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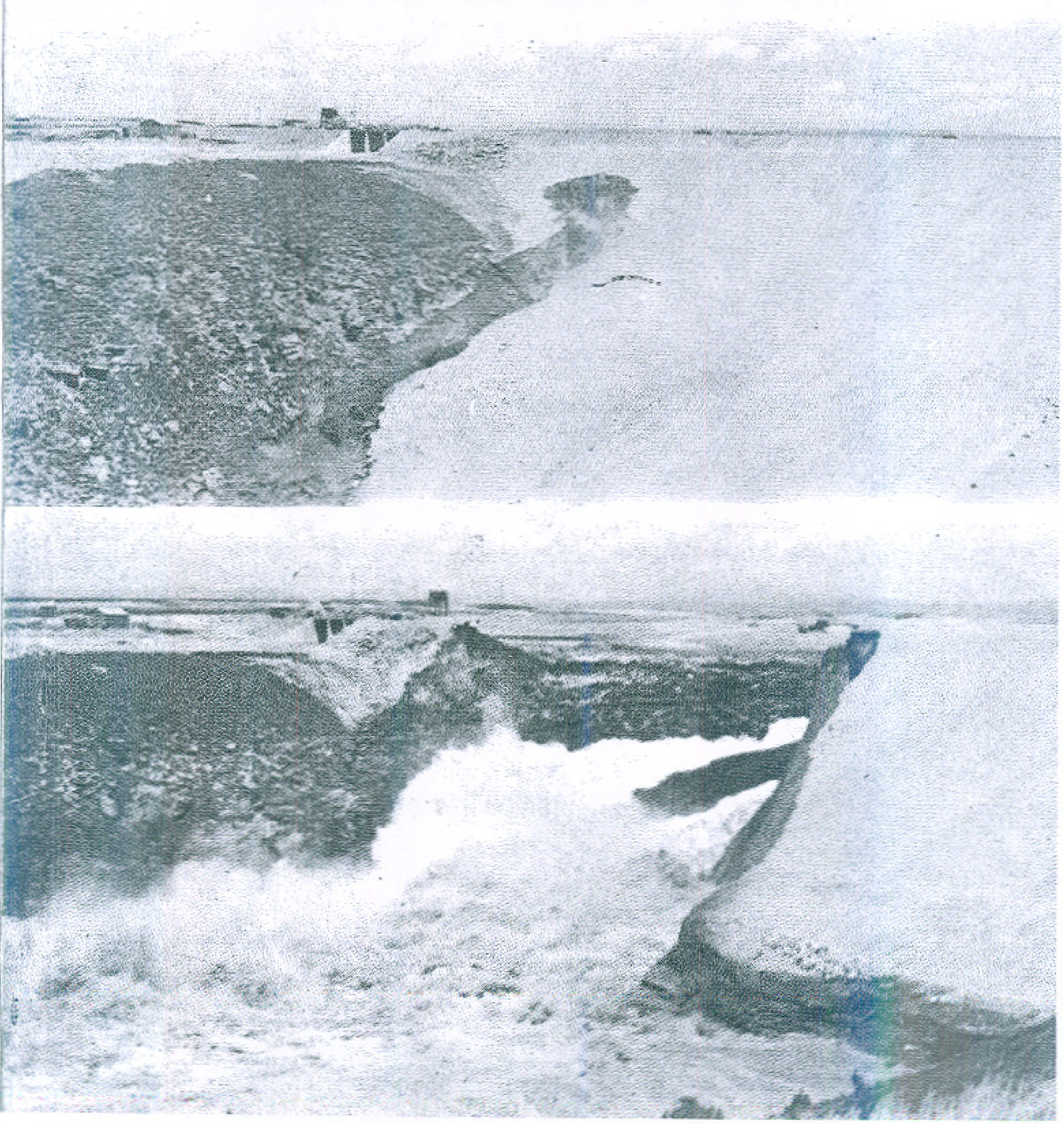

CODE OF ETHICS

PAKISTAN ENGINEERING CONGRESS

بِسْمِ اللّٰهِ الرَّحْمٰنِ الرَّحِيْمِ

In the name of God, the Beneficent, the Merciful.

WHEREAS Allah enjoineth upon his men to faithfully observe their trusts and their covenants ;

that the practice and profession of engineering is a sacred trust entrusted to those whom Nature in its magnificent bounty has endowed with this skill and knowledge ;

that every member of the profession shall appreciate and shall have knowledge as to what constitutes this trust and covenant, and

that a set of dynamic principles derived from the Holy Quran shall guide his conduct in applying his knowledge for the benefit of society.

Now, therefore, the following Code of Ethics is promulgated. It shall be incumbent upon the members of the Pakistan Engineering Congress to subscribe to it individually and collectively to uphold the honour and dignity of the engineering profession :

۱- إِنَّ اللّٰهَ يَأْمُرُكُمْ أَنْ تُؤَدُّوا الْأَمٰنٰتِ
إِلٰى أَهْلِهَا ۙ وَإِذَا حَكَمْتُمْ بَيْنَ النَّاسِ
أَنْ تَحْكُمُوا بِالْعَدْلِ ۚ إِنَّ اللّٰهَ نِعِمَّا
يَعِظُكُمْ بِهٖ ۗ

"Allah commands you to render back your trusts to those to whom they are due, and that when you judge between people, you judge with justice. Allah admonishes you with what is excellent". iv : 58

1. You shall be honest, faithful and just, and shall not act in any manner derogatory to the honour, integrity or dignity of the engineering profession.

