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TWENTY SECOND YEAR OF PUBLICATION

ENGINEERING NEWS

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In this Issue

Double Your Money?

Get rich quick. Turn your copper into gold. Double your money ! These are standard baits with which the clever trickster lures the credulous simpleton into his age old trap—and invariably deprives the poor fellow of his meagre belongings. The story is repeated with monotonous regularity all the world over. Such wicked sorcery is practiced as shamelessly amongst nations as amongst individuals because there are foolish nations as surely as there are foolish men ; and the extraordinarily bizarre world of international relations, trade and commerce, is without mercy. It is a world dominated by tricksters who thrive on relentless exploitation of the naivette and gullibility of their poor worldly neighbours. It is a world where poverty and aspiration make an odd couple whose unbridled, sinful cohabitation generally leads to frustration, chagrin and dis-enchantment.

The poor, under-developed countries of the world are being taken for a ride. Their genuine desire for self improvemant is an easy, tempting target for the rich, advanced countries. The routine goes something like this : The poor brother is dazzled by the wealth around him. The rich brother offers a patronizing helping hand ; prepares a special magic formula ; provides the technical know-how (complete with the expert manpower) ; even throws an attractive loan to remove any doubts ; and, of course, promises to double the poor man's money. Later, the rich brother generously offers to investigate why the special magic formula did not work and quickly comes up with yet another plan with the usual paraphernalia. And so, gradually but surely, the noose tightens around the poor brothers' neck.

Pakistan is a classic example of such a poor brother, patronised by more than one rich brothers who are only too eager to help her acquire instant wealth.

To meet the requirements of her ambitious development plans (which are generally prepared financed and implemented by outsiders), Pakistan has resorted to deficit financing for the last several years, thus perpetually increasing the country's debts and liabilities. No satisfactory means have yet been found to comfortably repay the debts while a major chunk of Pakistan's foreign exchange earnings has been going into payment of interest on these debts. Inspite of having tried every magic formula, the country's economy has remained virtually stagnant. The overall production in the country, and hence the export capability, has remained limited, both in the industrial and agricultural sectors. It is now being hoped that a substantial oil find will comfortably tilt the balance in Pakistan's favour but, in this field, we are still very much at the exploratory stage,

Luckily for Pakistan, new factor has recently permeated into the country's economy which has provided the proverbial silver lining to an otherwise black cloud. The invisible hand of the toiling Pakistani labourer and worker abroad has performed a surprise rescue act, giving a strong support to the country's shaky economy.

Last year, more that eight hundred crores of rupees were remitted home by Pakistani workers employed abroad. Most of this money came from the Middle East. This amount represents a major share of Pakistan's foreign exchange earnings. In fact, the amount is so large that one easily tends to overlook a real possibility of increasing it many fold. The trick, of course, is to organise manpower export and eliminate the possibility of exploitation and under-payment by the foreign employer. This can best be done if Pakistani labour is employed by Pakistani contractors to execute the various development and construction projects In the Middle Eastern countries, thus getting the maximum value for manpower on the international market.

The construction market in the Middle East is a unique phenomenon of the present decade. Out of an estimated oil revenue of over 1000 billion dollars per annum, the Middle Eastern countries are expected to spend more than 66 billion dollars on their development budgets. If Pakistan can capture just 1% of this construction market, the country will be well on the way to an economic recovery.

A start in this direction has already been made by some state owned companies. Working in isolation, and using individual contacts, these companies have managed to get some contracts-large by Pakistani standards—but they have just managed to touch the tip of a huge iceberg. There are genuine difficulties in the way, quite apart from the usual starting troubles. The potential of the market—and Pakistanis ability to exploit it—is not fully grasped at the government level. There is insufficient support from Pakistani embassies and trade offices abroad and little help from agencies like the State Bank and the Emigration department. The banks are reluctant, making it difficult at times for the Pakistani companies to submit the necessary bid bonds or, in case of successful bidding, to furnish performance guarantees.

If speedy, timely steps are taken at the highest level to co-ordinate the isolated efforts, pool the available resources, streamline the local procedures and eliminate the bottle-necks, Pakistan can enter the Middle East construction market in a big way. Here is a completely self reliant magic formula that can certainly transform the shape of our economy by not just doubling our money but increasing it many fold !

Irrigation
and
Power Section

Earthern Embankments For Flood Control & Irrigation Channels

By

Shafaat Ahmed Qureshi[1]

A large number of flood protection bunds have been constructed in Pakistan along rivers, streams and nullahs to save towns, agricultural lands and village abadis from their fury when in spate. All these earthen embankments, however, according to current practice, are being designed and constructed on the basis of empirical rough guides given in the departmental books of references written more than 30 to 40 years ago. These rough guides for design sometime do not hold and failures have occured.

It is also ironical that even as an irrigation engineer who spends major parts of ones life dealing with "Water" and "Earth" in the construction and maintenance of earthen embankments, flood bunds and other earthen dyks etc. adequate attention has not been paid to the knowledge of soil science and accordingly its application has not received the attention it deserved. Without much improvements, we seem to cling to the old practice of using empirical formulae although we have seldom tested if these formulae really hold in field. Soil classification on the basis of which the design depends is also vaguely done ; and vague terms, from which correct inference about the properties of soil cannot be drawn, are being used. Like wise in the field of construction no testing to check for the standards assumed in design is practised. Costly massive embankments are thus being constructed without proper knowledge and practice of the principles of soil mechanics or even datailed classification of soils. When failures occurs invariably the poor maintenance or construction staff is blamed though in some cases faults may not be due to them.

The art of flood protection by construction of embankments, low

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dykes or bunds as we call them is infact very very old. From the earliest periods when people began to live together usually close to rivers or streams the method of raising these bunds has been found to be very useful. Earth being the cheapest construction material has been extensively used. Soils available in the vicinity are made use to cut construction costs.

In Pakistan from its very inception in 1947 floods have been a constant source of anxiety and concern. Often they result in enormous damage to our economy, besides irrepairable loss of human lives. At times even national security appeared to have been jeopardised. In Lahore the Provincial Capital of Punjab, the experience of 1950 & 1955 floods, in the River Ravi, would be still vivid in the minds of elders and senior people who had to wade through the spill water to reach their offices in the P.W.D. Secretariat.

During the past many years therefore a large number of flood protection embankment have been raised to save Towns, Agricultural land and other important installations. A bund nearly 14 miles long on the left bank of river Ravi exists for protection of Lahore. Likewise on its right side there are other bunds for the protection of famous Jehangir Tomb, Shahdara Town and industrial area there. Several other such bunds have been constructed along other rivers, besides, marginal bunds and spurs which have been constructed on all Barrages, Headworks, Rail Road and Highway bridges etc: to provide river training and control during floods and to afford protection to the adjacent lands against river spill. It is estimated that in Punjab alone nearly 1300 miles long flood protection embankments exist, while 600 miles long are being contemplated. Because of various constraints to the development of other resources, heavy reliance is placed on these earthen structures and we all cherish and pray for their safety during heavy floods.

Probes into adequacy and improved standards of Flood Protection measures.

After the record floods of 1955 which was experienced in Ravi and Satluj Rivers in Punjab, Government appointed special committees and commissions to probe into the causes of the floods and suggest remedial measures. West Pakistan Engineering Congress also organised symyosium on floods in 1958 and published number of papers concerning the causes of floods, forecasting needs, and flood control requirements and practices. In view of all these deliberations various new bunds in Punjab were raised and old bunds strengthened. However, unfortunately little or no attention was paid towards the real and physical factors affecting their design and construction; and stability of these bunds from the point of view of soil characteristics. The new design standards laid down only increased flood heights (HFL's) for raising of the existing embankments or construction of new ones; and more width at top of the bunds to provide for easy transport of material and inspection of officers (after accounting for the rain cuts and gharas which generally occur during rains etc.). Therefore in 1973 when very high floods again hit Punjab and Sind many of these embankments failed. Though failures were generally attributed to poor maintenance levels resulting from the long period of dry spell after 1958 (which had also caused gradual cuts in the financial allocations and therefore postponement of works). However, in a number of cases it was also observed that bunds had collapsed due to soil characeteristics. For example in case of Shahdara Flood Protection Bund, which had been built some time after the 1955 floods, with high clay content it was observed that initially many leaks had appeared as the flood water started rising against the bund. (The rate of rise was approximately 0.5 feet per hour) Puddling and staunching of its upstream face had therefore to be resorted to stop these leaks. When water levels rose higher than previously observed maximum levels no more leaks occured. However in its reach RD 7200—15000 the bund suddenly collapsed at seven places when the H F.L rose nearly 4.5 higher than the designed H.F.L. In the portion which survived it was noticed that bund had a long and almost continuous crack which appeared to be running through the middle of the crest, besides other cracks in transverse direction. The long continuous crack was also observed in case of the Spur which was located, in this reach of the bund, for river training, but which did not have to face so much difference in water level on its two sides. The manner of cracking and failure indicated horizontal bedding planes, structural weaknesses, shrinkage in the body of the bund, excessive settlement of the foundation soil and thus the need for proper placement of soil, blending, and compaction for bund reconstruction. Likewise the Madhodas bund near Balloki Headworks at some places of its run has very tricky soil. This soil when dry is very hard and difficult to be even dug but when in contact with water it easily gives way. Here again the soil blending was found necessary.

Similar failure due to poor soils is reported to be responsible for a breach in Thal Canal Main Line Upper during 1976. In this case massive earthen banks sufficient to cover even 1 : 6 'hydraulic gradient' existed behind the lined concrete section of the canal. These earthen banks were formed of the local soil which is silty and clayey in character. In normal cases such slope of H.G.L. would be provided for unlined section. However, the bank on the non patrol side is reported to have suddenly collapsed before the very eyes of the inspecting staff. In this reach, after the repairs, the average slope of water surface observed through installation of piezometers in the body of the earthen embankment was found to be even more flat than 1 : 10. The D/S of the bank in almost entire reach was always wet with a small quantity of seepage water oozing out from the same. Evidently the soil was poor and did not meet the standards assumed in the design of the bank.

Like the above cases there may be more examples of failures, (of embankment) due to soil characteristics, which went undetected or where blame for breaches etc. might have been attributed to other causes. Nevertheless after the floods of 1973 and even after 1975 & 1976 floods no attention was paid to the characteristics of the soils involved; and repair to breaches and raising/strengthening of banks have invariably been made without going into the details of foundation soil or characteristics of embankment material viz-a-viz its grading and mix required to form more stable embankments. Emphasis was laid on addition of pushtas to stop the seepage, and preference to machine work was given to obtain resultant compaction, which tend to better the appearence and also preformance of the embankments formed from even the poorest of the soils otherwise unfit for bund construction.

