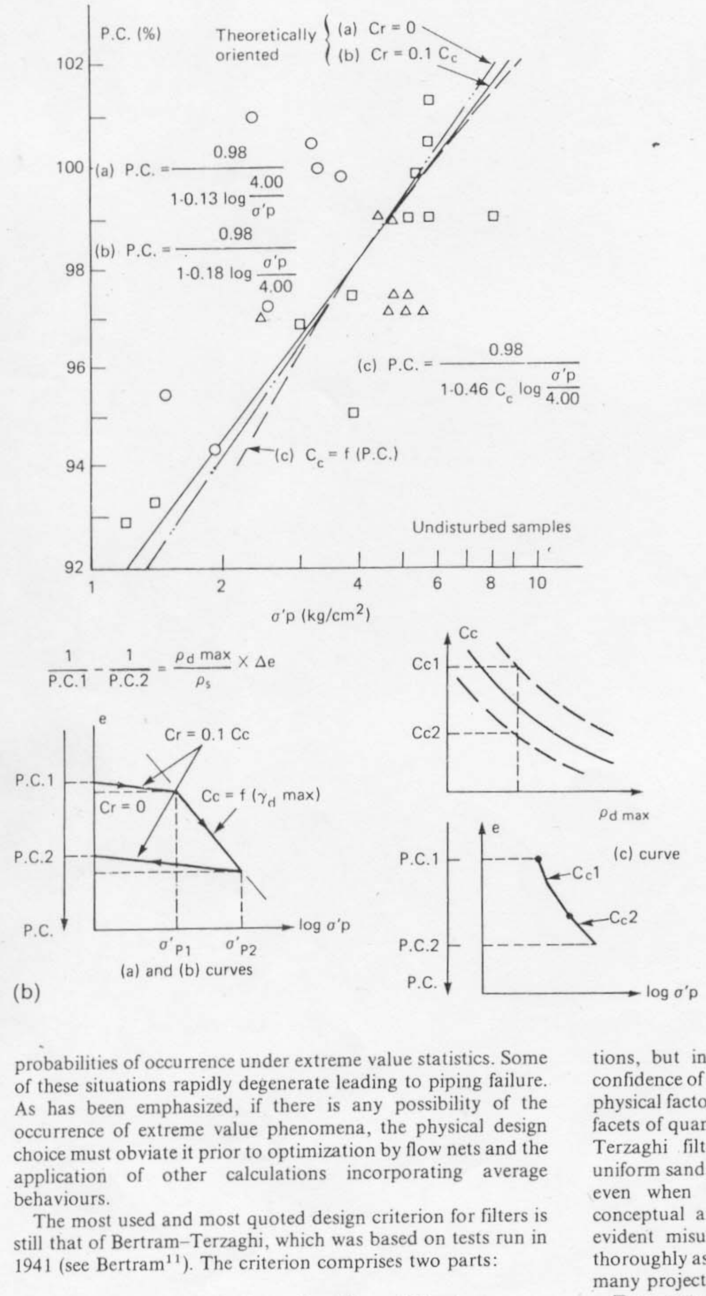


Figure 44.14 (*continued*)

probabilities of occurrence under extreme value statistics. Some of these situations rapidly degenerate leading to piping failure. As has been emphasized, if there is any possibility of the occurrence of extreme value phenomena, the physical design choice must obviate it prior to optimization by flow nets and the application of other calculations incorporating average behaviours.

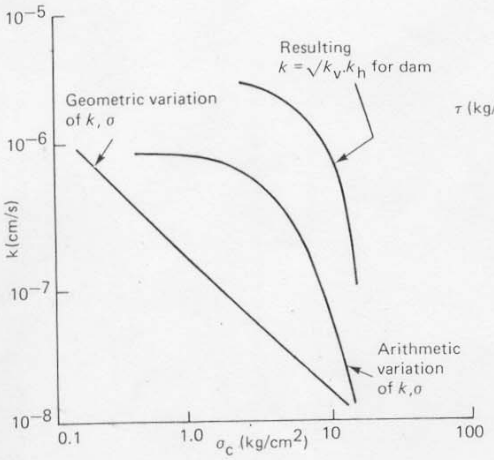
The most used and most quoted design criterion for filters is still that of Bertram–Terzaghi, which was based on tests run in 1941 (see Bertram[11]). The criterion comprises two parts:

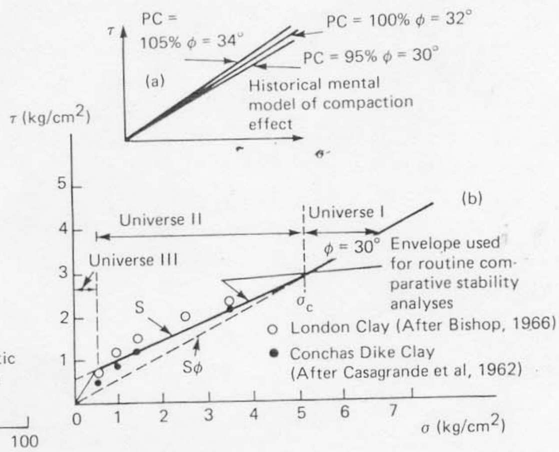
The permeability criterion, D_{15} filter $> 5 D_{15}$ base
The filtering criterion, D_{15} filter $< 5 D_{85}$ base.

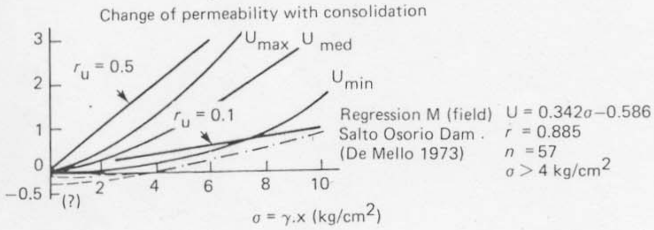
The principle of filtering action is to prevent the movement of particles of the soil through the voids of the filter. It is therefore directly concerned with the particle sizes of the base material and void sizes of the filter.

Many research programmes have aimed at improving the original criteria. They have shed light on parameters of importance not incorporated in the erstwhile recommendations, but in general have not systematically improved the confidence of theoreticians in the analytical understanding of the physical factors at play, and their deterministic and probabilistic facets of quantification. In practice one finds that the Bertram–Terzaghi filter criteria, originally developed for relatively uniform sands, continue to be used in the vast majority of cases, even when representing an extrapolation of the original conceptual and empirical bases to the point of constituting evident misuse. It is important to discuss the questions as thoroughly as possible, since piping problems continue to plague many projects, even those of very great importance.

Four separate avenues may be discussed from the outset. Firstly, there is a gross conceptual error in referring to median sizes, D_{50}, in any frequency distribution curve, be it of grains or of pores. Thus, despite the meticulous tests by Karpoff[12] from which the USBR (United States Bureau of Reclamation)[13] criteria developed, the fact that they concentrated on ratios of median diameters suggests that they should be questioned. Many soils may have the same D_{50} but widely different ranges of grain and pore sizes depending on the coefficient of uniformity ($U = D_{60}/D_{10}$) of the distribution curve.







σ/σ_c %	10	20	30	50	70	
OCR	10	5	3.3	2	1.4	
$S/S\phi$ %	237	200	173	139	116	○
$S/S\phi$ %	165	151	140	113	108	●

Figure 44.15 Data on behaviour of compacted clayey materials used for illustrative optimizations (de Mello[1])

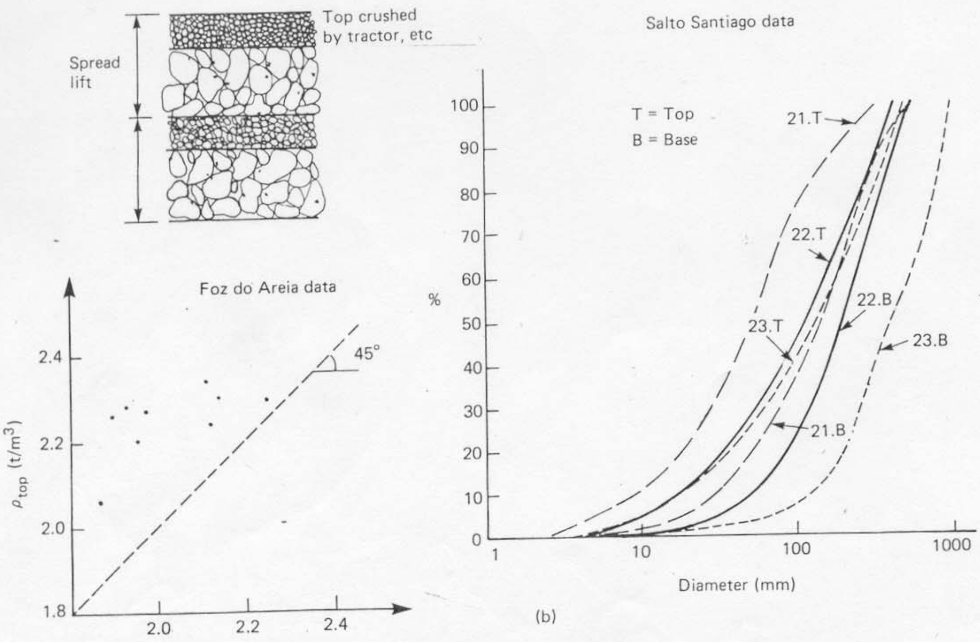
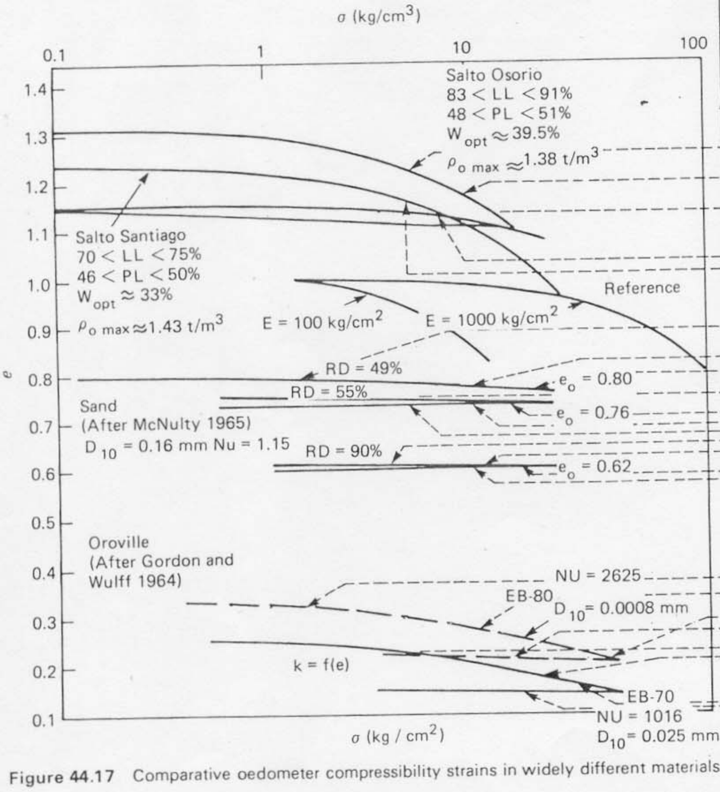


Figure 44.16 Comparative density top-base on rockfill lifts



Oedometer % strain $\varepsilon = \Delta e / 1 + e_o$ E kg/cm²							
Compression						Recompr.	
σ 0.5 – 4.0 kg/cm²		4.0 – 10.0 kg/cm²		4.0 – 20.0 kg/cm²		0.5 – 10.0 kg/cm²	
ε (%)	E	ε (%)	E	ε (%)	E	ε (%)	E
3.03	120						
		3.79	160				
				8.01	210		
2.47	141					1.96	460
		3.69	163	8.76	183	2.47	385
0.78	450						
		0.44	1360	0.61	2600		
0.45	780						
		0.43	1400	0.91	1760	0.14	6800
0.25	1400	0.06	10 000				
				0.46	3500	0.15	7000
2.41	150						
		2.11	280				
				4.21	380	1.53	620
2.70	130	2.59	230				
				4.77	340	1.19	800

Figure 44.17 Comparative oedometer compressibility strains in widely different materials

Secondly, the suggestion of running nominal piping tests at each project should be regarded as somewhat risky and misguided. Laboratory tests are indispensable to comprehend and formulate scientific criteria, but design conditions must cover situations beyond those synthesized in the laboratory and presumed representative at a given project. What combination of conditions of base soils and adjacent filters will cover the wide dispersion possible in a project involving thousands of square metres of base-filter interface? How do these interface conditions change due to stress–strain adjustments in the embankment during construction and as a result of reservoir seepage? Will the tests be conducted with seepage stresses causing compression or tension, and to what degree? What conditions of obliquity of seepage stresses versus overburden stresses will be investigated? What limiting gradients will be considered representative? It is unreasonable to visualize adequate conditions for experimentation of such complexity in each project, even when major.

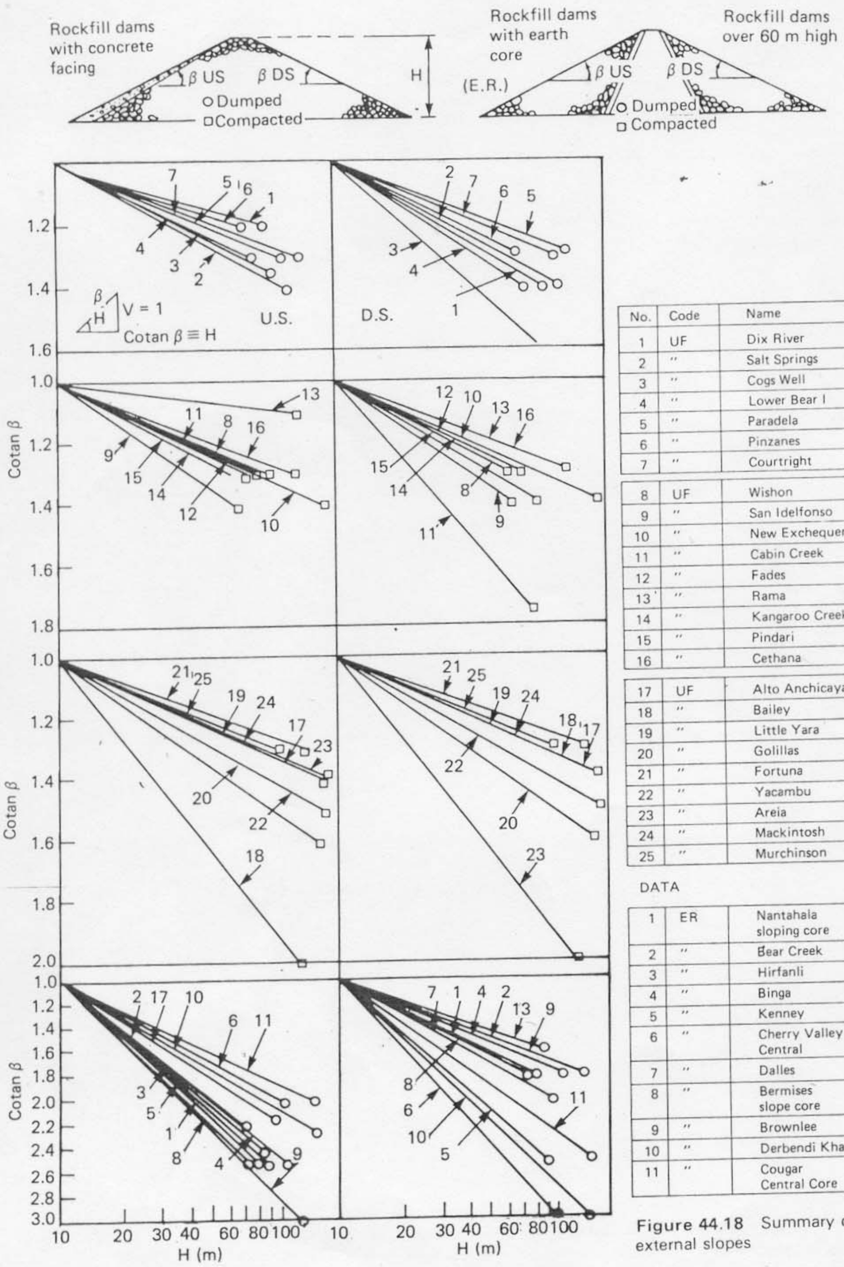
Thirdly, one might set aside the thought of separate testing of cohesive soils. The cohesion parameter is merely one of the two shear resistance parameters in the equation $s = c' + \sigma' \tan \phi'$, and the resistance to erosion is a shear resistance. A cohesive material is obviously more resistant to erosion, while the cohesion c' persists. However, c' varies significantly with variations of σ' and under unfavourable conditions of $\sigma' \to 0$ or $-$ve (going to tensile conditions), the cohesion may be totally removed with time. The fact is that the secondary effects of the seepage stresses on σ' have been omitted in discussions of cohesionless filters. When such theorization advances, it will be possible to consider the advantages of cohesive materials whenever σ' suffers moderate compressive increases due to the seepage gradient itself.