After 1973 floods wetting channels have been introduced along many marginal bunds and other places where breaches would cause severe damage. These wetting channels, as a matter of soil characteristics (of the embankment section) are required to eliminate risks due to shrinkage cracks in clayey soils by presoaking the bunds before floods. Here again it has been observed that these are being constructed indiscriminately and without any regard to the need and type of soil e.g; at Balloki Headworks along its L.M.B. and R.M.B. wetting channels have been constructed, although previously to justify their strengthening it was explained in the reports of the estimates that the existing bunds, were found to be sloughing at the outer toe; and thus the bunds needed to be remodelled with 1 : 6 hydraulic gradient line (criteria usually adopted on sandy soil which do not develop shrinkage cracks and need of the indulgence in wetting channels).

In the succeeding sections therefore an attempt is being made to discuss the influence of soil type (in the foundation and embankment) on the behaviours of these embankments and the various factors which affect the design and construction of these earthen embankments. It is hoped that with ever increasing need for higher and more stable embankments the role of soil mechanics in the design and construction of these earthen structures will be properly appreciated.

Influence of foundation soil on the design and performance of earthen embankment.

On the foundation soil depend thə rate of under seepage and formation of boils on the downstream natural surface of bunds, the base width required to support the weight of the embankment super structure, differential settlement (including total settlement and reduction in free board) ; and the stability of the slopes. Reference books of the department which lay down specifications for design of bunds clearly state that "width at base should be sufficient to support the super structure and to prevent creep". This however does not prescribe for any systematic or detailed foundation exploration, nor it is practised in the field.

In fact, the natural soils are generally eratic ; and often less dense than embankment material. The excessive seepage and piping through the embankment section having been taken care of by selecting suitable section and compaction, formation of sand boils on the downstream natural surface can still be expected if the exit gradient or residual pore pressure of the under seepage flow exceeds certain limits and if the natural surface layer of foundation is sandy. Studies on measurement of foundation pore water pressure along Missisipi river dikes where large under seepage and boils developed indicate that while conditions required to form boils differ considerably from site to site, boils should be expected in fine, cohensionless soils whenever the upward seepage gradient exceeds 0.5 to 0.8.

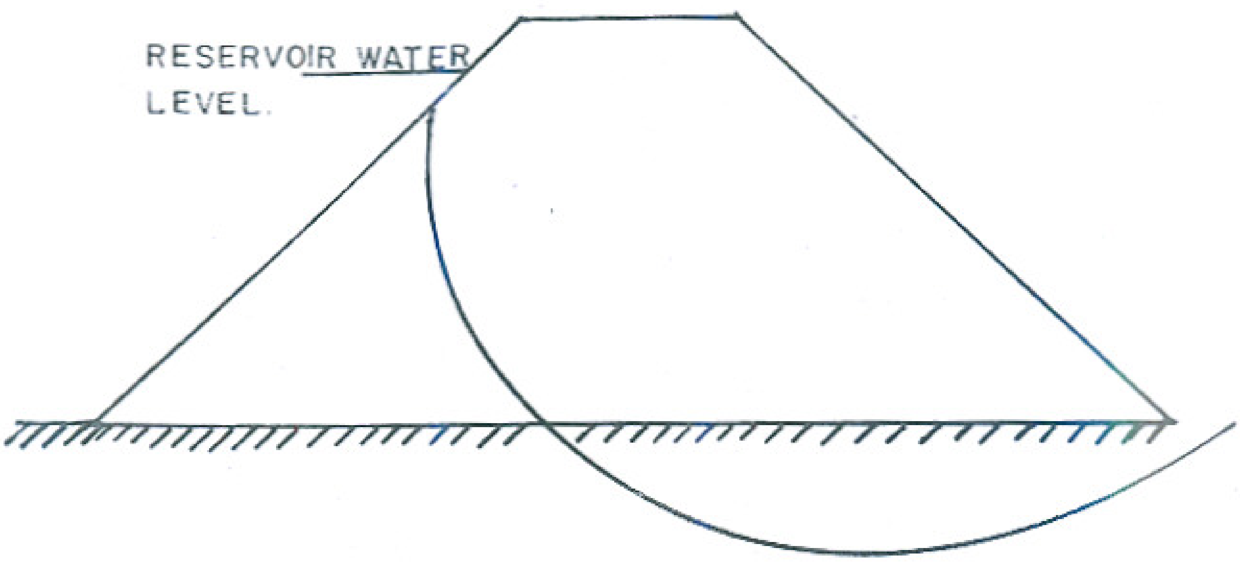
Progressive sloughing (revelling) of d/s slope starting from toe is also associated with high exit gradients due to reduction in margin of safety against shear failure.

The ability of the foundation soil, without excessive settlement/differential settlement, to support the weight of the over-burden determines the size of the embankment (base width). If the soil has low shear strength it will require broader embankment and flatter slopes. Similarly, soft soil will receive consideration from the point of view of embankment settlements and possible reduction in free board.

Again depending upon the geometery and relative compressibility of the foundation, the embankments built thereon, can be twisted in different ways (under varying type of pressures) resulting in different cracking patterns. Some of the cracks may be very dangerous—particularly when they occur only internally and are not visible on the surface. These transverse cracks (which can create paths for concentrated seepage) are usually caused by differential settlement taking place between adjacent sections of embankment. This happens when higher parts of the embankment are under-lain by relatively more compressible soils than beneath its parts where the height is low. This type of cracking may cause vertical or inclined cracks. The amount of cracking however, depends on the magnitude of the strain imposed and on the deformability of the embankment itself.

In case of high embankment section slides of slopes involving foundation can also occur during construction (by mechanical means) or during operation. These slides occur in the same way that land slides develop in natural earth slopes when average stress along any poetential surface becomes greater than average strength. In every case of construction slides (based on observation for some dams) the foundation was found to be of either soft, brittle or sensitive clay, usually of high plasticity. Slow slides occured when the foundation was more or less homogenious deposit of clay which was not sensitive (which did not loose its strength appreciably under the shearing movement). Rapid slides, however occured when foundations were of clay and contained horizonal bedding plane, layer or lenses of coarse silt or fine sand through which the high pore water pressure developing under the centre of water retaining embankment can be transmitted towards more highly loaded area under the toe.

During operation of embankments, which retain water, deep slides through the d/s slope and foundation usually take place in clay foundation. They frequently reduce the free board by extending further than the u/s edge of the embankment crest. The internal pore pressures which cause deep slides are the result of seepage through or under the structure. Recorded evidence of such slides come from low dams such as Fruit grower's dam in western Colorado (32 ft. high dam) and Great Western dam near Danvor Colorado (height 50 ft.). In the later case the slides began after the new reservoir elevation was at its max. for two weeks. The sliding surface passed into the natural clay foundation and extended well u/s of the up stream edge of the crest, actually cutting below the water level in the reservoir. Free board by sand bagging could not be created because any added weight near the top of the



CASE OF SLIDING SURFACE INTO NATURAL CLAY FOUNDATION AND EXTENDING U/S OF THE UPSTREAM EDGE OF THE CREST CUTTING BELOW WATER LEVEL IN THE RESERVOIR sliding mass would cause the movement to accelerate. The dam was saved from bursting by gocd fortune and hard work when water level was pulled down at more rapid rate than at which the slide moved. However, subsequently the dam had to be rebuilt.

In view of the above, therefore it is necessary to explore the foundation charcteristics. The Purpose of foundation exploration should be to determine the dimension, location and physical charactistics (strength, permeability and compressibility) of the various soil layers which comprise the embankment foundation.

Determination of the strength of the foundation soil.

If the soil consists of natural strata of clean pervious sand and gravel, the sheer strength can be approximated by means of rough statistical relations between the strengths and the relative-densities and grain size characteristics. Strength in fine grained impervious soil is estimated from laboratory tests on undisturbed samples or from vane shear test. If the soil is clay with slicken sided fracture plan or has a jointed structure the laboratory analysis should be supplemented by careful field examination in test pits or trenches to investigate the nature of cracks.

Likweise for permeability where sub-soil generally consists of relatively uniform layers of sand and/or gravel it is possible to obtain a rough indication of the foundation permeability from laboratory tests on undisturbed samples. However, reliable estimates of the permeability of sand and gravel deposits as well as of silt clay deposits with coarse soil can be made from field permeabilty tests. Above the ground water table these tests are performed by pumping water into bore holes or by measuring downwards seapage under the bottom of tests pits. Usual well pumping tests below the ground water are more reliable.

Extent and depth of exploration.

As a general rule the extent and depth of exploration depend upon importance and height of embankment above N. S. L. and subsurface conditions which vary from site to site. However, it is desireable that a depth equal to about the proposed height of embankment should be explored through tests pits and bore holes etc. The information available from nearby tubewells and local wells can be used to supplement the knowledge. The spacing of bore or test pits depend upon the soil stratification but it is suggested that initially such exproralary shafts should be 1000 feet apart.

Influence of soil properties on the embankment peformance and section of soils.

On the properties of the soils used

in the construction of embankment depend the amount of internal cracking, settlement and bank slopes. Rate of seepage and piping resistance are also functions of soil characteristics. Besides the above, compaction techinques and the control of water content during construction also depend upon the soils selected for embankment section. Likewise soil properties play a very distinct role in the design of embankment viz a viz whether it should be a homogeneous section of one soil or the performance could be improved through zoning/introduction of cores (of sand etc.) or use of admixtures of two or more available soils or by wetting channels etc. Some of the soils specially containing Kallar are treacherous and unfit for bund construction. Likewise high organic fiberous soils (peat) are not chosen because of low shear strength and high compressibility. In the same way organic clays of high plasticity are not selected because of construction difficulties.

Inspite of above, however, generally no Investigation of soil properities on scientific lines are being made nor selective use of soils through grading or classification is practicised. Field supervisors and other untrained or semiskilled persons just report the type of soil available from visual inspection. The same criteria is then carried in design to select the "slope of Hydraulic grade line" as per departmental books.

The reference books lay down that embankments be constructed generally from "inferior soils" which exist close to the site—without any reference to study of its properties or method of improvements in performance of structure. Likewise regardless of the low piping resistance (and without proper densification of cohensionless materials) these also appear to favour embankments made of sand. For embankments which may be constructed of clayey soil though compaction is desired for them, but there is nothing to ensure uniform results. Similarly the types of clays which are more susceptible to cracking, piping and slides have not been identified. Thus besides the need to know the influence of soil properties on the design, construction and performance of the earthen embankment, it is also necessary that proper investigation of the soil is made to predetermine as to how we can improve upon our designs, method of construction (according to available resources) and future performance of these bunds. The need for the above becomes greater for important and higher bunds. The following are some of the ways in which available soils can influence the selection of embankment section, method of placing and construction and also the future performance of the earthen structures.

(i) Homogeneous embankments

Homogeneous embankments, as the term implies are constructed entirely or almost entirely of a single embankment material. They are most often used in embankment usually encountered for flood control and irrigation purposes (including small dams) because their construction tends to become unduly complicated if they are zoned. Such type of embankments have been built since earliest times and are most economical even today whenever only one type of material is available locally. Some of the advantages of a zoned embankment can, however, be obtained in homogeneous structures when soils are placed "selectively" or when different construction methods are employed in diffetent portions of the embankments. Zones of lower permeabilities, higher resistance to erosion and settlement can be achieved by using more compaction or slightly higher construction water content than optimum. Using more compaction will make a considerable difference in the permeability of residual soils and other materials which break down as they are being rolled. Additional water content (1% to 3% above the optimum) during compaction makes the clayey soils, considerably more inpervious and capable of appreciable deformation without cracking than they are when compacted on the dry side of optimum water content (at the same dry density) and the difference persists with time.

Homogeneous embankments with a height of more than 20-25 ft. (as for example as the case of banks of Thal Canal waich breached in reach RD 25-60) will need some type of D/S drain so as to (i) reduce pore water pressure in the D/S part and hence to increase the stability of D/S slope against sliding; and (ii) to control flow of embankment soil with the seepage, i.e; piping does not develop. In an effort to intercept seepage water before it reaches the D/S slopes chimney drain or sand core can be used. Dr. Nazir Ahmad and S. L. Shah in their paper "improving stability of flood embankments with sand core wall" presented in the fourth congress of international commission on irrigation and drainage have suggested a core wall of 1/5 to 1/6th the width of bank at top and of greater permeability coefficient than the rest of the bank, to be kept close to beginning of the bank on u/s side. In addition installation of short vertical baked clay strainers joined together with a baked clay pipe for discharging the effluent to a safe place are also favoured.

(ii) Control of construction water contents—clayey soils.

During the placing of an embankment at a high water content it has to be ensured that the construction surface does not undulate directly in front of the equipment. Under certain conditions the shear stresses associated with this movement have caused shear plans in horizental surface to develop. Research by U.S.B.R. indicates that the shear plans can be produced in clayey soils in laboratory when the material is compacted with a tamper at a water content above standard proctor optimum.

(iii) **Resistance to piping—poor soils.**

From a study by USBR of 31 dams which developed concentrated leaks it was concluded that embankment soil properties and particularly the plasticity of fines have a large influence on the piping resistance than the method on which the embankment are compacted. It was also observed that embankments constructed of clay with plasticity index greater than 15 demonstrated highest resistance to piping; while the embankments constructed of fine uniform cohesionless sands had the least resistance.

(iv) **Cracking in embankments**

A large number of failures which have occured when flood water first touched the earthen embankment have been attributed to piping through leaks formed along tree roots or holes drilled by burrowing animals, but actually piping in many of these cases undoubtedly started in embankment cracks. There are many causes and tynes of cracks.

(a) The amount of cracking due to differential settlement which may develop in an embankment depends on the magnitude of the strain imposed and on the deformability of the embankment. There exists no reliable guides either from field observation or laboratory tests for estimating the max. amount of embankment settlement which can take place at a given site without development of cracks. However some evidence from the study of the performance of 17 dams some of which cracked and some of which were subjected to large strain without cracking indicates that, embankments of inorganic clays of low to medium plasticity (P. I. less than 15) with gradation curve falling within a specific range are probably more susceptible to cracking when compacted dry, than either fine or coarse material. Clays of higher P. I (more than 20) which are finer than the gradation range will withstand much large deformation without cracking. The study also indicated that susceptibility to cracking is high in embankments of residual soils containing coarse particles of soft materials which break down and become appreciably finer when they are being compacted.

(b) Excessive drying cracks in embankments constructed for flood control may appear if the material forming the bank is clay. Such cracks are sometimes serious in regions like Pakistan where floods come for only one to two months and that also for some days ; and for the rest of the year the embankment remain dry, thus all defects of occasional wetting and drying appear. Surface erosion concentrates into the embanment cracks which develop. In many cases (of dams where this trouble was noticed) it has been necessary to refinish the slopes and crests completely ; and to add a layer of cohesionless material to minimize the surface drying and erosion.
This could perhaps be one cheap alternative for the costly wetting channels in some of the cases.

(c) If the construction surface of an embankment of fine grained soils is allowed to dry in the sun (before next layer is laid) drying cracks can greatly increase the overall permeability of the material. The cracks may not be ordinarily visible on the surface (as in case of Bureau of Reclamation Lovewell Dam in Nebraska) but a system of drying craks may result in increased piping potential. This emphasizes the need for undertaking the work in mild climate and speedy completion of the embankment section in the reach selected.

Slides of U/S and D/S slopes of Embankment.

Strong corelation exisits between the incidence of slides and the use of fine grained and highly plastic soil in the embankment.

From study of 56 old homogeneous dams in the western United States it was concluded that all those constructed of clay having an average grain size (D 50) finer than 0.0006 m.m. failed by sliding and from those made from clays of medium grain size, ranging between 0.02 to 0.0006 m.m. approximately half failed by sliding. None of the dams constructed with grain size coarser than 0.06 m.m. failed by sliding.

Danger due to Burrowing Animals.

This is another type of danger associated with soil type. This is due to burrowing animals which operate in dry banks of cohesive material to make homes or to dig passage to reach upstream face of the embankment. Along these holes, in clayey soils, serious leaks can generate. Sand banks, sand cores or other type of diaphragms (impregnable) can prevent danger due to burrowing action.

It is also believed that noise of

traffic on the top of bund specially trucks, prevent the rats, and such other animals from operation in the bank. However in some parts of Saskatchewan highway embankments with swamps and sloughs on both sides of the road the embankment get so much riddled with muskrat holes that they must be rebuilt every third or fourth year.

Need for Exploration.

The exploration for prospective embankment materials should thus begin at the same time as the foundation investigation. The possible borrow areas, shall be selected first through field reconnaissance excluding those having excessive kallar, and high organic fiberous content.

The following factors must be studied, beside usual laboratory tests to determine grain size, moisture content, in-place denisty and Atterberg limits :

(a) The lead in-volved i.e., the relative distance and elevation of borrow area with respect to embankment.

(b) The nature and thickness of organic top soil which must be removed.

(c) The natural water content (in case of excessively wet materials if it can be lowered practically during construction).

(d) The denisty of the natural soils in the borrow pit and an estimate of shrinhage i.e., difference between the volume of a given weight of soil in the borrow pit and its volume in the embankment after compaction.

(e) The best way to proportion or mix : from different soils or if they be placed in seperate Zones (as against uniform blend or mix :).

(f) Compaction required to make the embankment.

Other Factors Affecting The Design of Earthen Embankment.

(IV) Hydraulic Garde Line.

In Punjab, after the size of the embankment has been determined with a given top width and fixed side slopes, the section is tested and if necessary modified to contain with in the bank an imaginary line starting from the flood (or full supply) level on the u/s side of the bund. The slope of the "Hydraulic grade line" (as it is called) is taken as 1 : 3 for clay ; and varying between 1 : 4 to 1 : 5 for sandy and sandy loam soils. A low berm is added on the land side toe if the line there is not covered by a minimum of 2 ft, of earth (ground). In Sind, the slope of the line is assumed as 1 : 6 to 1 : 8 regardless of the soil type.

The above criteria, it may be seen is only emperical and does not hold always-as for example in case of banks of Thal Canal where recently a breach occurred (RD 30-31). In this case the record of observation of free water surface through the banks formed of the silty sand indicates an average slope of 1 : 10. Thus in the absence of more knowledge the empirical guide could be used conveniently (though not very economically) only for low embankment where any refinements would make little difference in embankment volumes or cost. For higher embankments, however, it is evident that we must deviate from the empirical Omni Bus rules, evolved when knowledge of soil mechanics and soil testing techinques had not spread so much ; and perhaps too high embankment were also not so frequent as we are building today ; and must go into more details to evolve better (and economical designs) The present method is based on assumed properties of soils—even assuming a distinct phreatic surface in a bank constructed of clay. The criteria also does not offer hel-p in evaluating the amount of seepage loss which has to be controlled through the embankments constructed for irrigation channels ; or for calculation of pore pressures for stability analysis of embankment slopes. Some discussions therefore appear necessary into various aspects of seepage problems, calculation of pressures and stability analysis.

Whenever an embankment, is subjected to a difference in water level on its u/s and d/S side, a seepage flow starts through the "pervious bank". After some times when flow is stabilized, the top most flow lines becomes the free surface along which the flow actually takes place. On first entry into the embankment a sudden drop in elevation of this line takes place. Another sudden drop take place near the exit end. However, it has regular grade in the body of embankmant along which it flows. This slope of the phreatic surface is sometimes called the slope of H. G. L.

Based on observations made through test pits in cores of dams. it has now been established that it is only in case of embankment of coarse soils that a well defined upper surfce of seepage will develope if reservior is kept full for . . . a sufficient length of time. For rolled embankment of fine grained clay or silt the embankment materials have the same appearance from crest to foundation even when when the reservoir has been filled for long periods. In such cases the capillary forces commonly have more influence on pore pressure distribution than the gravity heads and the seepage pattern is accordingly vastly different from that which would result in a similar embankment constructed of pervious sand etc. Therefore, the very concept of free surface (and attempts to contain the same within the bank) is meaningless in case of embankment constructed of clayey or silty soils having very low permeability coefficient. Similar argument can be advanced for a regular grade of the phreatic surface through the flood embankments constructed from even coarse grained soils where water is not retained for considerable length of time. For homogenious banks constructed of previous soils and even subjected to constant water pressures from the U/S side (as for banks of irrigation channels) through which seepage flow have stabilized, it is well known that flow characteristies depend upon the geometry of thebank and difference in water levels. The point of emergance of the free surface on the downstream end is almost constant at some dis tance above the water level on the D/S side, irrespective of whether it be througha narrow or wide bank. Case-grande has derived certain rules and relationship to determine the point of emergance etc.) and G. Gilboyhas given a chart for solution of the same. Therefore, attempts to shift and/or burry the point of emergence of this free surface by addition of pushta on the land side the (as per prevalent design standards) does not materially change its position. It is however true that increase in the path of flow will mean more resistance to seepage and consequently less discharge through the bank which adds to its stability.