Finally, it seems inappropriate to establish filter criteria based on coefficients of permeability. Determinations of permeability coefficients intrinsically mask any localized preferential conditions, either of a single macropore compared with multiple smaller pores, or of a single seepage path shortened by a crack as compared with the average thickness of specimen, used for computing average gradients. It is an approach totally dependent on averages, and could not be appropriate for extreme value conditions.

In the Bertram–Terzaghi criteria, there are many points of indisputable validity in concept, and the points wherein improvements are needed become clear. Terzaghi evidently limited his considerations to continuous distribution curves, both of base material and of filter. Criteria for confirming the indispensable conditions of continuity are discussed in Section 44.4.5.

In any frequency distribution curve one should avoid discussing the tail ends, and therefore the finer sizes have been intuitively included as D_{10} or D_{15} sizes while the larger sizes are generally included as D_{85} sizes. Note that these percentiles should be judiciously adjusted in accordance with the presumed nature of the deposits. The intent is to represent the finer and coarser sizes of significance, pertaining to the continuum, and not those of spurious occurrences. Thus, if self-filtering action is assumed within the base material grain sizes, it should be sufficient to retain the D_{85} of the base.

After decades of professional use the following simple criterion was proposed (see de Mello[14]) for checking on the self-filtering action of a material. This was prompted by the problems faced in gap-graded open work piedmont gravels of a very major dam,



No.	Code	Name	Location	Year
1	UF	Dix River	Kentucky	1925
2	"	Salt Springs	California	1931
3	"	Cogs Well	"	1934
4	"	Lower Bear I	"	1952
5	"	Paradela	Portugal	1955
6	"	Pinzanes	Mexico	1956
7	"	Courtright	California	1958
8	UF	Wishon	California	1958
9	"	San Idelfonso	Mexico	1959
10	"	New Exchequer	California	1966
11	"	Cabin Creek	Colorado	1966
12	"	Fades	France	1967
13	"	Rama	Yugoslavia	1967
14	"	Kangaroo Creek	Australia	1968
15	"	Pindari	"	1969
16	"	Cethana	Tasmania	1971
17	UF	Alto Anchicaya	Colombia	1974
18	"	Bailey	USA	1974
19	"	Little Yara	Australia	1977
20	"	Golillas	Colombia	1978
21	"	Fortuna	Panama	1982
22	"	Yacambu	Venezuela	1982
23	"	Areia	Brazil	1980
24	"	Mackintosh	Australia	1981
25	"	Murchinson	"	1982

DATA

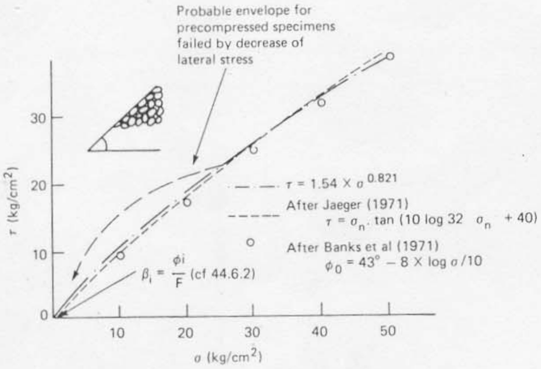
1	ER	Nantahala sloping core	North Carolina	1942
2	"	Bear Creek	" "	1953
3	"	Hirfanli	Ankara	1956(?)
4	"	Binga	Philippines	1956(?)
5	"	Kenney	British Columbia	1957
6	"	Cherry Valley Central	San Francisco	1954
7	"	Dalles	Dalles	1957
8	"	Bermises slope core	Canada	1955
9	"	Brownlee	Oregon	1959
10	"	Derbendi Khan	Iraq	1959
11	"	Cougar Central Core	Oregon	1959

Figure 44.18 Summary data on some rockfill dams and external slopes

and in glacial till materials subjected to construction segregation. Any grain size distribution curve can be subdivided into two (or more) curves, of finer and coarser fractions, and the filter criteria can be employed to check if the latter serves as a filter to the former (*Figure 44.21*). In truth such a criterion is very conservative because generally the intergranular structure of the composite material is denser than that of each fraction.

In order to retain the D_{85} base grains the filter should possess a slightly smaller D_{85} pore size. In the early 1940s there were no data on pore sizes, and therefore intuition led to specifying the limiting grain size (D_{15}) of the filter. In uniform materials of $1 < U < 5$ it has been established that pore sizes are of the order of 1/5 to 1/10 of the grain sizes. If such a relation is adopted as constant, it is feasible to specify grain sizes to signify corresponding pore sizes. The two bounds of the filter criterion are clearly established for the following:

1. D_{15} filter $> 5\,D_{15}$ base as a permeability criterion, to guarantee adequate relative permeabilities. Hazen's empirical relationship $k \approx 100\,D_{10}^2$ (see Taylor[15]) indicates



Rock type / Regression	Ref.
El Infiernillo diorite $\tau = 1.10\ \sigma^{0.870}$	Marsal – 1967, 1971, 1973
Idem silic. conglomerate $\tau = 1.27\ \sigma^{0.846}$	Marsal – 1971, 1973
Pizandaran sand + gravel $\tau = 1.27\ \sigma^{0.876}$	Marsal et al. 1967
San Francisco basalt $\tau = 1.54\ \sigma^{0.821}$	Marsal 1971
Netzahualcoyotl conglom. $\tau - 1.79\ \sigma^{0.881}$	Gamboa & Benassini – 1967
Malpaso conglomerate $\tau = 1.59\ \sigma^{0.808}$	Marsal – 1973

Figure 44.19 Some summary data on curved strength envelopes in quarried granular materials

that this criterion would guarantee filter permeabilities of the order of twenty-five times the base permeability. Therefore, by continuity of flow, the gradient (that is, erosive stress) within the filter would be 4% of the gradient in the base. Thus, if erosive instability arises in the base material the filter would have an average factor of safety, FS > 25 against the start of a similar phenomenon within itself. A good filter must needs be a good drain.

2. D_{15} filter $< D_{85}$ base for a presumed stereometric filtering criterion. Intuition cannot justify this criterion. Assuming perfect self-filtering within the continuous base grain size one is forced into accepting D_{85} base as a reference, and to refer it to D_{15} filter grains. But if any probability of segregations is feared, by intuition one should consider the risky pores as those based on occasional clusters of D_{85} grains and neighbours, and the grains most subject to washing through as the D_{15} base and neighbours.

Doubtless there must be some washing through of fines in the use of conventional Bertram–Terzaghi criteria, if one considers merely stereometric hindrance. The crux of the problem lies in analysing the seepage stresses at play and their consequences, especially towards progressive aggravation. Since a seepage gradient is an effective stress acting on the soil mass traversed, it must produce consequent strains (volume changes of the pores). Therefore compression conditions are definitely favourable in comparison with those causing expansion. Also, with the start of washing through, and concomitant deposition of fines in the filter, there are changes in the relative permeabilities. But it is in the base material that arching resistance to continued washing out must be generated to cause a stagnation of the phenomenon. Within the filter, the fines washed in will be difficult to stabilize.

In short, in analysing filter criteria and piping phenomena it is necessary to consider more than grain and pore sizes, and to include secondary redistributions of stresses and permeabilities. All such effects have greater probabilities of erratic occurrence in well-graded materials, and are discussed in Section 44.4.5.

In the design of filter drain features it is important to dimension the drains with an ample factor of safety to avoid developing hydraulic heads that might impair either the downstream slope stability or the erosion-free drainage within the filters themselves, even under local exceptional seepages. The drains themselves are analysed by flow nets, and some designs require sandwich drains of successive filters for increased flow capacity.

As regards deformability, it has been noted that uniform sands and gravel or crushed aggregate successive filters exhibit much higher nominal moduli of elasticity than either core or shell materials. Thus, subvertical filter drains in earth core dams have been responsible for some notable cases of hang-up of the core. The present tendency is to minimize compaction of such elements; light vibratory pads and copious watering for downward seepage are methods of treatment used to avoid honeycombs. Despite the trend in the right direction it is difficult to achieve a sufficient increase of compressibility to match the settlement of contiguous zones without design expedients on the zoning itself.

It should be noted in passing that grain shapes also exert some influence. For instance, crushed rock filters give roughly 30 to 50% higher porosities and definitely higher compressibilities than subrounded gravels of equivalent diameters.

44.4.5 Well-graded materials of wide range of grain sizes

Well-graded materials ranging from cobbles or gravels to silt and clay sizes have been considered among the best materials for dam construction. With minor adjustments, such as scalping of cobble sizes for the core or washing of the fines for shells, they have been sought as ideal materials for both. Recently, however, they have been recognized as very sensitive to radical variations in behaviour as far as permeabilities and internal erodibility are concerned.

It is unnecessary to discuss compressibility and shear strength parameters, both of which are very favourable, as can be seen from the very high densities of the compacted materials. In fact,

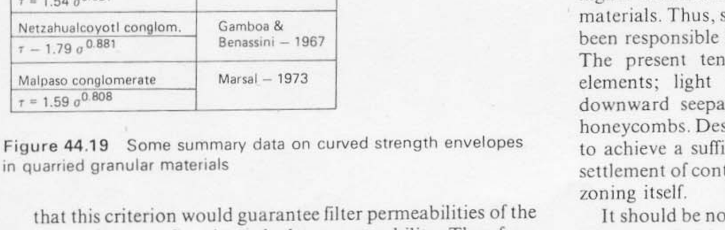


Figure 44.20 Used and proposed field test methods for compacted coarse granular materials

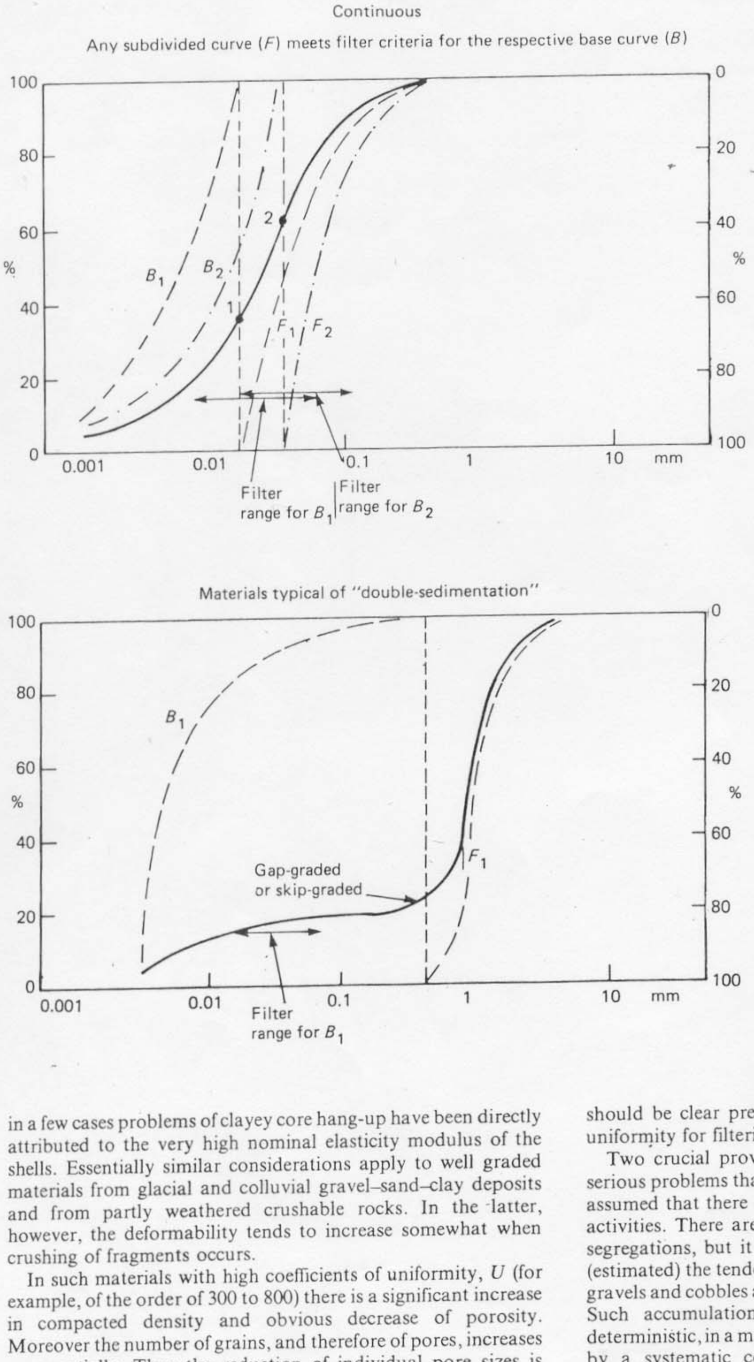


Figure 44.21 Grain size continuity compared with gap-grading

in a few cases problems of clayey core hang-up have been directly attributed to the very high nominal elasticity modulus of the shells. Essentially similar considerations apply to well graded materials from glacial and colluvial gravel–sand–clay deposits and from partly weathered crushable rocks. In the latter, however, the deformability tends to increase somewhat when crushing of fragments occurs.