The fixed slopes for H. G. L. (free surface) as applied in design of embankments presently do not help in determination of the amount of seepage (for control of seepage loss through embankments of Irrigation channels or other water retaining structures) and to calculate pressures (as would appear from the name-hydraulic gradient line) necessary to analyse the stability of section against shear and to evaluate the piping potential.

The amount of water seeping through and under an earthen embankment together with the distribution of pore pressures can be estimated by using the theory of flow through porous media.

The general hydrodynamic equation for steady state seepage (Laplace equation) can be written :

$$\frac{d^2h}{dx^2} + \frac{d^2h}{d^2y} + \frac{d\ h}{dz^2} = 0$$

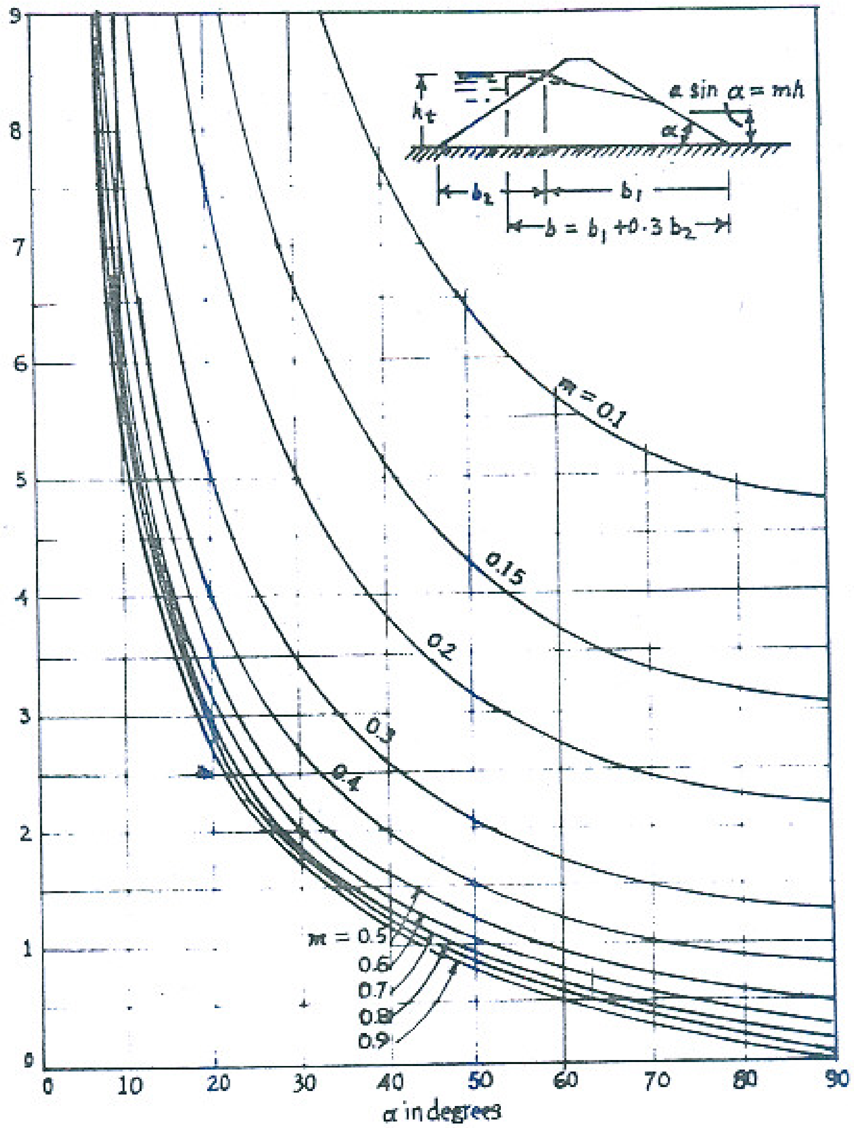
The solution of the above equation gives the variation of pressure head and direction of flow at all points-within the area of soil through which the water is flowing. Based on Forchheimen and Richard Sons discoveries special solution can be produced by graphical method-called "flow net", which is the easiest and most widely used technique.

The total qnantity of seepage through an homogenious bank which has the same coefft, of permeability as its foundation can be calculated as follows :

$$Q = Kh \times \frac{Nf}{Nd}$$

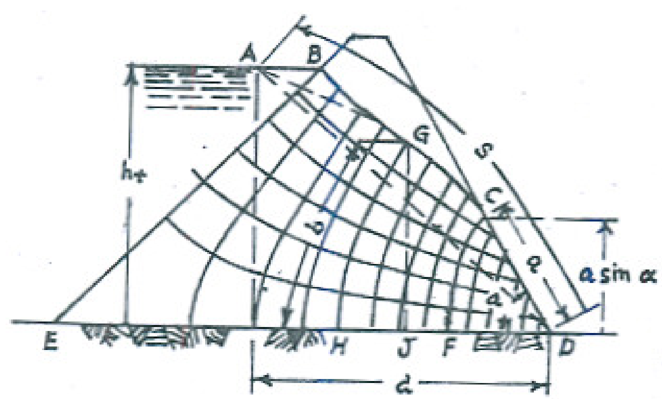
Where K —Coefft of permeability (ft/hour).

h —Total pressure head loss (ft·)



GRAPH FOR CALCULATING THE POINT OF EMERGENCE OF FREE WATER SURFACE THROUGH AN EARTHEN (POROUS) EMBANKMENT SOLUTIONS BASED ON L. CASAGRANDA METHOD

(G. GILBOY)



CASAGRANDE METHOD

FOR THE DETERMINATION OF THE TOP FLOW LINE

IN TRIANGULAR DAM. ON IMPERVIOUS FOUNDATION

Nf—No of flow channels.

Nd—No of equipotential drops.

Q—Rate of seepage ($ft.^3$/Hour).

If the flow takes place through stratified media or through a zoned embankment (using different soils in the cores) flow pattern does not remain similar to one through a homogeneous bank but it changes. More area is required for the same quantity of flow when seepage moves from a zone of higher permeability to one of lower so that the flow lines in this case deflect to make the flow channels wider. In the opposite situation where the water seeps to more pervious zones, the flow channels are narrowed. If i_1 and i_2 are the angle of incidence and emergence to a normal to the inter-face of the two media then $\frac{\tan i_1}{\tan i_2}$ is directly proportional to the ratio of permeabiliy coeffts of these media i.e., K_1/K_2.

The pressures existing due to seepage flow can thus be found from equipotentiai lines and the stability of the bank analysed according to any of the numerous methods of slopes stability analysis which have been devised over the past half century.

Selection of side slopes and their stability analysis

(a) Side Slopes.

Although the stable slopes may vary depending on strength of the embankment soil and foundation condition. In Punjab side slopes provided are 1 : 3 on the river side of the bund and 1 : 2 on the land side. In Sind the side slopes are selected depending on the height of bund above ground. For bund of 8 ft. height or less the side slopes are 1 : 2 and for higher levels they are 1 : 3 on both sides.

As a general procedure it is good to make a first rough cost estimate/preliminary design on the above basis. These slopes should, however, be later modified after making detailed analysis keeping in view strength of the foundation and embankment materials, internal zoing together with the construction techniques that are selected. Variable slopes can be used in higher embankment constructed on softer foundations because for any given safety factor-against shear failure an embankment with minimum volume is usually obtained when side slopes are made steeper at the upper elevations and flatter near the bottom. Similarly depending on different foundation conditions and embankment heights, different designs may be followed along the various sections of the long bunds extending over many miles. Detailed analysis for stability of slopes should be made for high embankment sections to economise in the cost of construction.

(b) Stability of Slopes—Analysis.

When the ground is not level there are forces acting which tend to cause movement of soil from high to low points. These forces cause shearing stresses throughout the soil and a mass movement occurs unless the shearing resistance on every possible surface through the mass is of sufficient magnitude to withstand these shearing stresses. The most important of such force is the component of gravity that acts in the direction of probable-motion. Also important, but not so well recognised untill some years before, is the force of seeping water. The resisting force is the total cohesion and all friction that can be brought into play by normal pressures acting on the plane rupture.

In the stability analysis, in order to avoid unnecessary complications the following simplified conditions and assumption are used :

(1) An average or typical X-Section of unit dimension in the direction perpendicular to section is used. It is assumad that no shearing stresses act on the plane of section.

(2) Shearing strength of such individual soil occuring in the X-Section can be represented by :

$S = C_e + \sigma \tan \varnothing_e$

Where C_e & $\varnothing_e$ are effective cohesion and effective friction angle respectively that apply for each soil under the existing condition; and σ is the intergranular pressure on the failure plane. It may be either the intergranular pressure at failure or to which the soil has been consolidated, depending on whether drainage does or does not occur under the existing condition.

(3) Ground water/seepage conditions and pressures are represented by a given flow net.

(4) Embankment fails unless the resultant resistance to shear on every surface traversing the embankment is greater than resultant of all shearing forces exerted on that surface by the mass above.

(5) Before failure can occur the shearing strain at all points of the critical surface must be large enough to mobilize all available shearing strength.

The surface which is most liable to fail is called critical surface. Determination of its location is sometime simple but sometimes it requires a tedious process of successive trials. Sometimes there is a sharply defined surface of rupture as with brittle clays and dense sands but on other occasions as with soft clays failure occurs throughout a zone of plastic flow.

The most widely used method of

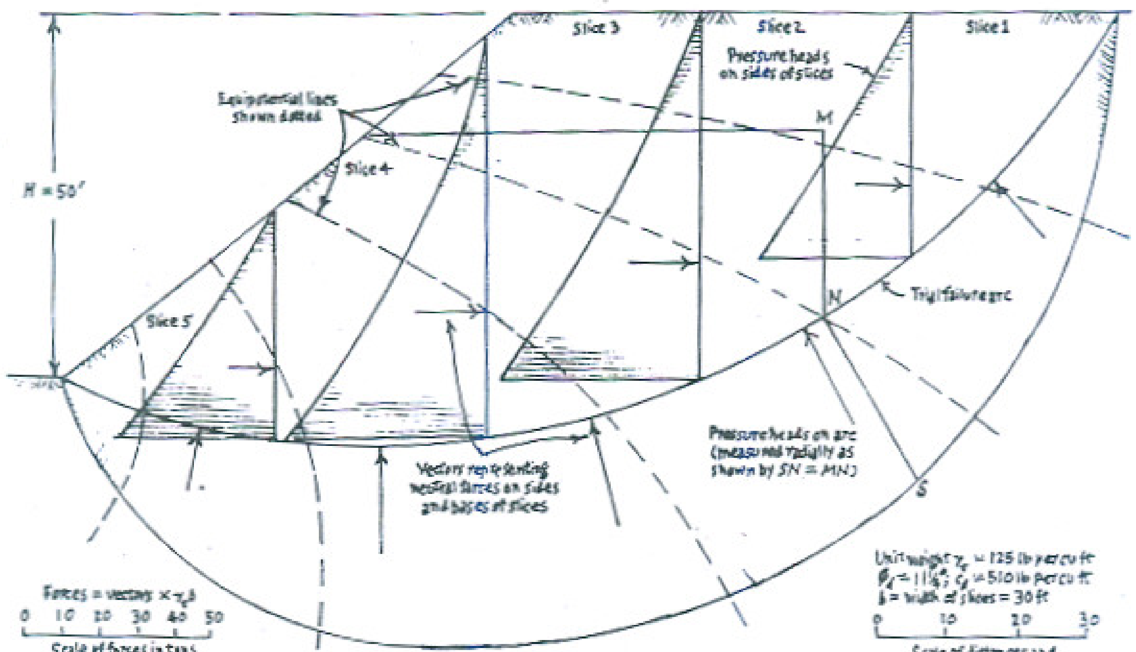
analysis of homogenious isotropic finite slopes is the swedish method based on circular failuresurfaces. The method of slics was developed by W. Fellenius. It is based on the statistical analysis of the mass above any trial failure area with this mass considered to be made up of vertical slices. The method is explained as ueder :

Let it be assumed that the circular arc on the section of fig. is an arbitrarily chosen trail arc. Steady state of seepage is represented by the equipotential lines, which are shown by dashed curves. This section has been arbitrarily divided into five vertical slices of equal width.

From the equipotential lines the neutral pressure may be determined at any point on the section. For example, at N the pressure head is MN. Point S is determined by setting the radial distance NS equal to MN. A number of points obtained in the same manner as S give the curved line through S which is a pressure head diagram. On the sides of the slices pressure head diagrams are also shown. Thus the forces acting on any slice are its total weight, the neutral forces acting on its sides and on its base and the resultant intergranular forces on its sides and on its base. In this example the developed strength characteristics $\varnothing d$ and Cd are assumed to be constant over the circular arc, and the intergranular forces on the base thus consist of the cohesion, the normal component of intergranular force Pn and the frictional resistance that is developed by Pn and that is equal to Pn tan $\varnothing$d.

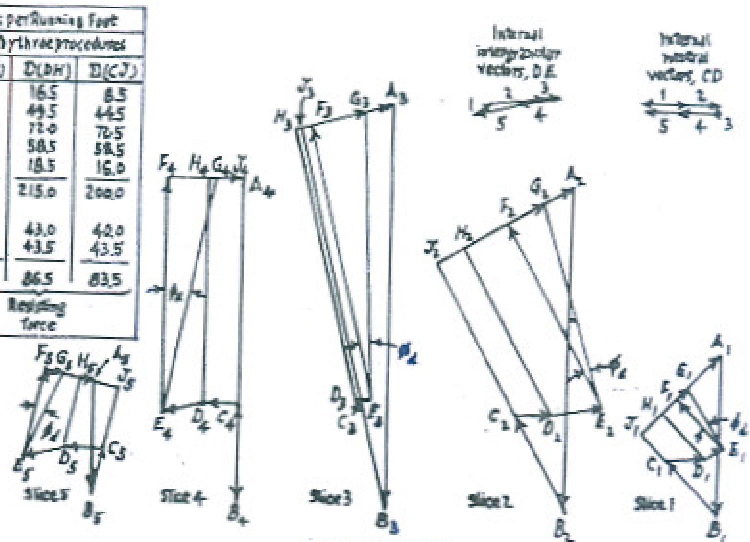
In (b) vector polygons are given which show all forces for all slices. The total weight of any slice is equal to the volume muliplied by unit weight $(G+e)$ $(1+e)$ γw and is represented by vector AB. The neutral force across the base is represented by vector BC, which acts normal to the arc. The combined neutral force on the two sides of the slice is represented by vector CD, acting horizontally. For the steady seepage case with the flow net known, vectors BC and CD are completely defined, and from the pressure head diagrams these vectors may be determined.

Vector DE represents the combination of intergranular force on the two sides of the slice. Since it is staistically in-determinate, in the figure merely arbitarrily chosen values that are reasonable, to obtain a solution is used. Nevertheless since total weight of the mass must be transmitted across the rupture arc, the sum of all boundary forces, including both neutral and intergranular components, must equal the total weight, regardless of the distribuion among the several slices. The assumption made is, therefore, an assumption relative to distribution only and fortunately the various possible distributions usually lead only to minor



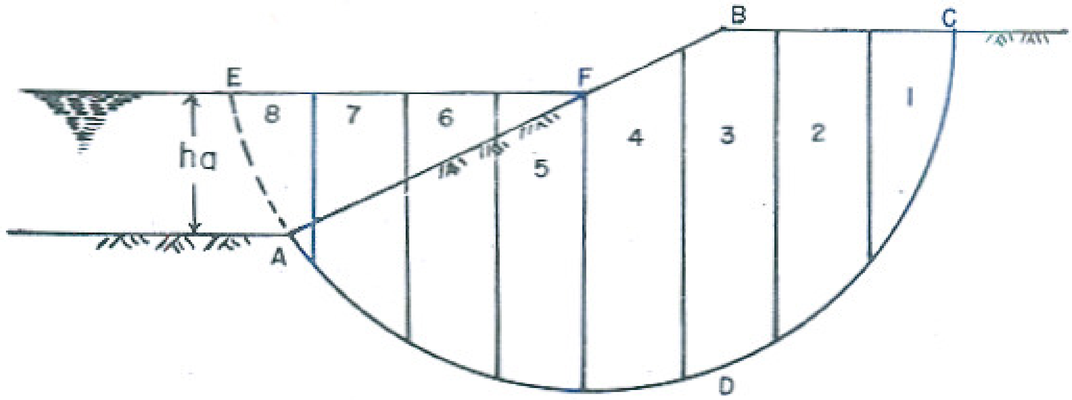
(a) Cross section showing slices and neutral forces

	Forces in Tons per Running Foot			
	ΣW_s	ΣP_N by three procedures		
Slice	= $\Sigma(AJ)$	$\Sigma(EF)$	$\Sigma(DH)$	$\Sigma(CJ)$
1	29.0	17.5	16.5	8.5
2	41.0	54.5	49.5	44.5
3	26.0	73.0	72.0	72.5
4	0.0	62.5	58.5	58.5
5	−7.0	23.5	18.5	16.0
Σ	89.0	229.0	215.0	200.0
$\tan\phi_d \Sigma P_N$		45.5	43.0	40.0
$c_d L_a$		43.5	43.5	43.5
	89.0	89.0	86.5	83.5
	Actuating force	Resisting force		



(b) Vector polygons

STABILITY OF SLOPES— THE METHOD OF SLICES



THE CASE OF PARTIAL SUBMERGENCE.

differences in the stability conditions.

The remaining forces are the normal and tangential components of the integranular forces across the arc. These forces are represented by EF and are commonly designated by Pn and Ps. Requirement for equilibrium is that shearing force Ps is supplied by the sum of the frictional force FG, which is equal to Pn tan $\varnothing$ d, and the cohesional force C_d, represented in the figure by GA. This may be written :

$$P_s = P_N \tan \varnothing_d + C_d$$

With respect to the mass as a whole the requirement for stability is expressed in terms of the equilibrium of moments about the center of the Circle. In the case under consideration boundary neutral pressures exist only on the circular arc and introduce no moment. Thus the actuating movement is the moment of the total weight. If weight AB is separated into shearing and normal components AJ and JB, designated respectively by Ws and Wn, the normal component has no moment, and the actuating moment may be written R Σ Ws in which R is the radius and the moment is clockwise.

For a correct consideration of the equilibrium of individual slices, forces CD and DE, which act on the sides of slices, must be included. However, relative to the mass as a whole they form closed polygons, as shown in the diagrams at the upper right in (b) and the summation of their moments must be zero. Force EF also has no moment about the center of the circle. Thus the only other forces introducting moment are FG and GA. Force FG gives a moment which may be expressed R Σ (FG), or R tan $\varnothing$ d Σ PN. Forces GA given a moment R Σ (GA) or Rc_d L_a, in which L_a is the length of the arc. These moments are counter clockwise, and together they form the resisting moment.

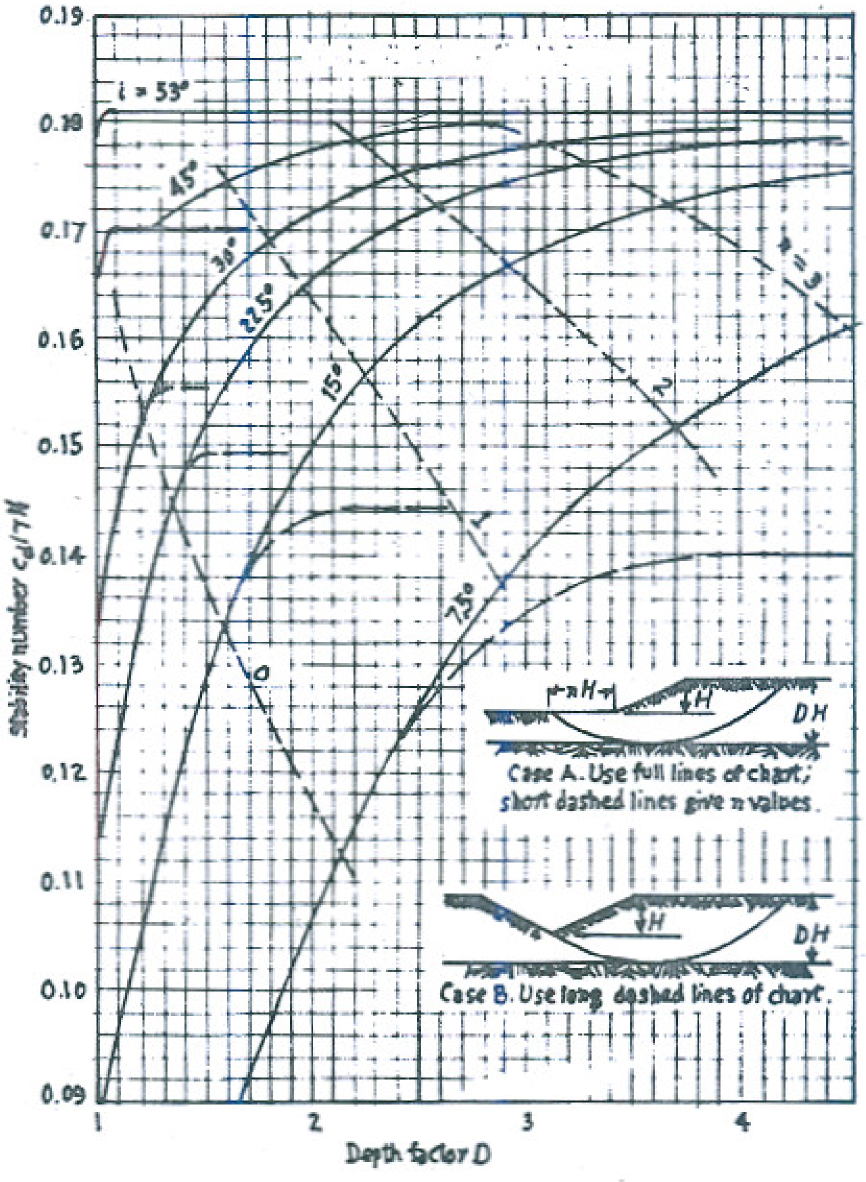
Equating the actuating and resisting moments gives the expression of moment equilibrium,

$$R \Sigma Ws = R. \tan \varnothing_d \Sigma P_N + RC_d \times L_a$$

The terms of this equation have the dimensions of forces, but since they are tangential summations they are not vector quantities in the ordinary sense.