In such materials with high coefficients of uniformity, U (for example, of the order of 300 to 800) there is a significant increase in compacted density and obvious decrease of porosity. Moreover the number of grains, and therefore of pores, increases exponentially. Thus the reduction of individual pore sizes is much greater. Hence with their low permeabilities they serve as core materials and even develop construction pore water pressures. Moreover, with the fines removed by washing (e.g. the minus #4 sieve) they have been considered as 'gravels', and therefore presumed pervious. As the principal reduction of porosity is by exclusion of the larger pores, it appears that there should be clear preference for materials of high coefficient of uniformity for filtering action of filters and transitions.

Two crucial provisos must be judged as the source of the serious problems that have occurred in many projects. First, it is assumed that there is no grain size segregation in construction activities. There are no published data on tests on grain size segregations, but it is clear that with any materials of $U > 50$ (estimated) the tendencies to segregation can be great, the larger gravels and cobbles accumulating along the end of the spread lift. Such accumulation is not random statistical, but clearly deterministic, in a manner analogous to that of 'natural selection' by a systematic construction procedure. This is especially dangerous since construction specifications indicate preference for spreading and compacting parallel to the axis of a dam, and therefore preferential flow lines will be directly upstream–downstream, subhorizontal.

The second point concerns what may be called a 'continuity of dense packing' of all the fractions. This is generally favoured by

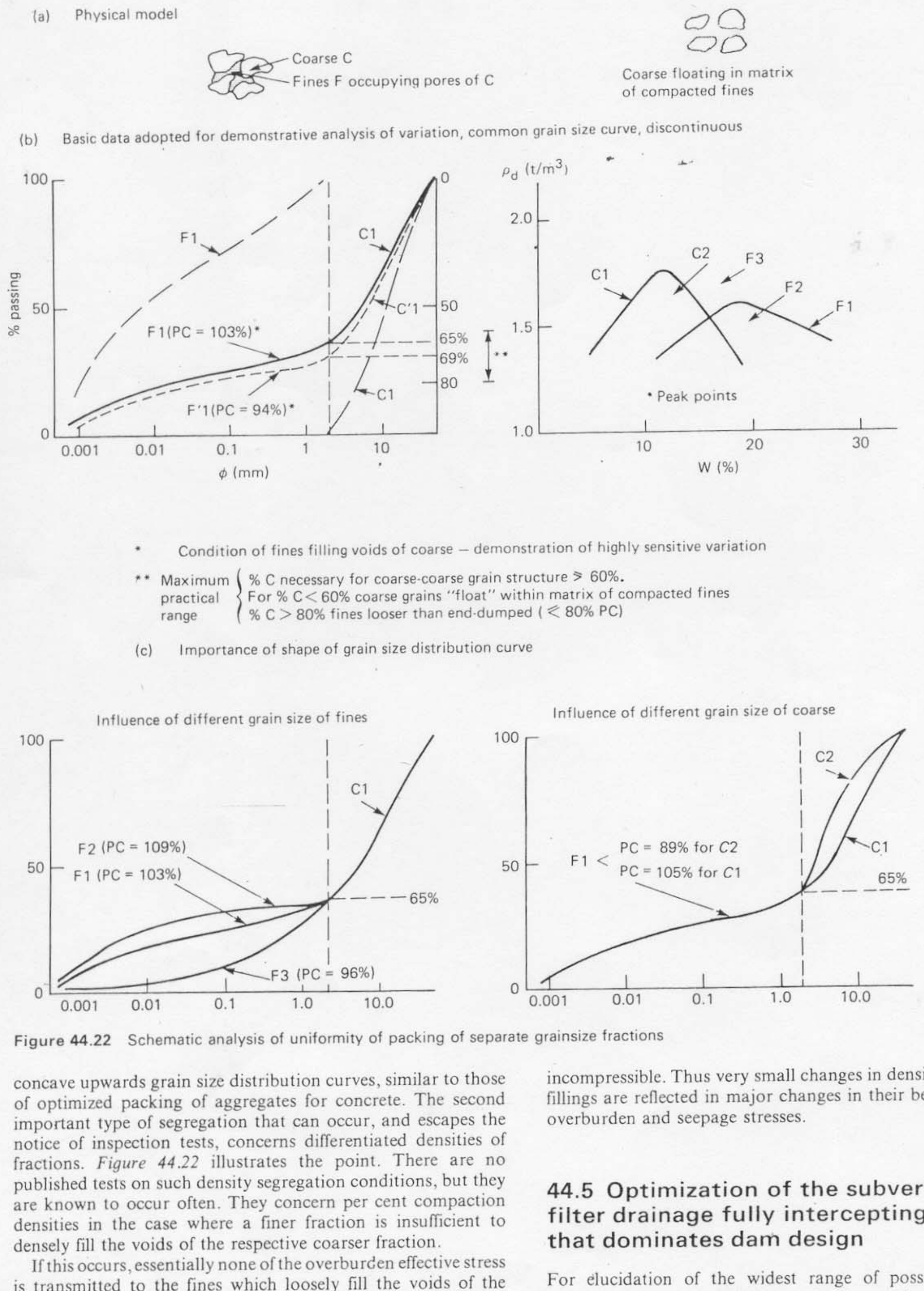


Figure 44.22 Schematic analysis of uniformity of packing of separate grainsize fractions

concave upwards grain size distribution curves, similar to those of optimized packing of aggregates for concrete. The second important type of segregation that can occur, and escapes the notice of inspection tests, concerns differentiated densities of fractions. *Figure 44.22* illustrates the point. There are no published tests on such density segregation conditions, but they are known to occur often. They concern per cent compaction densities in the case where a finer fraction is insufficient to densely fill the voids of the respective coarser fraction.

If this occurs, essentially none of the overburden effective stress is transmitted to the fines which loosely fill the voids of the coarser less compressible structure. Such fines under no effective overburden stress and subjected to seepage stresses are very easily eroded out of the voids, leading to piping. Gravels with fines, with wide total range of grain sizes and discontinuous grading are very sensitive to differences, because they are very incompressible. Thus very small changes in densities of the void fillings are reflected in major changes in their behaviour under overburden and seepage stresses.

44.5 Optimization of the subvertical filter drainage fully intercepting feature that dominates dam design

For elucidation of the widest range of possible instability analyses, the most appropriate embankment dam to consider is the so-called homogeneous section with chimney filter. In order to illustrate the optimization on the dam superstructure, five alternative positions of the filter drain chimney were chosen (see de Mello[1]), with routine flow net assumptions (*Figure 44.23*).

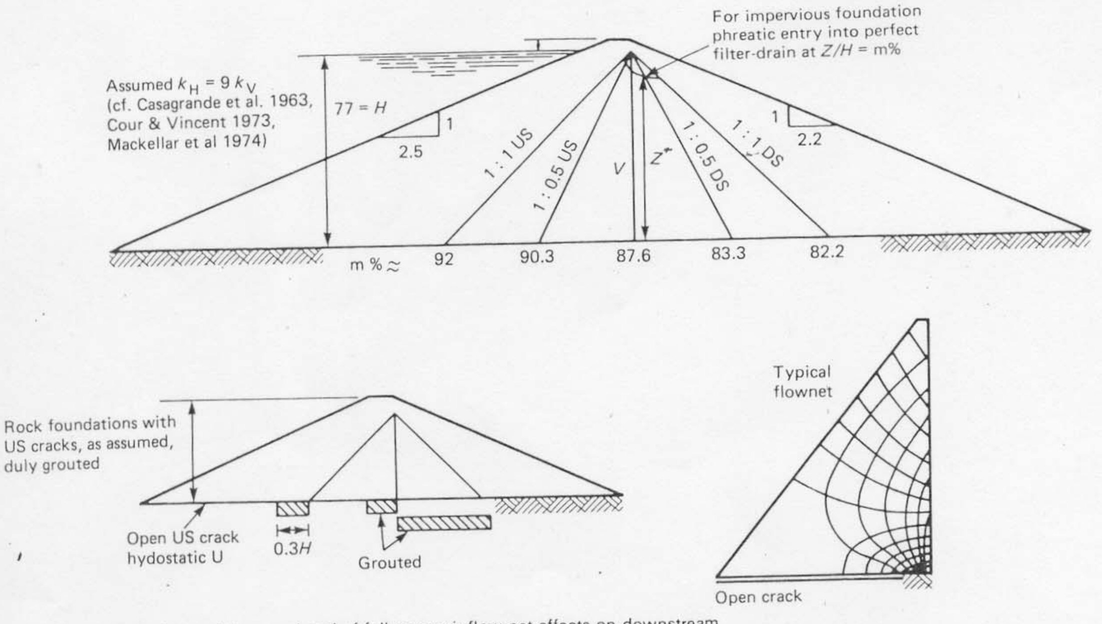


Figure 44.23 Basic conditions analysed of full-reservoir flow net effects on downstream

The fully efficient filter drain (phreatic boundary) is a nearly attainable condition, though extreme and favourable. The most frequent assumption regarding the foundation is that it is completely impermeable which represents a conservative case. Its pore water pressures are designated u_{imp}. Moreover, a frequently possible very unfavourable foundation condition can be represented in first-order approximation by an open horizontal crack under the upstream zone of the dam, as indicated at the base of *Figure 44.23*. In many competent rocks (e.g. sheeted granites, basalt flows, etc.) such cracks are a frequent feature, and over a large part of the upstream foundation it is reasonable to anticipate that unevenly distributed vertical effective stresses may not compress such cracks sufficiently for them to become impervious. Obviously an open crack should be effectively sealed by grouting (DP1). It is assumed that the crack is grouted for a minimum length of $0.3H$ upstream from the chimney filter, and at least as far as the dam axis for cases of the chimney filter inclined downstream. A typical flow net is shown for the vertical chimney. The pore water pressures involved are designated as u_{cr}.

First-order comparative merits of the different positions of chimney filter may be assessed by highly simplified hypotheses of stability wedges. The crude simplicity of the physical models and statics does not alter the comparative conclusions. Absolute priority must be given to minimization of destabilizing features of the reservoir soft load on the downstream wedge. A second criterion of preference may be established with regard to minimizing the change of conditions (or pore water pressures) from end-of-construction to full reservoir.

The destabilizing soft load due to reservoir filling may be considered as the sum of the horizontal pore water pressure thrust, u_H, at the back of the wedge (*Figure 44.24*) plus the component $u_v \tan\phi$ which the vertical pore water pressure thrust, u_v, applies unfavourably. Meanwhile, the stability of a downstream wedge of weight, W, is related to the component $W \tan\phi$. The relative destabilizing potentiality in the different cases may be reflected through an index $C = (u_H + u_v \tan\phi)/W \tan\phi$. Preference for a design section is established not only through comparison of the indices, C, for different chimney positions, but also on the basis of the comparison of the range of uncertainty affecting such values for the different chimney positions, depending on the variations possible in the total thrusts u_H and u_v. Of course, the greater the proportional dependence of the downstream stability on the reservoir pore water pressure thrusts, the greater the range of uncertainty (which would signify a poor design choice, at least requiring increased FS values in design calculations).

Figure 44.24 summarizes the computed indices, C, for hypothetical sliding wedges of vertical back AA along the dam axis. The indications strongly favour inclining the chimney at least 1:0.5 upstream. Extended calculations with different locations for the line AA confirm such a design preference, with regard to downstream stability, and certainty thereof (substantially independent of the flow net assumed).

In *Figure 44.24* the same simplified model is used to indicate the comparative ranges of change of conditions of pore water pressure thrusts on first reservoir filling. For mere illustration two hypothetical construction period pore water pressure coefficients were used. Once again the preference for upstream inclined chimneys is demonstrated.

Since a downstream failure under full reservoir must, in design, be rendered really impossible, the validity of the preference must be confirmed under as many models of hypothetical failure as possible. The additional model of *Figure 44.25* was visualized as an example. The possible sliding wedges were assumed separated by the chimney filter and horizontal drainage blanket. The upstream pore water pressure thrusts were taken along a hypothetical plane 5 m upstream of the chimney. Foundation uplift was excluded, in view of the purely comparative analysis. The physical model is closely akin to the limiting case of rockfill dams (i.e. fully drained downstream masses) with upstream impervious membranes (upstream-deck dams).

The computed indices, C, clearly repeat the indications in favour of upstream inclined wedges. Special attention is also drawn to the tabulation at the bottom of the graph. It should be noted that in routine uses of flow nets, a small percentage (4 to 8%) of the total hydrostatic thrust survives to be transmitted to the final 5 m behind the filter, no matter what flow net is

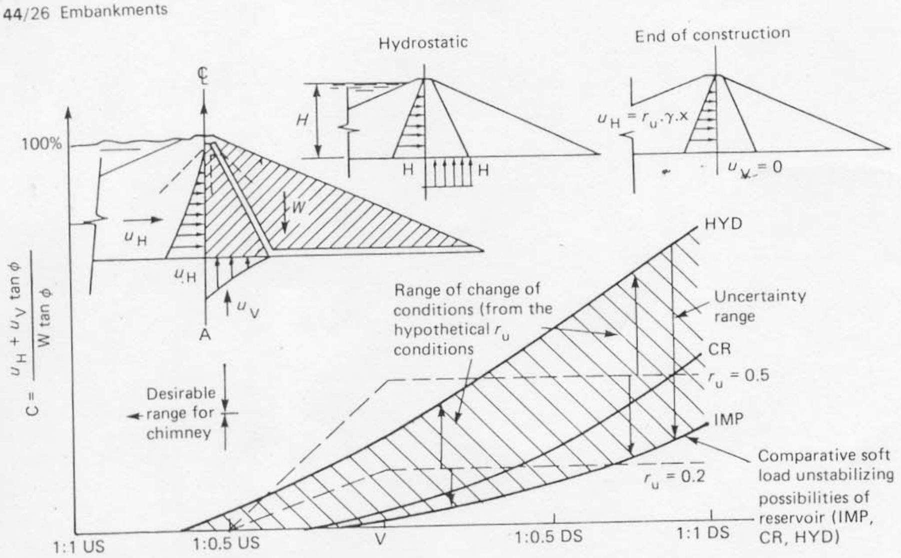


Figure 44.24 Priority optimization of chimney position for DS stability

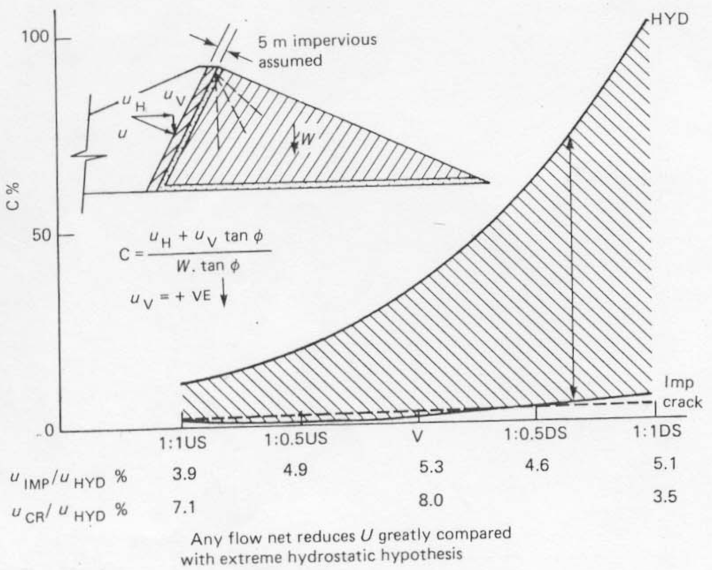


Figure 44.25 Optimizations of chimney position using another simple stability model (de Mello[1])

assumed. The automatic assumption of gradual loss of head in accordance with flow nets must be questioned first.