The only term which offers any difficulty in the analysis is Σ PN. If the resultants of lateral intergranular pressures DE are correct the vectors EF, representing P_N, are correct. The table in the figures shows that the forces polygons are in equilibrium if vectors EF are accepted as representing forces : P_N. For each running foot of slope the actuating force W_s equals 89 tons ; the resisting force (tan $\varnothing_d P_N Cd\ L_a$) equals $0,189 \times 229$ tons$+510$ Lb. per sq. ft.$\times 171$ ft., which also equals 89 tons.

Fellenius proposed the use of the assumption that the laterial forces are



STABLITY NUMBERS FOR THE CASE OF ZERO FRICTION ANGLE AND LIMITED DEPTH.

(FOR $i < 54°$)

equal on the two sides of each slice. This assumption is apparantly incorrect if we consider individual slices but for the mass as a whole this assumption gives fairly reasonable results.

Ignoring determination of the neutral pressures on the sides of the slices, it saves a considerable amount of labour. Since it gives conservative results it is probably the more satisfactory procedure for general use.

Procedure for the case of partial submergence.

When partial submergence exists because of a pond or a stream adjacent to a slope the resultant boundry neutral force includes water pressures on the surface of the slope in addition to the water pressures across the rupture arc. A modification in the method discussed earlier will therefore be necessary. The failure mass is here assumed to include area AEF, which is within the water. Therefore the weight of water above AF must be included in the weight of slices 5 to 8 inclusive, Slices I to 4 will require no further revision of the procedure. Along the base of the slice 8 there is no sheering strength thus EA is not included in the arc length La in the cohesion determination and the normal component of the weight of slice does not enter the friction determination. However the tangential component of the weight must be included in two weights. It acts in counterclock-wise direction and gives appreciable aid to stability.

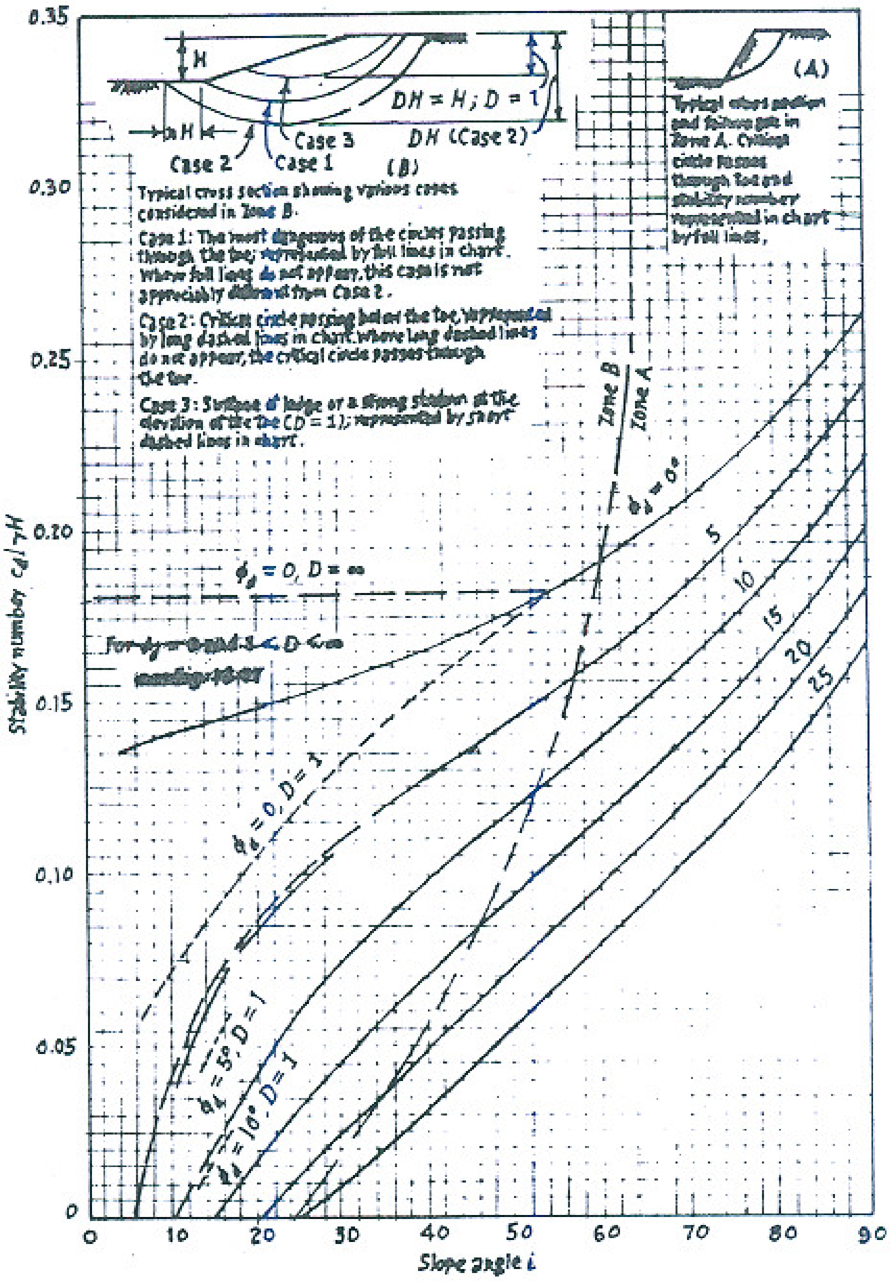
Simple, finite slopes without seepage.

Solution for simple X-Section of homogenious soils, with which no seepage is occurring may be obtained somewhat more easily and once obtained they can be made available in the form of relatively simple charts as given in the enclosed table and charts.

Compaction—Precaution during Construction.

To have maximum strength and imperviousness, to make an embankment free of excessive settlement by shrinkage (in case of granular soils, free from danger due to liquefaction) and to reduce gharas and rain cuts, it is necessary that such embankments must be compacted properly. Since rigid controls over manual labour is difficult under usual present day working conditions, proper and effective compaction cannot be ensured without mechanical means ; which must be used in addition to what ever manual effort can be expended in this direction. Departmental specification regarding removal of grass roots and other organic matter from the material brought for bund construction are however essentially to be enforced. Amongst the mechanical means, one of the most efficient type

Stability Numbers



STABLITY NUMBERS FOR VARIOUS

SLOPE ANGLES AND Ø d's

i > 54°

of the compacting equipment and as type which is now widely used is the sheeps-foot roller.

As the name implies the sheeps foot roller was a result of observation that the travel of animals over earth surfaces resulted in excellant compaction. More than 100 years ago in England herds of cattle and sheeps were actually used for earth compaction. Probably, the first sheeps foot roller was developed in California about 1905, when a wooden log 3 ft. in dia and 8 ft. long with rail road spikes for feet was adopted for road surface compaction. A steel sheeps foot roller was employed during construction of Drum Dam in 1907 and subsequently the use of similar rollers increased. The weights and dimension of sheeps foot rollers vary according to their use, i.e., in wider or confined sections ; and the tractive effort available i.e., animal of mechanical means. The current specification of USBR rollers. where high powered mechanical means are used, however, are :

1. The drums must be not less than 5 ft. in diameter and between 4 and 6 ft. long, with the space between adjacent drums from 12 to 15 in.
2. Each drum must be free to pivot about an axis parallel to the direction of travel.
3. One tamping foot must be provided for each 100 in^2 of drum surface.
4. The space between the feet must be equal to or greater than 9 in.
5. The length of the feet must be maintained at a minimum of 9 in.
6. The cross section of the feet must be equal to or less than 10 in,^{2} at a distance of 6 in, from the surface of the drum. It must be equal or greater than 7 in.^{2} but not greater than 10 in.^{2} at a distance of 8 in. from the surface.
7. The weight of the roller when fully loaded with sand and water must be not less than 4,000 lb/ft. of drum length.

Whether sheeps foot rollers be driven by motors and graders (or by animals in case of small size embankments) compaction is done at optimum water content.

When with a given method and effort of compaction a soil is compacted at some water content a unique density is obtained. The dry density of the compacted soil increases with increasing water content untill a maximum dry density results. At this point the dry density begins to decreases as the water content continues rising. This water content after which the density begins to decrease is called the optimum water content.

A sample at the optimum water content has nearly as large a degree of saturation as can be reached by compaction, because the limiting degree of saturation is reached when all small individual pockets of air within the pores of the soil become entrapped or surrounded by pore water.

If water is added to dry soil of some types and the soil is compacted before there has been time for the water to soak into it, the degree of compaction is some what smaller than it would be if more time were allowed.

It is interesting however that for any soil the optimum water content and maximum density vary considerably with different method of compaction. The higher the compactive effort the higher is the maximum density and the lower the optimum water content. On the wet side of optimum water content, there is maximum degree of saturation (or max. density at a given water content) which can be obtained practically with given type of compaction equipment, regardless of the compactive effort expended. Thus, an increase in compactive effort is more effective in increasing the density of soil when its water content is on dry side of optimum than on wet side. Therefore degree of compaction should be specified clearly keeping in view the type of equipment to be used and effort to be expended. On the same basis will depend the thickness of layers upto which soil can be placed. Proctor compaction test are used to determine the optimum water content in the laboratory,

In the field the construction of an embankment from compation point of view must be controlled in a manner to assure uniform results and such that the average properties obtained are atleast equal in quallity to the values assumed in design. Basic construction control is exercised by visual evaluation of :

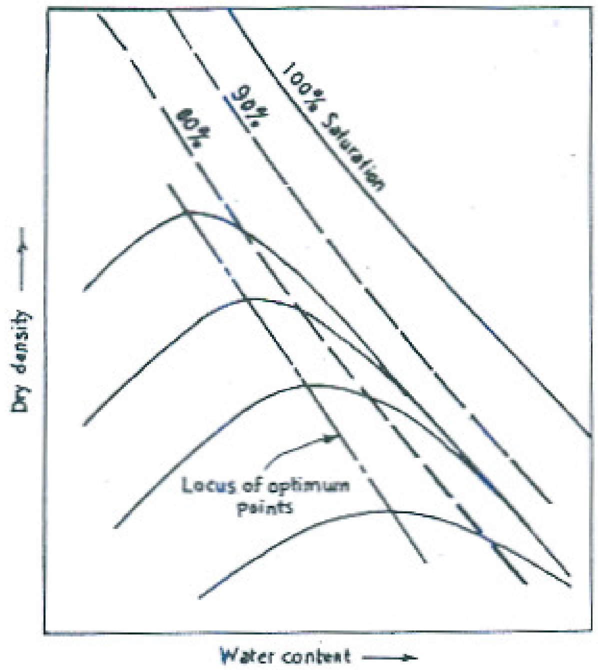
(a) The uniformity of the water content and the properties of the soil layers before compaction.

(b) The thickness of the compacted layers.

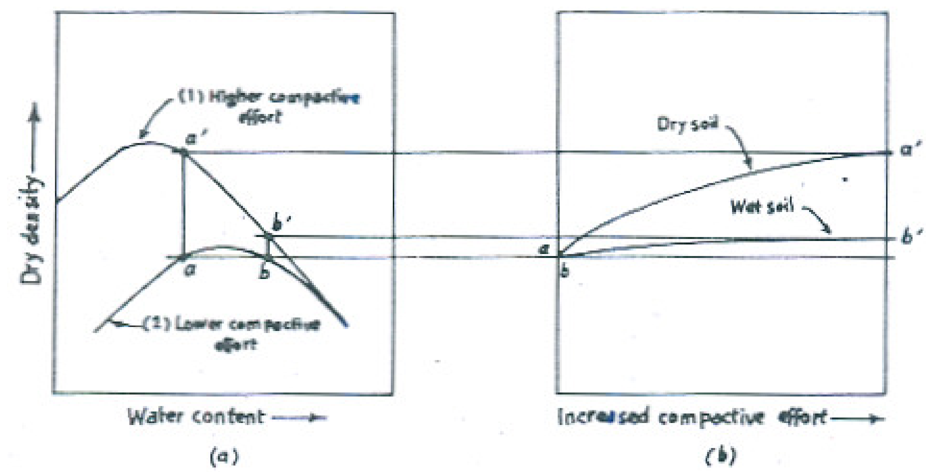
(c) The action of the roller and heavy hauling equipment on the construction surface.

In addition to above the heavy roller itself can also be considered as a testing apparatus. If the water content is uniform and layer is not too thick the action of roller will indicate whether the water content is satisfactory and whether required compaction is being obtained. If the sheeps foot roller continues churning up the construction surface the water content is too high for the roller. If water content is too low, for good compaction, the sheeps, foot roller will walkout several inches after only 3 or 4 passes.

During construction the control on moisture content, specially during



Typical family of density-water content curves for one soil obtained using the same compaction method and equipment but different compactive efforts.



Influence of increased compaction effort on a compacted soil at the same density and different water contents (using the same compaction equipment).

dry weather is particularly necessary because under the usual prevailing condition and practices in the country where water is to be carted from long distances for use on lengthy bunds, there may be tendency to ignore strict adherence to specification; and without water good compation cannot be obtained.

As the embankment is constructed field density tests on specimens which are often removed from the fill, at intervals, are performed. These specimens should be obtained at some depth and not from the surface of the fill. Most common test is the simple "caliberated sand test" in which volume of the hole from which the soil is taken can be measured by the volume of dry sand or water required to fill tne hole levelfull. From the volume and weight of a sample after drying, the unit weight and water content are easily obtained.

If the soil were at the optimum water content but the optimum density is not obtained it is possible that enough roller passes have not been made.

Over compaction is undesirable condition specially when the foundations are relatively weak. In such cases lateral swelling of the over compacted soil may introduce shearing strains and these may distrub the structure in the soil of the foundation.

For pervious embankment section consisting of sand or sand and gravel etc. less laboratory testing is required because the strength of compacted sand does not vary greatly with small changes in the density and good compaction can be obtained without close water content control necessary for impervious soils. Pervious materials are in fact best compacted with an excess of water and viberations.

The only field tests usually carried out for pervious section are density control tests. Relative density R.D. (%) is defined :

$$FD\,(\%) = \frac{e\ max - e}{e\ max - e\ \ min}$$

Where e max = void ratio of the soil in the loosest possible state.

e min = Void ratio of the soil in the denset possible state.

e = Void ratio of the soil in the embankment.

In term of dry density the equation becomes :

$$R.D.\,(\%) = \frac{e\ max - e}{e\ max - e\ \ min.} =$$

$$\frac{.\ D\ max\ (D - D\ min)}{D\ (D\ max - D\ min)} \times 100$$

Where D max—the dry density (lb./ft. 3) of the soil in its densest possible state.

D min—the dry density of the soil in its loosest possible state.

D —the dry density of the soil in the embankment as measured in the field density test.

The max. dry density (D max) is obtained by viberating the soil in a completely saturated, state in a large laboratory container with a small hand viberator. The minimum density (D min) is obtained by pouring the oven dried material into a container in such a way as to result in the least possible densification.

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STABILITY NUMBERS FOR HOMOGENEOUS SIMPLE SLOPES WITHOUT SEEPAGE—BY SEVERAL METHODS

Values of Cd/rH

		If critical circle passes through toe of slope				If critical circle passes below the toe.
(1) i	(2) ø d.	(3) Culmenn (Plane)	(4) Slices	(5) ø Circle	(6) Legarithmic spiral.	(7) Cd/rH
90	0	0.250	0.261	0.261	0.261	0.261
	5	0.229	0.239	0.239	0.239	0.239
	10	—	—	—	—	0.218
	15	0.192	0.199	0.199	—	0.199
	20	—	—	—	—	0.182
	25	0.159	0.165	0.166	0.165	0.166
75	0	0.192	0.219	0.219	0.219	0.219
	5	0.171	0.196	0.195	—	0.195
	10	—	—	—	—	0.173
	15	0.134	0.154	0.152	—	0.152
	20	—	—	—	—	0.134
	25	0.102	0.118	0.117	—	0.117
60	0	0.144	0.191	0.191	—	0.191
	5	0.124	0.165	0.162	0.162	0.162
	10	—	—	—	—	0.138
	15	0.088	0.129	0.116	0.116	0.116
	20	—	—	—	—	0.097
	25	0.058	0.082	0.079	0.078	0.079
45	0	0.104	(0.170) I	(0.170)	(0.170)	0.170
	5	0.083	0.141	0.136	—	0.136
	10	—	—	—	—	0.108
	15	0.049	0.085	0.083	—	0.083
	20	—	—	—	—	0.062
	25	0.023	0.048	0.044	—	0.044
30	0	0.067	(0.156)	0.156	0.156	0.156
	5	0.047	0.114	0.110	—	0.110
	10	—	—	—	—	0.057
	15	0.018	0.048	0.046	—	0.046
	20	—	—	—	—	0.025
	25	0.002	0.012	0.009	0.008	0.009
15	0	0.033	0.145	0.145	0.145	0.145
	5	0.015	0.072	0.068	0.068	0.068
	10	0.004	—	0.023	—	0.023

Use of Saline Water For Irrigation

By

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Effects of different saline waters with varying depths of irrigation applications on soil conditions and crop growth in a set of crop rotations have been studied and the results achieved from them are reported here.

Salts are added into the soil with irrigation water. Their amount is different depending on the source from which it is obtained. About ·6 tons of salts are added with the application of two acre feet of canal water in the Punjab for which only a minor fraction provided for leaching keeps the land in fit condition for crop production.

The canal supply being limited, the use of subsoil water to grow the crops has shown much increase in the recent past. More than one lac tubewells have been installed in the province ; both in the Public and Private sectors, to get the regular supply of ground water which always contains more salts than the canal water. Their effect on soil conditions and crop growth need careful investigation for which the monitoring studies have been in progress in different SCARPS. The work was also under taken in the Laboratory with waters of similar qualities as are obtained from the tubewells. They were prepared by mixing the salts in the local tubewell water. The results of the experiments conducted are discussed here.

Review of Literature.

Hussain observed that the water with high residual sodium carbonate

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and low concentration of salts had injurious effect on soil conditions, the adverse effect was faster when both the components : the salt concentration and RSC was high. The deterioration period could be prolonged with increase in depth of irrigation application. The hazardous effects were more pronounced when a water of poor quality was used on normal soils.

Haider and Forooqi reported the deterioration in the soils where water having TDS more than 750 PPM was used. The extent and degree of degradation of the soil was progressively more under the increasingly poor quality of the water applied.

Hussain et al found that in a period of nine years the salinity level of soils was reduced considerably or it remained unchanged with the application of a group of water having EC varying from 750 to 2000 micromhos/cm. It, however, increased appreciably in other two cases where the EC of the water used were 2000 to 3000 micromhos/cm. They observed further that in case of marginal or poor quality waters, the growing of sensitive crops were eliminated in a period of 9 years from the area where it was used, the yield of the tolerant crops were also adversely affected.

Kelly observed that natural precipitation might leach salts that accumulate as a result of applying a saline water. If the amount of precipitation is sufficient to penetrate the soils to depth below the root zone, the accumulation of salts might be largely prevented thereby. With waters relatively high in sodium percentage the periodic application of gypsum is advisible.

Khan and Rana pointed out that the waters having TDS from 1616 to 1832, SAR from 18 to 19 and RSC from 9.39 to 11.12 were not at all safe for use for irrigation.

Material and Methods.

The experiment was conducted in specially designed plots 3′×3′ and 5′ deep, the side walls of which were made of pacca bricks, and plastered with cement on which a covering of bitumen was given to avoid any side leakage of moisture. They contained permeable type of fine sandy loam soil in them which had the direct contact with the earth at the bottom to allow free upward and downward movement of moisture. The soil was analysed before starting the experiment and after completing the same.

The following crop rotations were tried :

(1) Maize-Berseem-Cotton-Sorghum-Gram.

(2) Maize-Melilot-Sugarcane-Wheat.

(3) Maize - Alfalfa - Wheat-Sorghum - Gram.

The waters used for irrigation had EC, 860, 2000, 2500 and 3000 micromohs/cm. The analysis results of which are given below :

Table No. I.