On recalling the preoccupations of the past score of years with regard to cracking and pervious discontinuities, one further notes that, especially under transient conditions of very rapid reservoir filling, the ratios of apparent impermeability tend to be accentuated, fostering membrane-type behaviour. The disproportionate uncertainties regarding magnitudes of the important soft load, u_{imp} to u_{hyd}, impose a design choice whereby the direction of such a load is the initial guarantee that its magnitude is immaterial. With interceptors inclined upstream, it is practically immaterial to downstream stability whether the interceptor is a drain or a seal.

As a final illustration, reaching similar conclusions, the sliding circle model was used, as indicated in *Figure 44.26*. After the priority optimization with regard to catastrophic mass downstream instability, under Design Principle 4 (DS4), further optimization on satisfactory operational behaviour of consequence (to DS) seeks to minimize the important change of conditions in moving from the well controlled end of construction (EC) situation, to the full reservoir condition on first filling. Analyses conducted on many possible failure circles confirm the indication of the single analysis shown, purposely made deep for most salient effects. Once again, the important driving influence subject to change is the pore water pressure thrust, u. Simple indices for evaluating optimizations are defined

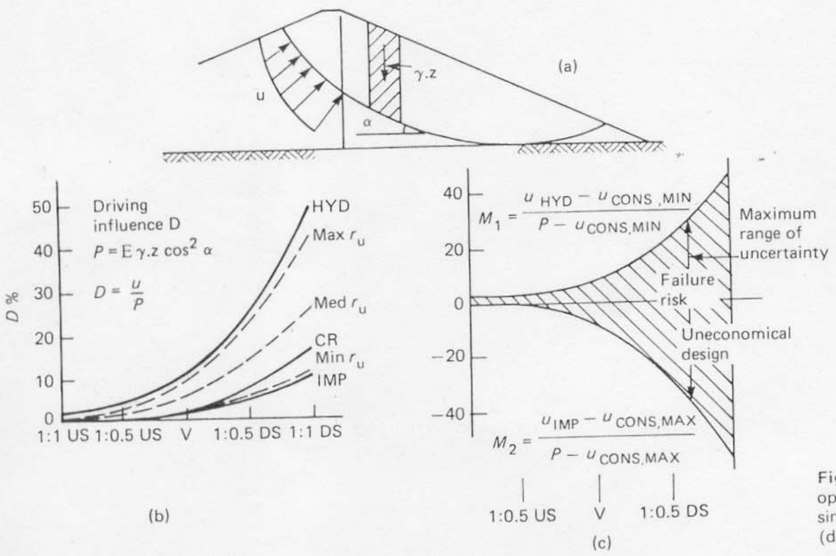


Figure 44.26 Confirming optimization of chimney position by simplified sliding circle analyses (de Mello[1])

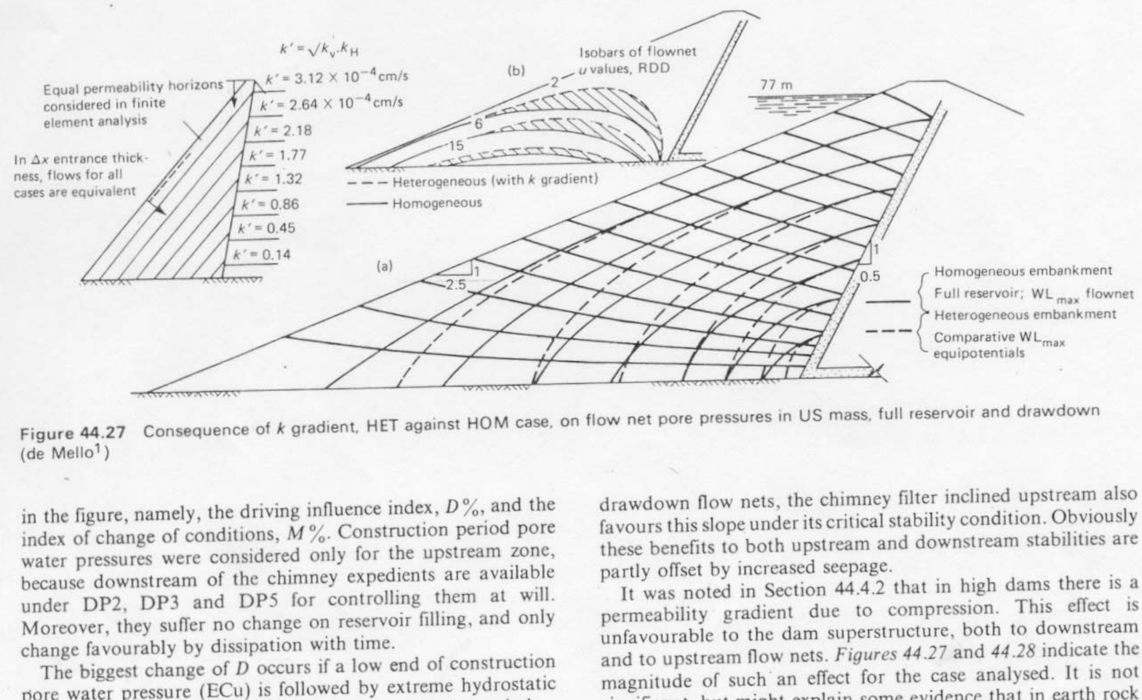


Figure 44.27 Consequence of k gradient, HET against HOM case, on flow net pore pressures in US mass, full reservoir and drawdown (de Mello[1])

in the figure, namely, the driving influence index, $D\%$, and the index of change of conditions, $M\%$. Construction period pore water pressures were considered only for the upstream zone, because downstream of the chimney expedients are available under DP2, DP3 and DP5 for controlling them at will. Moreover, they suffer no change on reservoir filling, and only change favourably by dissipation with time.

The biggest change of D occurs if a low end of construction pore water pressure (ECu) is followed by extreme hydrostatic reservoir loading (cracks). Such a major possible change index, M_1, reflects the worst risk of failure. At the other end lies the possible condition of change index, M_2, corresponding to a high ECu followed by average flow net conditions u_{imp}. If a design is dictated by the condition of high ECu (and negative value of M_2) an uneconomical condition prevails, because flatter slopes will have been required merely because of a transitory instability. The maximum range of uncertainty is represented by the span between the two limits of M. *Figure 4.26c* clearly shows that the chimney interceptor should not be inclined downstream.

Concomitant studies have demonstrated that for rapid drawdown flow nets, the chimney filter inclined upstream also favours this slope under its critical stability condition. Obviously these benefits to both upstream and downstream stabilities are partly offset by increased seepage.

It was noted in Section 44.4.2 that in high dams there is a permeability gradient due to compression. This effect is unfavourable to the dam superstructure, both to downstream and to upstream flow nets. *Figures 44.27* and *44.28* indicate the magnitude of such an effect for the case analysed. It is not significant, but might explain some evidence that in earth rock dams the loss of head concentrates mostly near the downstream 25% of core width.

In compensation for such a minor disadvantage, a definite advantage ensues if the design uses an optimized internal blanket for minimizing problems of foundation permeabilities. The internal impervious blanket, as illustrated in *Figure 44.29*, has self-evident merits of design concept, both for a homogeneous, HOM, permeability condition, and principally for the heterogeneous, HET, case. The priority need of grouting to seal larger erratic cracks in foundation rock is not discussed in this

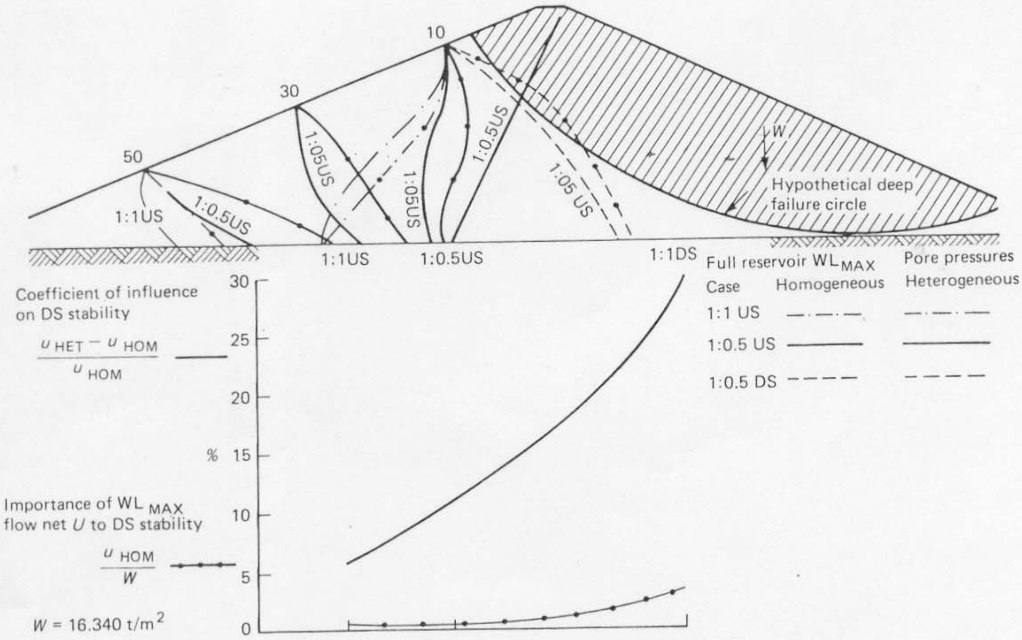


Figure 44.28 Comparative consequence of permeability gradient on DS stability (de Mello[1])

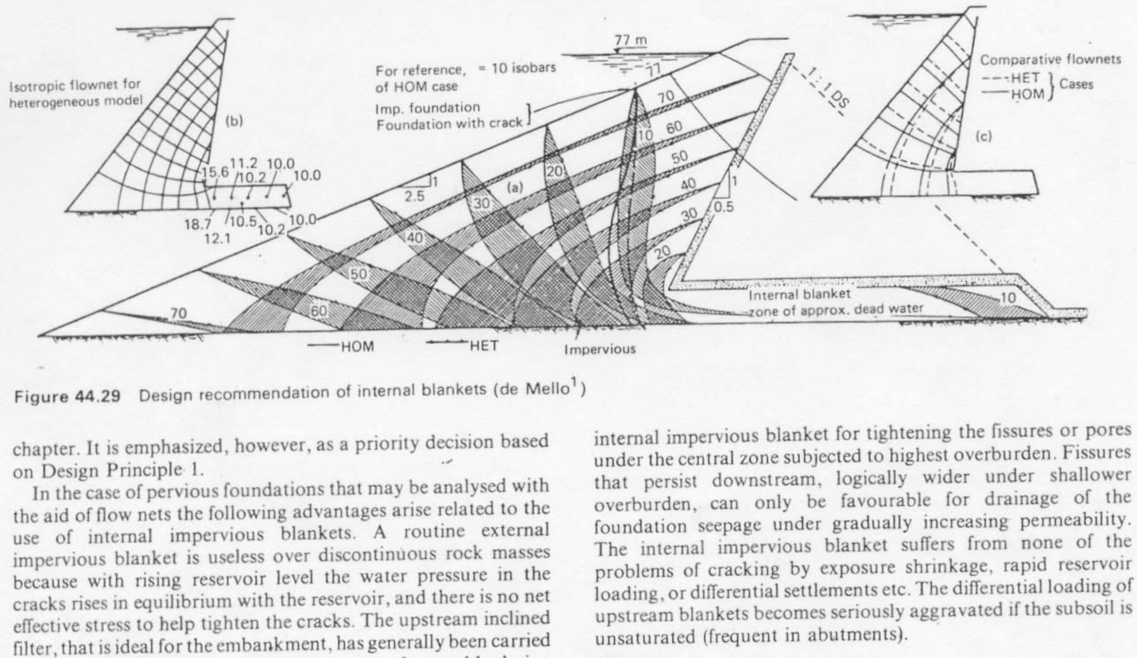


Figure 44.29 Design recommendation of internal blankets (de Mello[1])

chapter. It is emphasized, however, as a priority decision based on Design Principle 1.

In the case of pervious foundations that may be analysed with the aid of flow nets the following advantages arise related to the use of internal impervious blankets. A routine external impervious blanket is useless over discontinuous rock masses because with rising reservoir level the water pressure in the cracks rises in equilibrium with the reservoir, and there is no net effective stress to help tighten the cracks. The upstream inclined filter, that is ideal for the embankment, has generally been carried to the foundation. This constitutes a very unfavourable design decision, shortening the pervious seepage paths of the foundation and heavily taxing both the unfavourable seepage exit gradients into the filter, and the flow carried by the drainage blanket.

Moreover, no matter what the foundation conditions, since the control of uplift is only required downstream of the 1:1 downstream position, every benefit must be extracted from the internal impervious blanket for tightening the fissures or pores under the central zone subjected to highest overburden. Fissures that persist downstream, logically wider under shallower overburden, can only be favourable for drainage of the foundation seepage under gradually increasing permeability. The internal impervious blanket suffers from none of the problems of cracking by exposure shrinkage, rapid reservoir loading, or differential settlements etc. The differential loading of upstream blankets becomes seriously aggravated if the subsoil is unsaturated (frequent in abutments).

44.6 Sliding stability conditions for embankment dams

Conventional stability analyses of the slopes of embankment dams have been and are routinely conducted by the analytical methods of limit equilibrium of sliding bodies (of circular or composite shapes) applied to all slopes. These methods of computation are well established as working tools of geotechnical engineers. However, it is necessary to emphasize that the case of dams requires much higher assurance against any probability of sliding failure, and therefore the very concepts of factors of safety and of indices of satisfactory behaviour must be re-examined.

44.6.1 Fundamental principles of slope design and of statistical definition of acceptable indices of behaviour

Some fundamental principles of good design (see de Mello[1]), particularly relevant in embankment dam engineering are summarized as follows. First, the principle of the pretest enforces construction conditions which are more critical than the operative ones. Secondly, the principle of gradual changes in conditions seeks to avoid rapid changes of conditions disproportionate with experience and/or the *status quo*. Thirdly, the all-important aim is that foreseeable changes with time, however minute, should be favourable. The net consequence is that in good design often a stability computation under presumed critical operational conditions may have been consciously made unnecessary; that is when wisdom supersedes knowledge. However, collated repetitive data on plausible indices of slope behaviour are still lacking, and so also are indices of acceptability of such behaviour.

By convention the Factor of Safety (FS) is defined as the ratio of:

$$\frac{\text{Predicted Resistance, } R(\pm e_r)}{\text{Predicted Stresses, } S(\pm e_s)}$$

where $\pm e$ = dispersion.

If during construction satisfactory stable (elastic?) behaviour is established up to a given stress level, S_c (c = construction), then deterministically $R \gtrless (FS)R_c$. Now, if it is ensured that $S_c \gtrless S_o$ (o = operational time) and it is reasonable to anticipate that $R_o \gtrless R_c$, then the ratio $R_o(\pm e_r)/S_o(\pm e_r)$ cannot be used as a value of FS. The author therefore proposes that a different nominal factor of safety should be referred to as a Factor of Guarantee, GF. Obviously $FG \ll FS$ may be accepted without risk of dissatisfaction (e.g. $FG = 1.1$ might well prove satisfactory in a material and condition that would require $FS = 1.5$).

Whereas soil engineering has generally considered only one definition of Factor of Safety, *Figure 44.30* suggests the need to recognize three distinct Factors (considering only the differentiation of statistical dispersion around resistances, R, without any further delving into the histograms of acting stresses, S). The condition of a Factor of Guarantee is that wherein, by some lower rejection criterion the engineer would be assured that the histogram of resistances can only be higher than some value already pretested and guaranteed. Obviously a value $FG = 1.5$ provides much greater assurance of success than $FS = 1.5$. The concepts can be illustrated with reference to examples of piles and tunnelling in soft ground.

A pile jacked down under 60 tonnes to absolute stoppage of penetration/settlement has a $FG = 2$ if used for a working load of 30 tonnes. If the resistance is predicted as 60 tonnes it has the conventional $FS = 2$. Setting aside the discussions on dynamic versus static resistances of piles and cases of sensitive clays, driven piles checked by refusal observations can imply factors FG. By contrast, a bored pile suffers from two disadvantages in its load-settlement behaviour. Firstly, it would not have been pretested, and therefore it should be affected by the FS (poorer than FG).

Secondly, upon closer examination it could be argued that it is even worse than that. All efforts of advancement of soil mechanics are towards minimizing sampling and testing

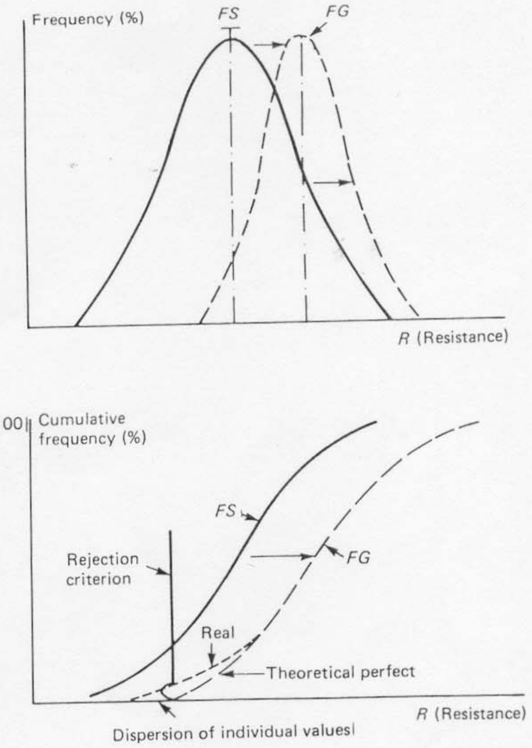
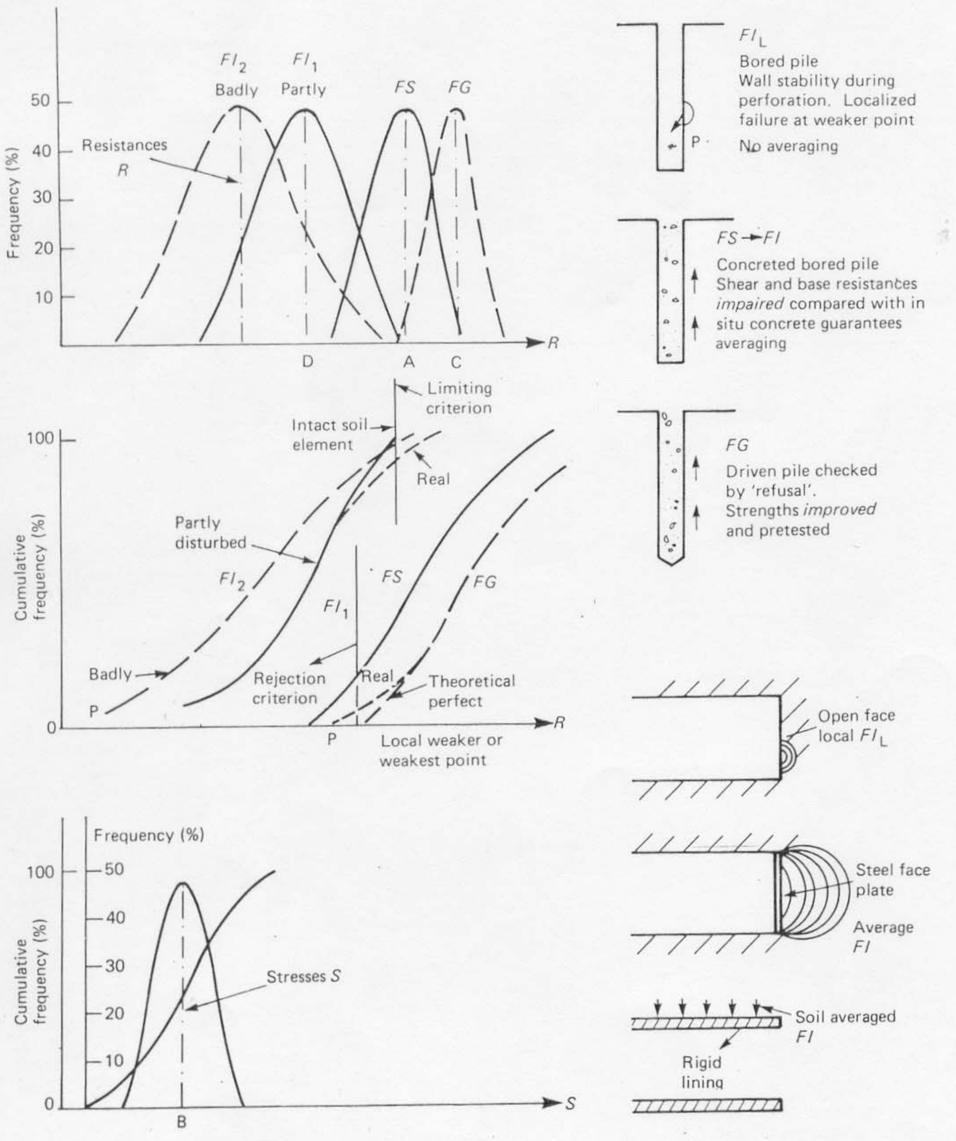


Figure 44.30 Proposed distinction between Factor of Safety (FS) and Factor of Guarantee (FG) (de Mello[2])

disturbances, and towards improving determination of the values of *in situ* soil parameters (see de Mello[17]). In reality the assessed intact parameters would establish an upper rejection criterion, since the soil affecting bored pile behaviour represents a histogram of resistances always lower, to varying degrees, truncated at the upper value. This is a situation diametrically opposite to that of FG, with the lower rejection criterion. Hence another ratio of averages (Resistances/Stresses), namely, a Factor of Insurance, FI, can be introduced. Insurance is against something essentially inevitable, the effect of which should be minimized.

The basic fact is that $FI < FS < FG$ and depending on the dispersion of the histograms the differences may be significant. If projects continue to be designed generally for (nominal) $FS = 1.5$ without recognition of this difference, all structures in which FI is at stake will record a greater degree of trouble, while structures in which FG is at stake will incorporate an unnecessarily higher degree of safety. Tunnels and bored piles involve execution effects that give rise to deterioration of *in situ* parameters (resistance, deformation) and therefore involve FI conditions. In the case of dams, if flow nets alter significantly the stability conditions of the downstream zone upon first filling of the reservoir, it may be inviting FS conditions rather than the desirable FG situation of pretested behaviour. Moreover, if the long-term flow nets generate uplifts or tensile stresses, which can only reduce the strengths achieved it may be inviting even worse conditions associated with FI. This should raise concern in some overconsolidated clays subjected to swelling.

There is yet another point to emphasize, namely, the distinction between conditions which involve averages (as



Relations: $FS = \frac{A}{B}$ (Safety factor)

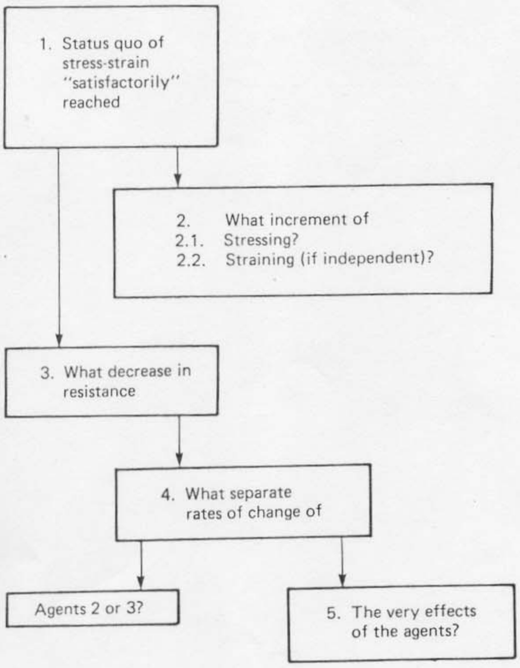
$FG = \frac{C}{B}$ (Guarantee factor)

$FI = \frac{D}{B}$ (Insurance factor)

Figure 44.31 Suggested distinctions in 'Factors of Safety' (de Mello[2])

above), and those that involve localized situations corresponding to somewhere along the tails of histograms. In other words, confidence limits and factors of safety might be related to individual events on the histogram (or fractiles) rather than on the median. Such is, for instance, the situation where unstable localized pockets occur along a bentonite-stabilized bored pile before concreting. After concreting, the rigidity of the concrete guarantees averaging over the profile. Similarly unstable conditions may be encountered when excavating a tunnel. The behaviour behind a steel face-plate in shield tunnelling, or around a lining, can be averaged, which implies an inevitable benefit in comparison with localized worse conditions. In the case of dams instability of sliding masses can generally be treated on the basis of averages, unless there is a discontinuity of shear weakness, no matter how thin.

In reasoning and testing under a complete stress–strain–time path trajectory the following must be considered:



From the above note that internal stresses (1) are frequently different from the simply adopted geostatic assumptions of early soil mechanics. Moreover, straining (2.2) is sometimes quite independent of load-stressing (e.g. collapse of structure). Further, the onset of failures can be due to any of the agents (2) and (3). Finally, what matters is not merely the rate of onset (4) of the agents (2, 3), but also the rate of onset of the effect (5) of the agent, since it is well known (e.g. viscous and other complex rheologies) that the rates of causes and rates of effects are not similar.

Thus in estimating pore water pressure development along a potential sliding plane due to a change of flow net pore water pressures, it is not sufficient to consider, for the effective stress analysis, the u value corresponding to the new flow net. One should consider the u value as composed of two parts. The first is the hydrodynamic flow net pore water pressure; and the second is an incremental transient (positive or negative) pore water pressure due to tendencies to variation of volume (ΔV) that would accompany the incremental straining (normal and shearing). This $\Delta u = f(\Delta V)$ depends on an estimate of the incremental stresses anticipated (always assumed on the pessimistic side). It depends also on estimates of the anticipated rates of change of stresses, and principally of strains. The latter important consideration is why liquefaction slides and mud flows are indicated when the stress–strain curves show a sharp post-peak drop, and the incremental shearing is highly contractive.

By recognizing the above, it merely emphasizes how nominal are procedures of sampling–testing–computing (stability). They will always continue to be so in engineering endeavours. That is why the author (see de Mello[1]) proposed an operational Satisfaction Index, SI, for assessing the behaviour of slopes associated with different FS (and/or FG) values. The important thing is to use in each statistical analysis (same embankment) several different slopes, to accumulate theoretical observational data (e.g. on plastic incremental movements compared with pseudo-elastic stable reference values). The object is to collect hundreds, thousands of such pairs of data (SI versus FG) so as to establish the necessary histograms. Then one will be in a position to make rational computed and economic decisions.

Statisticians conversant with mathematics will develop the relationships between the conventional FS and the proposed additional factors FG and FI as functions of the histogram truncations (see de Mello[18]). In good dam design, if consequences of risk are high, then the engineer should be dealing with FG conditions. Thus, in much of the following text the discussion will be limited to FG situations. Great care is recommended because FS conditions may be at play in most cases of conventional designs. If so the corresponding computed value for satisfaction must be decidedly higher than if FG conditions were guaranteed (under pretested situations).

44.6.2 Slope stability in dumped and compacted rockfill

The subject is briefly discussed (see de Mello[2]) to emphasize that:

(a) Infinite slope analysis given by $FS = \tan\phi'/\tan i$ is an extremely conservative lower bound and could well accept a $FG = 1.00+$.
(b) Stability is automatically self-tested as the fill rises at its constant slope (i.e. one is dealing with FG and not FS).
(c) There are advantages of deterministic $u = 0$ to permit very low FG.
(d) There should be advantages of locked-in prestress (in crushed angular contacts) whereby greater stability than implied by conventional computations can be expected.

The following additional facts are submitted, summarizing a good number of observations on rockfill and corresponding aggregate stockpiles (heights 30–45 m) of very large projects. Field observations are important for complementing indications from laboratory tests and conventional stability analyses.

1. The stable slopes (angles of repose) are surveyed in detail in minimum stretches involving more than about 15 large-size rocks. The histogram for the end-pushed loose rock may be considered conditioned by the most unstable surface rock which has stopped after moving (*Figure 44.32*).
2. In comparison, the excavated slopes show two distinct trends, a steep stretch (even partly subvertical), dominated by the more stable rocks having to be moved out of their interlocked positions of rest (static versus dynamic friction?), and the lower stretch comprising mixed excavation-slope and rolled-slope material.
3. In comparison with smaller granular material, one deals with a histogram that is not so tight as that derived from laboratory tests on uniform sands.
4. However, even in loose end-dumped angular rock stockpiles there is a definite strength gain from prestress.
5. The rhetorical question posed is: which ϕ' value should be used in nominal stability analyses, that of slopes of fills, or that of stable slopes excavated from the bottom, after benefit of prestress?
6. Considering the very significant prestress contributed by compaction of rockfill in lifts, how much steeper can one go without any risk of unsatisfactory behaviour?