ANALYSIS RESULTS OF WATER USED FOR IRRIGATION

Conductivity in micromohs/cm at 20°C	PH	MILLI-EQUIVALENT PER LITRE							Total Cations	T.S. by evaporation ppm	Res. Na_2 CO_3 meq per litre	SAR
		Ca	Mg	Na	CO_3	HCO_3	Cl	SO_4				
850	8.38	1.70	2.50	5.40	—	6.90	1.9	0,70	9.60	543.8	2.70	3.72
2000	8.40	0.09	1.55	18.00	—	1.90	9.1	8.60	19.60	1431.0	0.30	19.98
2500	8.40	0.10	1.60	22.35	—	2.45	11.4	10.20	24.05	1788.7	0.75	24,13
3000	8.45	0.15	1.85	26.80	—	2.90	14.2	11.70	28.80	2146 5	0.90	26.80

The depths of irrigations tried were 3", 4", and 6" with two replications in each case. All the agriculture practices employed by the farmers were adopted here.

Results and Discussion.

Table No. II shows the irrigation delta applied for various treatments in different crop rotations. Considerationg 3" deep irrigation as control, where the chances of Leaching were limited 33% and 100% more water was given with 4" and 6" deep irrigation applications respectively. It was expected that the accumulation of salts in the rootzone especially in case of water of high salt content and with shallow irrigation applications could not be avoided, but the analysis results of the soil given in table No. III differed from this concept. It was due to the high rainfalls received during the growth period of some of the crops. In the moonsoon season the rainfalls were had in reasonably high amounts after short intervals of time and a large amount of them passed below the rootzone. So often a heavy rainfall was received when the crop had already been irrigated and the moisture in the soil was almost at the field capacity level. Thus nearly whole of it passed down below the root-zone. The observation is supported by Kelly. According to which the natural precipition might leach the salts that accumulate as a result of applying saline water. If the precipitation is sufficient to pene-

THE TABLE NO. 11 SHOWING THE IRRIGATION DELTA APPLIED AND THE RAINFALL RECIEVED DURING THE GROWTH PERIOD OF CROPS IN INCHES

EC—10^6	Depth of	Maize			Berseem			Cotton			Chari			Toria		
of I. W.	Irri.	Rainfall	Irri.	Total	Rainfall	Irri.	Total	Rainfall	Irri.	Total	Rainfall	Irri.	Total	Rainfall	Irri.	Total
850	6"	6.21	48.00	54.21	1.59	42.00	-43.59	11.04	36.00	47.04	23.94	18.00	41.91	2.48	18.00	20.48
	4"	6.21	32.00	38.21	1.59	28.00	29.59	11.04	24.00	35.04	23.94	12.00	35.94	2.48	12.00	14.48
	3"	6.21	24.00	30.21	1.59	21.00	22.59	11.04	18.00	29.04	23.94	9.00	32.94	2.48	9.00	11.48
2000	6"	6.21	48.00	54.21	1.59	42.00	43.59	11.04	36.00	47.04	23.94	18.00	41.94	2.48	18.00	20.48
	4"	6.21	32.00	38.21	1.59	28.00	29.59	11.04	24.00	35.04	23.94	12.00	35.94	2.48	12.00	14.48
	3"	6-21	24.00	30.21	1.59	21.00	22.59	11.04	18.00	29.04	23.94	9.00	32.94	2.48	9.00	11.48
2500	6"	6.21	48.00	54.21	1.59	42.00	43.59	11.04	36.00	27.04	23.94	18.00	41.94	2.48	18.00	20.48
	4"	6.21	32.00	38.21	1.59	28.00	29.59	11.04	24.00	35.04	23.94	12.00	35.94	2.48	12.00	14.48
	3"	6.21	24.00	30.21	1.59	21.00	22.59	11.04	18.00	29.04	23.94	9.00	32.94	2.48	9.00	14.48
3000	6"	6.21	48.00	54.21	1.59	42.00	43.59	11.04	36.00	47.04	23.94	18.00	41.94	2.48	18.00	20.48
	4"	6.21	32.00	48.21	1.59	28.00	29-59	11.04	24.00	35.04	23.94	12.00	35.94	2.48	12.00	14.48
	3"	6.21	24.00	30.21	1.59	21.00	22.59	11.04	18.00	29.04	23.94	9.00	32.94	2.48	9.00	11.48

EC—10^6	Depth of	Maize			Senji			Sugarcane			Guara			Wheat		
of I. W.	Irri.	Rainfall	Irri.	Total	Rainfall	Irri.	Total	Rainfall	Irri.	Total	Rainfall	Irri.	Total	Rainfall	Irri.	Total
850	6"	6.21	48.00	54.21	0.86	24.00	24.86	13.34	132.00	145.34	24.48	24.48	48.48	2.59	36.00	38.59
	4"	6.21	32.00	38.00	5.86	16.00	16.86	13.34	88.00	101.34	24.48	16.00	40.48	2.59	24.00	26.59
	3"	6.21	24.00	30.21	0.86	12.00	12.86	13.34	66.00	79.34	24.48	12.00	36.48	2.59	18.00	20.59
2000	6"	6.21	48.00	54.21	0.86	24.00	24.86	13.34	132.00	145.34	24.00	24.00	48.48	2.59	36.00	38.59
	4"	6.21	32.00	38.21	0.86	16.00	16.86	13.34	88.00	101.34	24.48	16.00	40.48	2.59	24.00	26.59
	3"	6.21	24.00	30.21	0.86	12.00	12.86	13-34	66.00	79.34	24.48	12.00	36.48	2.59	18.00	20.59
2500	6"	6.21	48.00	53.21	0.86	24.00	24.86	13.34	132.30	145.34	24.48	24.00	48.48	2.59	36.00	38.59
	4"	6.21	32.00	38.21	0.86	16.04	16.86	13.34	88.00	101.34	24.48	16.00	40.48	2.59	24.00	26.59
	3"	6.21	24.00	30.21	0.86	12.00	12.86	13.34	66.00	79.34	24.48	12.00	36.48	2.59	18.00	20.59
3000	6"	6.21	48.00	54.21	0.86	24.00	24.86	13.34	132.00	145.34	24.48	24.00	48.48	2.59	36.00	38.59
	4"	6.21	32.00	38.21	0.86	16.00	12.86	13.34	88.00	101.34	24.48	16.00	40.48	2.59	24.00	26.59
	3"	6.21	24.00	30.21	0.86	12.00	12.86	13.34	66.00	79.34	24.48	12.00	36.48	2.59	18.00	20.59

EC—10^6	Depth of	Maize			Lucern			Wheat			Chari			Gram		
of I. W.	irri.	Rainfall	Irri.	Total	Rainfall	irri.	Total	Rainfall	Irri.	Total	Rainfall	Irri.	Total	Rainfall	Irri.	Total
850	6"	6.21	42.00	48.21	1.60	54.00	55.60	2.81	48.00	50.81	23.94	24.00	47.94			
	4"	6.21	28.00	34.21	1.60	56.00	37.60	2.81	32.00	34.81	23.94	16.00	39.94		Failed	
	3"	6.21	21.00	27.21	1.60	27.00	28.00	2.81	24.00	26.81	23.84	12.00	35.94			
2000	6"	6.21	42.00	48.21	1.60	54.00	55.64	2.81	48.00	50.81	23.94	24.00	47.99			
	4"	6.21	28.00	34.21	1.60	36.00	37.60	2.81	32.00	34.81	23.94	16.00	39.94			
	3"	6.21	21.00	27.21	1.60	27.00	28.60	2.81	24.00	26.81	23.94	12.00	35.94			
2500	6"	6.21	42.00	48.21	1.60	54.00	55.60	2.81	48.00	50.81	23.94	24.00	47.94			
	4"	6.21	28.00	34.21	1.60	36.00	37.60	2.81	32.00	34.81	23.94	16.00	39.94			
	3"	6.21	21.00	27.12	1.60	27.00	28.60	2.81	24.00	26.81	23.94	12.00	35.94			
3000	6"	6.21	42.00	48.81	1.60	54.00	55.60	2.81	48.00	50.81	23.94	24.00	47-94			
	4"	6.21	28.00	34.21	1.60	36.00	37.60	2.81	32.00	34.81	23.94	16.00	39.94			
	3"	6.21	21.00	27.21	1.60	27.00	28.60	2.81	24.00	25.81	23.94	12.00	35.94			

TABLE NO. III SHOWING THE ANALYSIS RESULTS OF SOIL BEFORE AND AFTER (B/A) A CROP ROTATION

IRRIGATION 6″

EC $\times 10^6$ (I.W)	Depth of Soil	R_1 EC $\times 10^3$ B/A	R_1 SAR B/A	R_2 EC $\times 10^3$ B/A	R_2 SAR B/A
850	1	1.5/2.9	8.0/12.5	1.45/2.1	2.34/5.65
	2	1.4/3.1	6.3/13.4	2.1/1.2	3.87/3.92
	3	2.5/3.0	1.0/12.8	1.5/3.10	4.90/9.7
	4	3.4/4.25	4.94/16.49	3.5/3.5	3.78/15.5
	5	3.1/5.5	3.2/17.0	2.8/3.8	5.44/16.0
2000	1	3·1/3.0	12.1/1.5	1.6/2.6	1.94/2.4
	2	3.7/2.0	5.12/6.0	2.1/2.5	3.9/5.65
	3	3.6/1.6	9.5/12.8	2.0/2.4	2.85/7.52
	4	3.9/1.9	6.08/20.3	2.5/2.20	4.68/8.1
	5	3·4/2.4	5.44/19.2	2.2/1.9	2.85/37.0
2500	1	2.2/3.4	12.0/17.4	1.55/2.6	3.05/17.0
	2	2.2/3.4	12.6/20.0	1.7/2.6	2.7/17.0
	3	2.0/3.5	13.0/18.0	1.45/2.1	1.03/19.0
	4	1.55/-	3.5/37.0	2.2/3.0	5.4/18.5
	5	1.6/3.0	4.0/5.1	1.4/2.1	2.35/28.1
3000	1	1.9/2.5	5.72/7.5	3.8/4.5	16.1/10.0
	2	3.0/2.9	14.49/14.1	3.6/2.8	13.7/15.7
	3	1.7/2.0	9.23/12.0	3.6/3.1	12.69/25.4
	4	2.0/2.9	10.23/32.0	1.0/3.0	5.74/24.5
	5	7.5/3.8	11.7/36.0	3.2/3.0	13.25/23.6

The analysis results of soil were almost similar in all the crop rotations. Hence they have been reported only for one crop rotation.

The PH of the soil varied generaly from 8.3 to 8.5. So it is not given here.