44.6.3 Construction period stability, clayey materials

Under modern heavy earthwork equipment the conditioning factor is trafficability, and in any well designed dam having a chimney filter it should be difficult to generate end-of-construction instability in well compacted material. There is an effective stress preconsolidation cohesion for the short-term

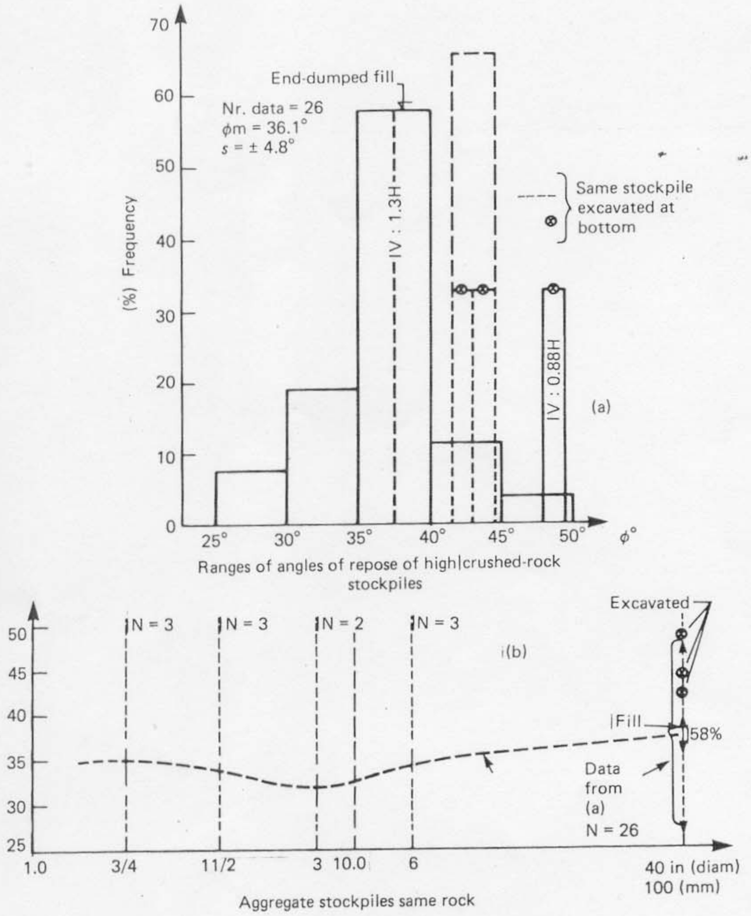


Figure 44.32 Behaviour of angular rock slopes at FS≈1.00 (de Mello[2])

condition considered. Moreover, there is generally an initial suction, and not the presumed u_c versus γz (c = construction) conditions suggested by the early (and many recent but faulty) laboratory tests and field observations (see da Cruz and Massad[20]; Gould[21]). Finally, the constant r_u or $\bar{B}$ coefficient assumed for early simplifying computations is now unnecessary, and misleading with regard to greater instability (see de Mello[22]) for shallower circles (benefit has been gained by suction and cohesion) (*Figure 44.33*).

In the zone downstream of the chimney, affecting the all-important downstream stability, it is preferable to generate some u_c in order to assure onself of the pretest principle and satisfactory *FG*, and especially its rigorous improvement with u_c dissipation with time. Should any u_c develop higher than desired, one may use the intermittent u_c-ADJUSTERS, comprising dry layers (functioning as 'blotting-paper' non-exiting filters), without fear of layered permeabilities (see de Mello[1]). The same expedient also may be used in the upstream zone of the dam, excluding what would be equivalent to a 'core'. In the core zone one must avoid unfavourable permeability $k_h \gg k_V$ effects on the flow net, and it is desirable to have a high u_c, preferably close to the u_{fn}(fn = flow net, full reservoir) so as to minimize the change of conditions in the core on first filling.

As has been often demonstrated, a core may have quite high construction pressures without impairing end-of-construction upstream slope stability. Stability analyses can be run, using appropriate estimates of internal stresses, cohesion, negative and positive u_c, and the effective stress envelope. Acceptable (and a desirable design aim) *FS* should be close to 1.3 for judicious pretesting. The analyses are merely to facilitate Bayesian insertion of successive u_c observations for continually improved assessment of *FS*, while deformation measurements furnish indications on SI (satisfaction index).

Since one cannot hope for a coincidence of achieving $u_c \approx u_{fn}$ it is worth discussing in which direction the tolerance should be more favourable, $u_c > u_{fn}$ or $u_c < u_{fn}$. The ideal situation would be to have had u_c developed to slightly higher than u_{fn} but dissipated to a value slightly lower than the u_{fn}. Thereupon the soil will have been pretested to values of u higher than necessary, and will be behaving within the precompressed or preconsolidated range wherein changes of void ratio and of behaviour with change of stress are small.

44.6.4 Downstream stability, full reservoir

First filling

The interesting problems of rapid versus slow filling will not be discussed herein. A traditional critical permanent flow net is assumed initially. This would appear to be the all-important case

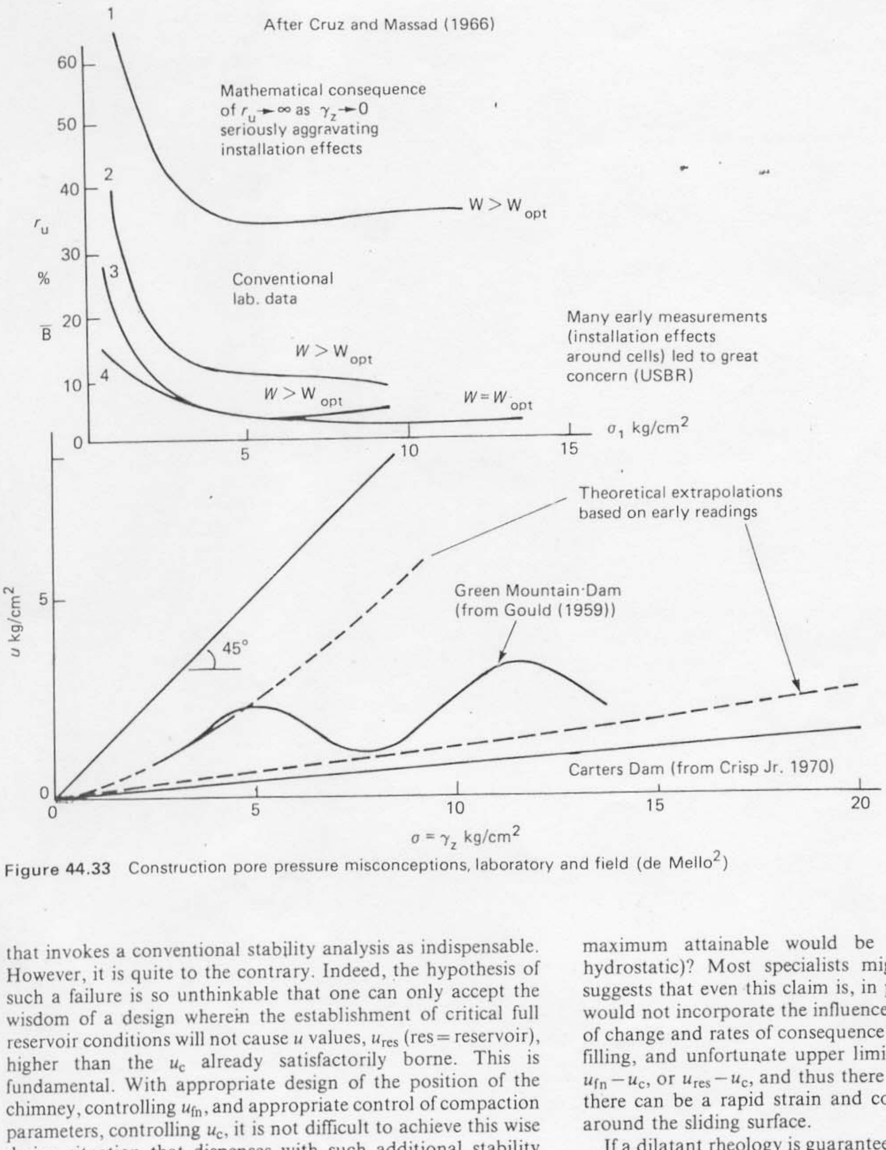


Figure 44.33 Construction pore pressure misconceptions, laboratory and field (de Mello[2])

that invokes a conventional stability analysis as indispensable. However, it is quite to the contrary. Indeed, the hypothesis of such a failure is so unthinkable that one can only accept the wisdom of a design wherein the establishment of critical full reservoir conditions will not cause u values, u_{res} (res = reservoir), higher than the u_c already satisfactorily borne. This is fundamental. With appropriate design of the position of the chimney, controlling u_{fn}, and appropriate control of compaction parameters, controlling u_c, it is not difficult to achieve this wise design situation that dispenses with such additional stability calculation. With $\Delta u = u_{res} - u_c$ = negative (modestly) the change of stability from end-of-construction to first filling can only increase (ΔFG = positive). (Note: There are other conditions that may similarly be reasoned to be satisfactory.)

Many a dam has behaved satisfactorily without any inkling of such principles. However, absence of evidence is not evidence of absence. The only way to guarantee zero probability of downstream sliding failure is to have pretested the $FG > 1^{++}$, and to have ΔFG + ve. Any number of design sections and conditions may be rapidly sketched showing how to compare u_c versus maximum possible u_{res}. It is a simple exercise, and causes no qualms of decision.

What is the most unfavourable u_{res} possible, the flow net hypothesis u_{fn}? Under which hypotheses should this be considered, as all are highly dependent themselves on other hypotheses? Would it then be right to assert that the very maximum attainable would be the full reservoir u_{hyd} = hydrostatic)? Most specialists might claim so. The author suggests that even this claim is, in principle, wrong, because it would not incorporate the influence of compressibility, its rates of change and rates of consequence thereof. If a rapid reservoir filling, and unfortunate upper limit, leads to a positive $\Delta u = u_{fn} - u_c$, or $u_{res} - u_c$, and thus there is a rapid drop in stability, there can be a rapid strain and consequent ΔV due to shear around the sliding surface.

If a dilatant rheology is guaranteed, then security is obtained. But if there is a tendency to compression, then there is an increase Δu_s (incremental excess pore water pressure due to contractive tendencies in shearing, s) such that the real maximum u involved in the sliding stability could be $u_{res} + \Delta u_s$. It is imperative to seek the right geomechanical and rheological statistical population that may dispense with the conventional full reservoir stability analysis, by assurance of better known FG analyses of truncated histogram of changes of conditions.

Long-term stability

For stability of a downstream slope in long-term conditions one must, again, inquire what changes of loadings and resistances can tend to occur to affect the already established first filling FG. It is really absurd to think of a slope analysis 'from scratch', because of the much greater probabilistic imprecision than by

Bayesian analysis of subsequent probabilities as superposed on the former. There is no difference in principle in using the probability changes of u from $u_{res\,ff}$ (ff = first filling) to $u_{res\,lt}$ (lt = long-term), in similar manner to the suggested progressive adjustment of u_c as a quantification of the Observational Method.

Whether or not u_{res} would tend to increase with time depends on how consolidation, secondary compression, tensile cracking, etc. would change relative permeabilities. It must be planned for strength to increase, and u to decrease, with time. Both trends are associated with a tendency towards compression (desirability modest). Strength may further gain from favourable cementing and thixotropic effects. It is not difficult to design to guarantee such compression, such that after $FG_{res\,ff} > 1^{++}$ a ΔFG + ve is guaranteed.

The fear of long-term instability in natural slopes has unduly influenced dam design. Whereas long-term destabilization is in principle inevitable in a natural slope at $FS \approx 1.00$, in a good dam design of $FG \gg 1.00^{++}$ and +ve ΔFG, instability should be avoidable by conscious action. Much depends on the shear strains and brittle stress–strain, and strain rates. One must judiciously appraise the rheological changes, $\Delta u = f$ (Δtime, Δstrain, etc.), $\Delta s = f$ (Δtime, Δstrain, etc.), and so on.

One loading condition that thwarts the postulated simplicity is the seismic one. Although this subject will not be considered here, it is emphasized that the probabilities of a catastrophic seismic event cannot be considered independently of the probabilities of occurrences of smaller seisms. Except for the extreme event of the very first seism being that of maximum credible intensity, the occurrences of smaller events could help accumulate improvement by successive cyclic compression. The ideal rheology would aim at a slightly dilative instantaneous behaviour for intensities higher than some moderately rare recurrence level.

44.6.5 Upstream slope, instantaneous drawdown

This is a topic in which both the theory, and the conventional practices, appear questionable. As a result, it appears that upstream earth slopes are significantly overdesigned, and any eventual sliding is limited to shallow scoop circles.

Conventional design has often applied reduced FS requirements (e.g. $FS > 1.1$) principally by reasoning that drawdown is never instantaneous and some drainage lowering of the phreatic surface would occur. This effect, however, is minimal. Although FS or even FI conditions (and not FG) prevail by reservoir soaking/saturation, the modest consequence of eventual failure justifies lesser FS values.

The very rare prototype observations cannot be multiplied in reservoir utility operation, and would never chance to be relevant for the sliding surface. Principal intuitions derive from reservoirs suddenly emptied by failures (e.g. by overtopping) after many years of full reservoir operations. The resultant failed eroded transverse upstream slope has mostly been very steep. The designed upstream slopes have not budged perceptibly.

Following the design dictum of examining critical bound hypotheses, the author limits himself to considering the hypothetical absolutely saturated embankment. In modest dams and/or shallow upper sliding circles, the compacted material should not be saturated. One readily recognizes this from laboratory data of the high back pressures (6–12 kg/cm^2) necessary to saturate triaxial specimens, especially if the air-filled micropores are first reduced in diameter by consolidation under confining stresses. The principles summarized below merely consider tendencies to change in σ' (assuming saturated incompressible pore fluid). In a generalized extension, tendencies to change both of σ' and of u will have to be considered (besides, of course, the incremental shear stress and strain rate Δu_s, and any other complications such as rotations of principal stresses, etc.).

The present questionable concepts may be summarized as follows:

(a) The division could not possibly be between:

'FREELY-DRAINING FILLS'	v	'COMPRESSIBLE FILLS'
(Terzaghi[23], accepting flow nets, rapid drawdown RDD)		(Pore pressures generated due to total $\Delta\sigma$ on dam face and r_u concept, $r_u \approx 1.0$) (e.g. Bishop[24])

Obviously draining versus non-draining has no obligation to any assumption on compresibilities. Likewise, incompressible versus compressible has no obligation of direct association with drainability or rates of drainage.

(b) Except in extremely differentiated materials, there is no black–white distinction of intrinsic qualities of draining versus undraining, compressible versus incompressible, as regards materials and conditions thereof. First, no material possesses a homogeneous tendency (to compress, to drain, or whatever) over the entire upstream body of a dam. Secondly, compressibility (etc.) is a temporary behaviour problem and not an intrinsic quality. A material will not behave as compressible (even if it is generally or frequently compressible) when it is subjected to a stress release. A soil element in one part of the critical circle may be subjected to stress release, and want to behave dilatantly, while another similar soil element in another part of the same circle may be subjected to a stress increment, and want to compress.

As a curious extreme example to emphasize the conceptual point one could say that in a material that is (intrinsically) homogeneous and pervious, functioning under a flow net, each flow line (surface) behaves as (temporary behaviour) impervious, since no water from one side or other of the surface crosses it (it is immaterial that it does not wish to cross the line). All materials are compressible, pervious, etc.; it is a question, firstly, of algebra (tendency to expand versus tendency to compress) and, secondly, of degree (compress more, or less).

(c) Thirdly, there are some over-simplifications in the $\Delta\sigma$ and r_u or $\bar{B}$ procedure, that in no way permit incorporating the advantages (or disadvantages) of the most fundamental design weapon of a good dam design, which is the chimney filter and the drainage (u-controlling) features (be they exiting or non-exiting drains). The degree of significance of such consequences varies considerably in different design cross-sections.

The principle proposed is simple, and consistent with the hypothesis of perfectly saturated (incompressible) pore fluid. The tendency to change of a flow net is 'instantaneous', and what matters are instantaneous pore water pressure changes (compare with rheological model of the Terzaghi consolidation theory). Therefore two flow nets can be drawn, WL_{max} (maximum water level) and RDD. The positions of the drainage features are duly incorporated. Once these internal flow net pore water pressure conditions are established, based on the rapidly changed boundary conditions, one can check what would be the tendencies to change $\Delta\sigma'_v$ that determine which soil elements would wish to behave as compressible. Any tendency to compression, obviously, should generate a corresponding increased transient pore water pressure (due to ΔV). The stability analysis should be based on the RDD flow net (valid for incompressible pores) complemented (at worst) by the Δu due to the compressive $\Delta\sigma'$ (and any consequent shearing ΔV). The principle is that routine flow nets presume incompressible pores and incompressible fluid. Therefore, one checks first what would be the tendency, to compress or to expand, in an instantaneously incompressible assumption. After this it can be decided whether

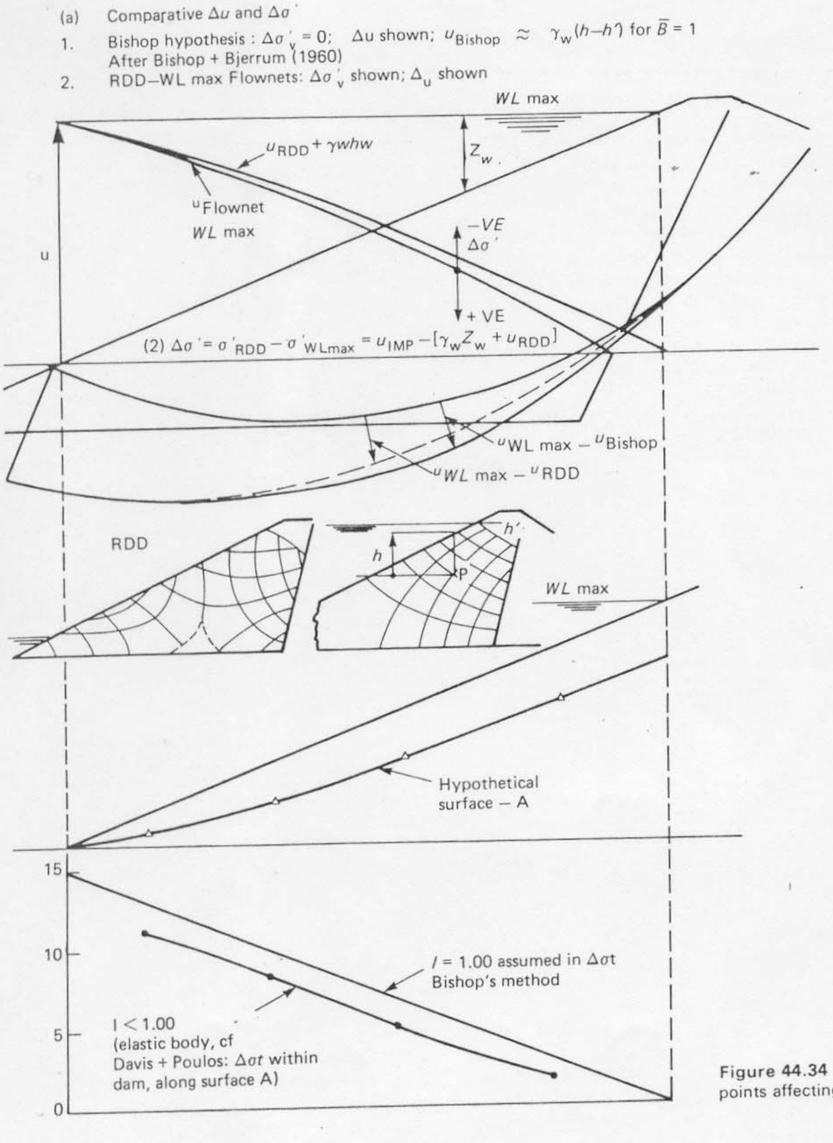


Figure 44.34 Discussions regarding important points affecting US slope RDD analysis (de Mello[2])

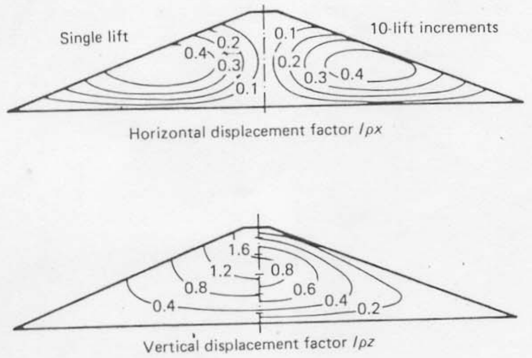
the material will behave as compressible or as dilatant, but in any case superposed on the background of the saturated instantaneous RDD flow net. *Figure 44.34* exemplifies this principle of stability analysis that is generally applicable.

As usual, instability is best analysed by way of changes of conditions superposed on the earlier proven slope obliquities of stress under WL_{max}. Incidentally, the occurrence of frequent partial pool drawdowns merits analysis concerning any cumulative trend toward compression or expansion, and consequent strength changes. The ideal design of cross-section and compacted material would aim at slight compressions under the frequent smaller drawdown episodes, and, hopefully, a tendency to dilate under the most critical drawdown. Thereupon, if construction u_c had been satisfactorily high in comparison with the new $u_{RDD}+\Delta u$, and upstream construction-period stability well established, one could deal with a pretested slope stability and an acceptable $FG \approx 1^+$ situation.

44.6.6 Conclusions

Recommendations are made that stability analyses be used as furnishing nominal *FS* (or preferably *FG*) values and that the acceptance levels of such index values be established by a great number of non-failure situations, treated statistically. For more critical conditions the only satisfactory approach for guarantee of design is to consider changes of conditions, starting from prestressed conditions, preferably more critical than the operational ones.

In a manner similar to the evolution of foundation design as it



$$V_x \text{ or } \rho_x = 14.82 \frac{\gamma H}{E} I\rho_x$$

$$V_z \text{ or } \rho_z = 14.82 \frac{\gamma H}{E} I\rho_z$$

(See : Poulos and Davis, 1974)

Figure 44.35 Displacement factor contours for standard embankment (after Clough and Woodward[27])

moved from discussing *FS* with regard to bearing capacity formulae, to the more fruitful approach of aiming at limiting deformation for avoiding minor damage, the field of dam slope design will only move significantly forward when such countless statistical data are collected, correlating *FS* or *FG* with Satisfaction Indices *SI* (see de Mello[1]).

44.7 Stresses and strains within the embankment

44.7.1 General

Knowledge of the distribution of stresses and strains within the embankment is of obvious importance for proper analysis of all aspects of its behaviour. First, it is necessary for estimation of the construction period pore water pressures. Secondly, it is of importance for evaluation of differential strains and secondary stresses tending to create hang-up of the core, cracking, hydraulic fracturing, and so forth. It is on such items that rests most of the assurance of adequate behaviour of a dam (see UNAM[25], NGI[26]). Finally, it is necessary for predicting post-construction long-term deformations and adjustments.

The problem is obviously three-dimensional, but for obvious historical reasons has been tackled initially as two-dimensional, for the upstream–downstream plane. Thereupon, up to the present, the vast majority of analyses continue to be two-dimensional, and this has been considered justifiable for dams across wide valleys. The three-dimensional analysis has been restricted to a few cases of narrow V-shaped valleys.

The upstream–downstream plane has received most attention because of seepage flow nets and slope stability problems (let us say, involving the plane of σ_1, and σ_3). Assuming, however, that such problems are adequately equated, the principal interest regarding transverse cracking should be focused on the σ_2 plane.

44.7.2 Routine idealized computations

The simplest possible method of calculation has been based on homogeneous embankments modelled as perfectly elastic, and the self-weight due to rising fill being applied by layered increments. *Figure 44.35* reproduces typical results on vertical and horizontal displacement factors from the now classic paper by Clough and Woodward[27]. The same computations also yielded diagrams of stress factor contours and strain contours that are reproduced in all publications on embankments.

In a similar vein, *Figure 44.36* reproduces the vertical displacements computed by Booker and Poulos[28] for an idealized reservoir loading on an earth–rock embankment, assuming water pressure as a boundary pressure on an impervious membrane. Such a hypothesis is acceptable in a transient condition for very rapid reservoir loading and unsaturated core. The two diagrams indicate comparative conditions of varying ratios of core to rockfill moduli E_c and E_R.

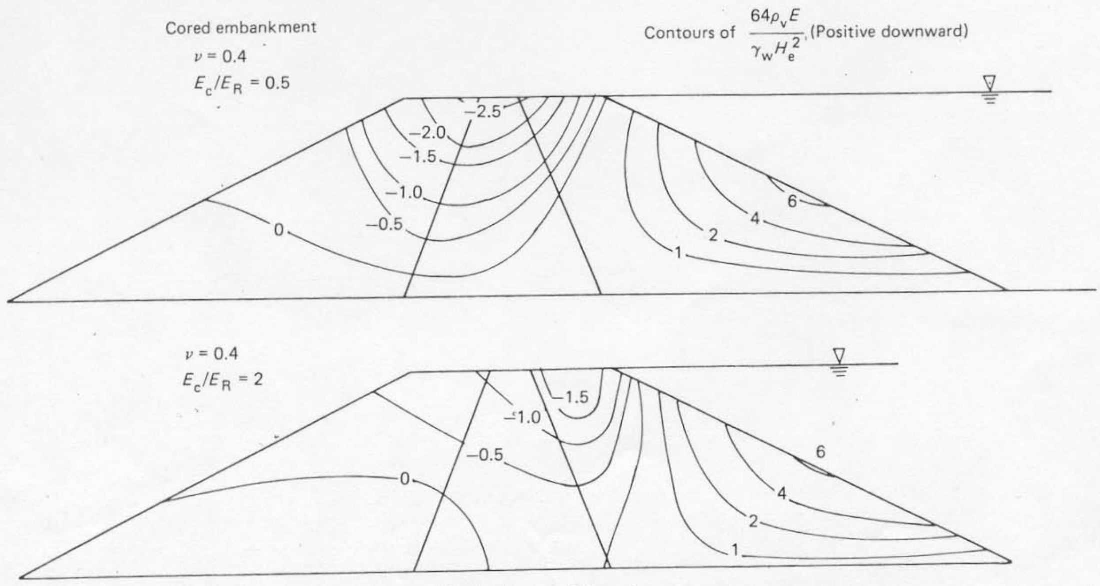


Figure 44.36 Contours of vertical movement ρ_v cored embankment. Full pool

A more realistic manner of considering reservoir loading is through seepage effective stresses, for example, from flow nets. Note that flow nets depend on starting hypotheses and suffer changes because of secondary strains introduced by the seepage. Therefore, calculations have to be iterative to reach final, nominally correct, adjustment.

Other complex situations have been, and are repeatedly being, analysed by appropriate numerical methods (finite element analyses, etc.), and employ different stress–strain constitutive equations. One commends the initiatives of such analyses, to accompany concomitant information from judicious monitoring of prototypes. At the present stage of knowledge, however, many factors intervene in making complex calculations less meaningful and profitable than they would seem.

Satisfactory preliminary computations and indications may be derived from the elastic homogeneous model (see Poulos and Davis[29]), with moduli altered by steps on the basis of trends derived from other projects; incidentally, since prototype observations are analysed in like manner, the cycle closes. The principal criticism lies in the hypothesis of a homogeneous embankment, with core and shell equivalent. However, this is a starting hypothesis, and as soon as computations progress, differences come to the fore. There are two notably divergent avenues to follow. For the design engineer, the strong preference is to adjust the materials and zones as far as possible aiming for the desired homogeneity of displacements. For the scientifically inclined geotechnician, the inexorable need is to resort to finite element analyses, as refined as possible, to follow the consequences presumed predictable, within the undesired physical model that exposed the differential behaviour.

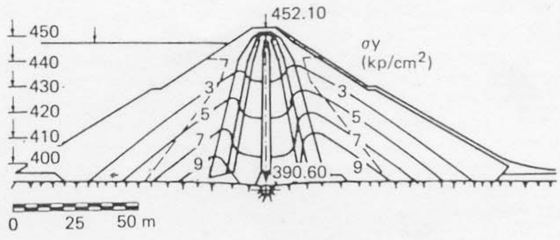
Figure 44.37 gives some typical observations from some of the more fully instrumented dams.

44.7.3 Some practical indications from monitored dams

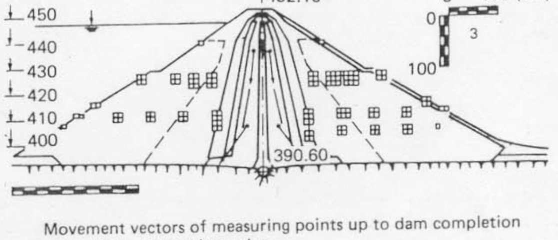
For the sake of design it is of interest to extract some practical conclusions from comparisons of predicted as opposed to observed settlements and displacements. As already mentioned, construction period settlements in very clayey compacted materials are much overestimated by laboratory tests, more so in remoulded specimens than in undisturbed block samples (*Figure 44.38*). Also, all materials reveal compaction preconsolidation pressures, as shown in *Figures 44.38* and *44.39*. In the latter case, one must note that most early indications from monitored rockfill dams did not include an influence factor, I_f, in analysing field data. The use of an influence factor alongside the pragmatic technique of plotting data in semilog plots accentuates indication of a compaction precompression in clean sound rockfills (*Figure 44.39*). It will be of interest to check if for much higher dams there are significant benefits from heavier compaction.

A number of dams have been analysed in *Figure 44.40* in order to extract an approximate ratio of observed to predicted settlements, using for such predictions the statistical regressions discussed in Section 44.4.2. It can be seen that very frequently self-weight settlements are of the order of one half to one third of the predicted values (*Figure 44.40*(*a*) and (*b*)).

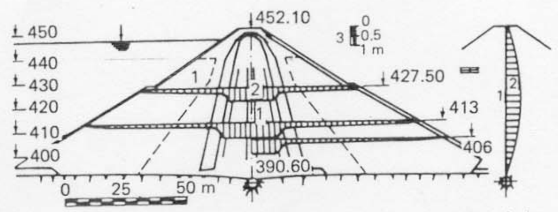
In *Figure 44.40*(*b*) the comparative data from three dams is so closely similar as to define a prototype model. For their very clayey cores of $\rho_{d\,max}=1.4\,t/m^3$, the discrepancies between observed and predicted internal settlements increased greatly with increased height. *Figure 44.40*(*c*) includes data from the 150 m Emborcacao Dam (see de Mello[6]) and indicates better agreement of predicted with observed settlements, which would call for analysis of possible stress redistributions between core and shells.



Vertical earth pressures in the dam cross section prior to impoundment

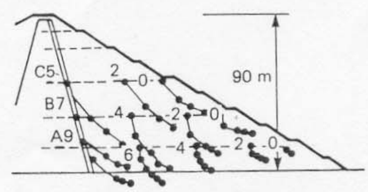


Movement vectors of measuring points up to dam completion
(1) Measuring point
(2) Movement vector
(3) Scale of deformations



Settlements in different measuring horizons up to dam completion
(1) Measuring point
(2) Settlement
(3) Scale of settlements

(a) Mauthaus dam (German Federal Republic)



(b) Llyn Brianne dam (Great Britain)

Figure 44.37 Typical observed strains and stresses from earth–rock dams more fully instrumented

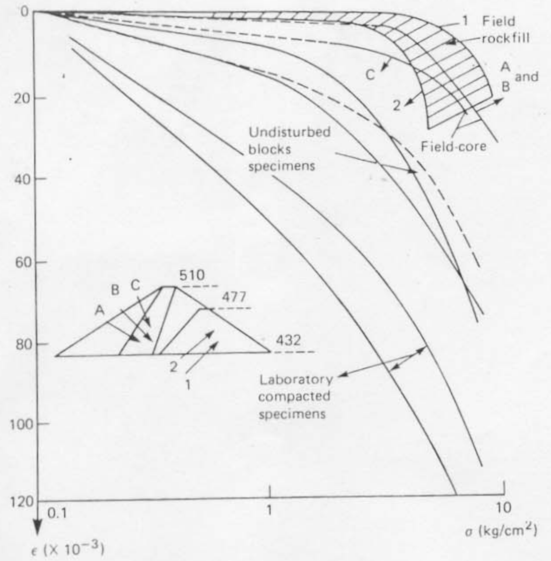


Figure 44.38 Comparative data, Salto Santiago Dam, laboratory and field

44.7.4 Quality control of construction

A prime aim of quality control of construction is the guarantee of achieving statistical homogeneity of each material as constructed. Engineering design in inconceivable without the premiss of fabricated homogeneity.

The early discussions on preferences between method specifications and end-product specifications have led to an understandable compromise, since method specifications can only be applied when, in closely similar materials and construction conditions, it has been proved that the end-products were satisfactory. Thus the present concept favours end-product specifications, aiming at the product desired by the designer. After some initial field adjustments, confirmed by inspection testing and all important visual-tactile inspection, it is obviously preferable to rely predominantly on closely controlled construction methods, assisted by spot-checking by end-product testing. Indiscriminate use of method specifications in relatively new materials and conditions has been responsible for many surprises and even failures.

The subject of construction specifications, inspection and quality control testing involves a vast separate chapter, of great consequence (see de Mello[30]). One must minimize statistical dispersions around the homogeneous average need. The question arises of what tolerance to accept. It is often equivalent to questioning the possible consequences of some consistent variation, and reflects on the basic inquiry of Design Principle 5. Many such inquiries can and should be tackled by judicious design computations. *Figure 44.41* is merely one such example, investigating the likely detrimental effect of some layer, poorly compacted, occurring at some level within the dam. Intuitions generally suggest that the greatest worry occurs with softer material at the bottom. In as far as construction period settlements are concerned, however, the most unfavourable position is found to be at about mid-height.

44.7.5 Instrumentation and its appropriate use

Instrumentation has been indispensable for monitoring incipiently unsatisfactory behaviour, in sufficiently repetitive situations to establish statistical laws. Once again, the topic involves a vast chapter of modern engineering, in which embankments have been more and more involved.

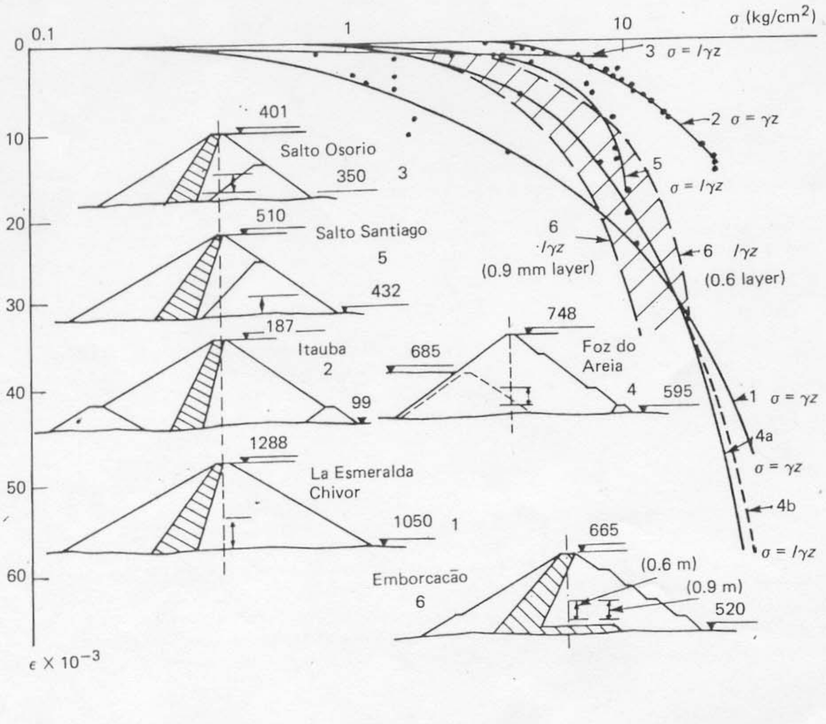


Figure 44.39 Rockfill compressibilities. Need to include influence I factors for stresses. Convenience of plotting s *vs.* $\log\sigma$ (de Mello[6])

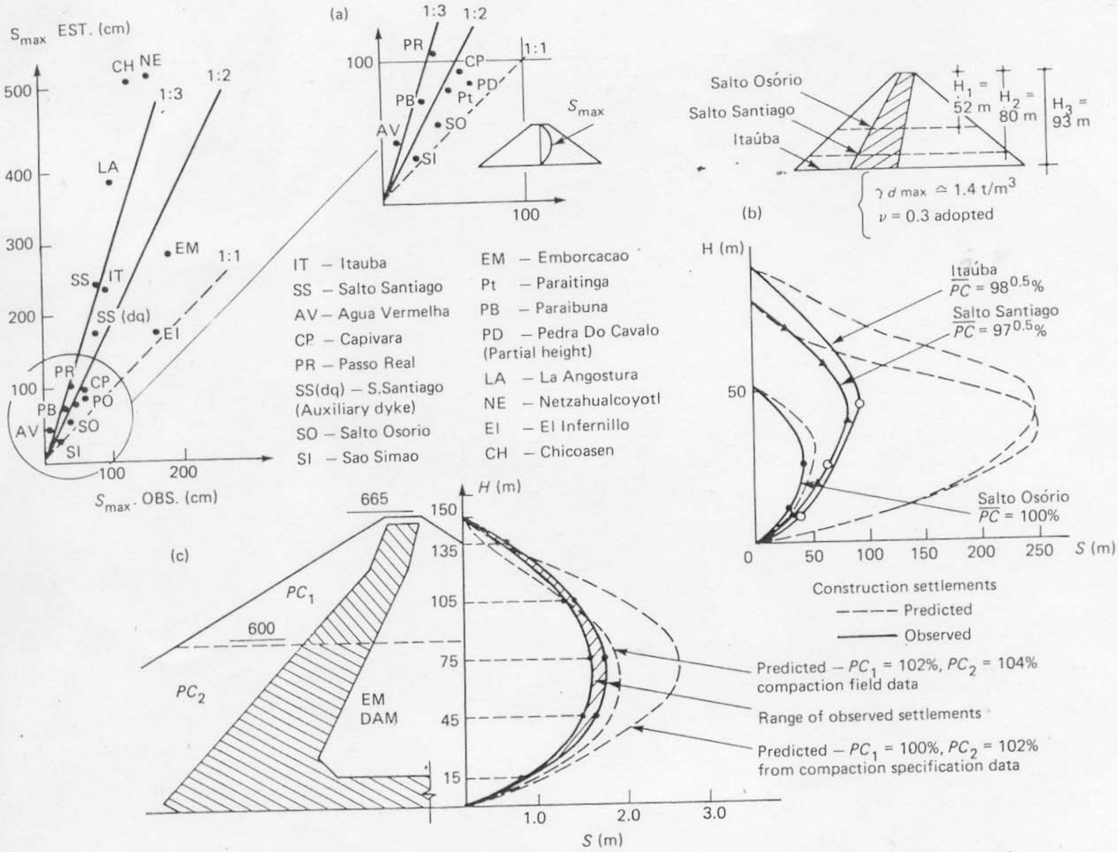


Figure 44.40 Sample cases: predictable *vs*. observed max settlements; predictions based on present suggestions (de Mello[6])

Many are the criticisms of most present practices of instrumentation planning, observation, and routine interpretation (see de Mello[1,31]). One momentous warning is against relying on instrumentation for purposes of safety. Failures as accidental singular events cannot be averted by instrumentation. In fact, the psychology of confidence in instrumentation has unwittingly contributed to shifted emphasis of attention, and increased incidence of unsuspected failure modes.

Instrumentation is tied to some implicit theory, and therefore must be consciously oriented on the basis of such a theory, or advances and revisions thereof. The natural tendency is to persist in instrumenting and monitoring for the already established theories. Thus, maximum benefit/cost ratios of investment result from reappraisals of current practices. *Figure 44.42* merely serves as one such example. Self-weight settlements of rockfill dams have been repeatedly monitored, and practically no attention paid collaterally to transverse displacements, associated with Poisson's ratio, ν. Recently emphasized evidence that upstream decks have suffered much smaller displacements under reservoir loading, than was predicted, served to raise the question. One sees from elastic solutions that values of Poisson's ratio should have much influence, even on self-weight settlements. Prototype monitoring along this hypothesis is presently a priority need.

44.8 Miscellaneous

The topic of embankments (see Casagrande[32]) was extended into a discussion of embankment dams, with the intent of discussing principles of design decision and computation relative to engineering structures of extreme importance and responsibility. It would be irresponsible, however, to close the chapter without emphasizing that one could not possibly cover some of the points of greatest importance and risk in designing embankment dams as parts of a hydraulic project. Impounded water makes its way along discontinuities, and the greatest risks are associated with the design and construction of carefully detailed connections between the embankments and the appurtenant hydraulic structures. The principles discussed persist, and must be applied much more judiciously, because of the significant difference in behaviour of contiguous materials, earth in contact with concrete hydraulic structures.

The fact that by far the greatest number of failures is associated with small dams should serve as an emphatic reminder that the main factor is not connected with greater problems and risks with the smaller dams. It is clearly connected with the tendency to attribute such dams to engineers insufficiently prepared to question and revise, as the case might require, the presumed conventional design, construction, and monitoring routines.

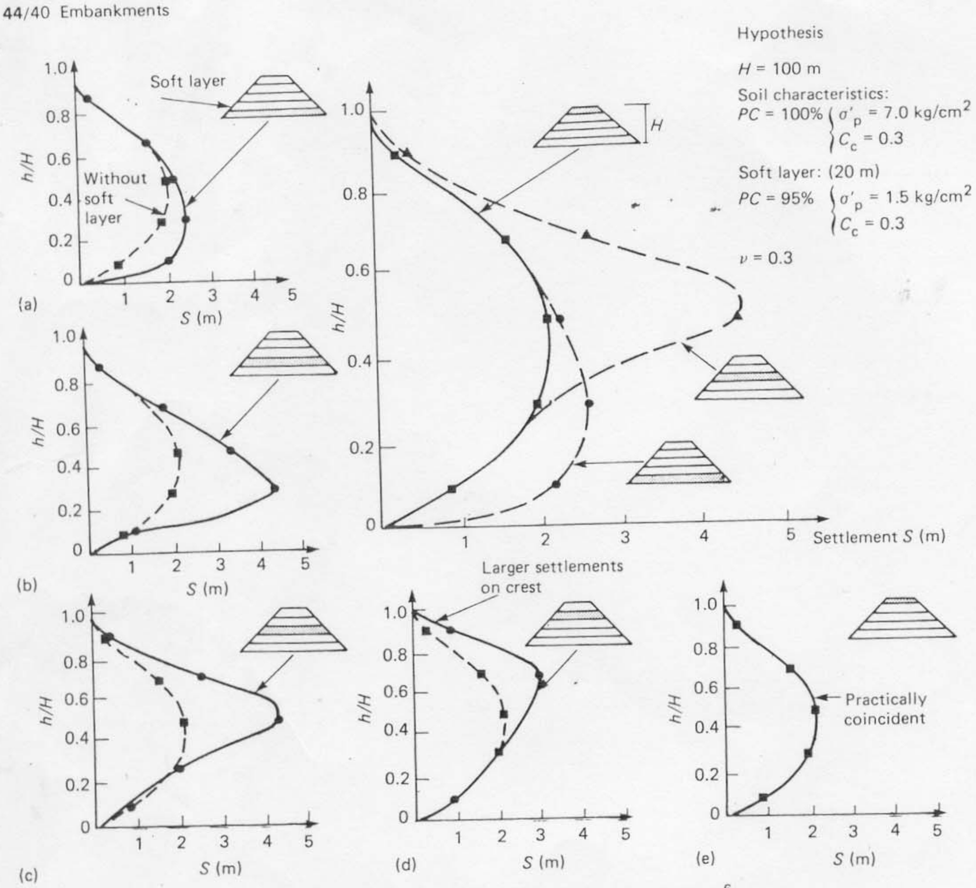


Figure 44.41 Analysis of position of greatest consequence of a 'softer-layer' (de Mello[6])

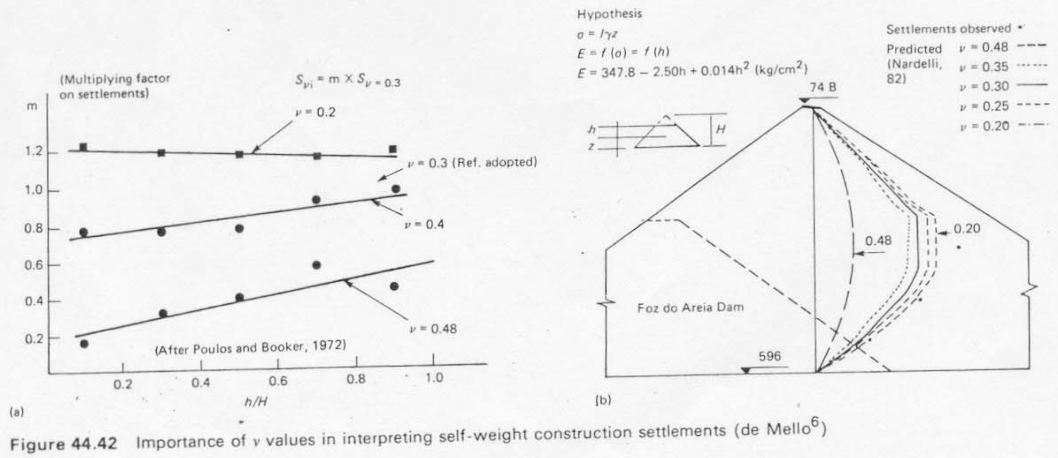


Figure 44.42 Importance of ν values in interpreting self-weight construction settlements (de Mello[6])

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