۲- أَوْفُوا الْمِكْيَالَ وَالْمِيزَانَ بِالْقِسْطِ وَلَا تَبْخَسُوا
النَّاسَ أَشْيَاءَهُمْ وَلَا تَعْثَوْا فِي الْأَرْضِ
مُفْسِدِينَ ○

"Give full measure and weight justly and defraud not men of their things, and act not corruptly in the land making mischief". xi : 85

2. You shall use your knowledge and skill of engineering for human welfare, and render professional service and advice which reflects your best professional judgment.

۳- وَلَا يَجْرِمَنَّكُمْ شَنَاٰنُ قَوْمٍ عَلٰٓى أَلَّا تَعْدِلُوا ۚ
اِعْدِلُوا ۟ هُوَ أَقْرَبُ لِلتَّقْوٰى ۫

"And let not hatred of a people incite you not to act equitably. Be just ; that is nearer to observance of duty". v : 8

3. You shall not injure maliciously, directly or indirectly, the reputation or employment of another Engineer, nor shall you fail to act equitably while performing professional duty.

۴- أَوْفُوا بِالْعُقُودِ ○

"Fulfil the obligations". v : 1

4. You shall faithfully observe and fulfil all your obligations.

TWENTY SIXTH YEAR OF PUBLICATION

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TITLE

Failure of
Teton Dam, (IDAHO STATE) U.S.A.
on June 5, 1976.
Top Photograph 11.30 a.m. just starting
Lower Photograph Early afternoon, the same day

HYDRAULIC FAILURES IN EARTH DAMS
and
DAM COLLAPSE WAVE

By
AMJAD KHAN*

PREFACE

This Paper highlights different type of hydraulic failures in Earth Dams with examples of some major disasters. This paper also gives brief introduction on Flood Prediction Formula and Flood Wave Generation as a result of such failures.

INTRODUCTION

The building of dam is as old as the recorded history of mankind. Traces of these ancient dams are found today in both the old and the new worlds and bear witness to the achievement of civilization which have long passed away. Nevertheless, most of the procedures which constitute modern practice for design and construction have evolved from the practical efforts to eliminate weaknesses exposed by unsatisfactory performances of older dams. A knowledge of the principal lessons learned from failure and damages in the past is an essential part of the training of a good dam designer.

The earliest record of dam failure relates to an earth embankment dam near Grenoble which failed in 1219 after 28 years service. A list of 1764 dams has been published in America, giving all those built up to 1959. Of these 33 had failed between 1918 and 1958 five of these failures were classified as major disasters involving the loss of 1680 lives. Most of the failures were of earth embankment dams. In 1961 the Spanish Publication "Revist de abras Publicas" mentioned about 308 dams which were failed. Out of these 308, 163 were classified as the failures in earth embankment dams.

The causes of the failure of dams, listed were as follows:

— Foundation failure	40%
— Failure due to inadequate spillway capacity	25%
— Failure due to poor construction	12%
— Failure due to sliding	2%
— Failure due to uneven settlement	10%
— Failure due to defective material	2%
— Failure due to in-correct operation	2%
— Failure due to acts of war	3%
— Failure due to earthquakes	1%

1. FAILURES

Hydraulic failures in earth dams are very common. Earth dams are low in height and long. Any small failure is potentially dangerous and can quickly enlarge due to high erosive powers of the discharging water

1.1 FAILURE DUE TO OVERTOPPING

The most common cause of complete catastrophic failure of dams has been the water.

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flowing over the top of the dam, during the great river floods when the spillway capacities were inadequate. Although there have been some cases where overtopping did not result in complete failure of the dam. It must be assumed that earth dam cannot be designed safely to withstand the erosive action of water passing over the top of the crest.

Nearly 60% of the failures in dams occur due to overtopping. The extend of damage (flooding etc.) resulting depends greatly upon the duration of the overtopping. Short periods may result in only minor damages to the dam, but once the initial breach is started on the dam crest then the complete erosion upto the foundation level can be expected. The sequence of events are as hown in Fig. 1. The erosion process, once started, forms triangular shape Fig. 1(a), cutting down the embankment with apex angle constant, until foundation level is reached Fig 1(c). The erosion process then extends laterally, changing the triangular shape, first to trapezoidal shape, and then to a rectangular shape Fig. 1(d) and 1(e). Now we will look in detail, about the dams which failed due to overtopping after and during construction and the extent of damage caused by these failures.

KELLEY BARNES DAM

One Sunday morning in November 1977, a 40 years old dam built in Toccoa, Georgia USA, bursted and flood wave was generated killing 39 people on the Toccoa falls College Campus, 3 km down stream and devasted large property. The cause of the dam failure as described by residents is that dam was leaking for 2 or 3 years and no remedial measures were taken and before two days of the dam failure, a stream of water about the size of the arm was running from the down-stream face. But college officials say that the dam was regularly checked. A day before, the dam was checked because of the fear that heavy rain - 50 mm in the previous six hours might have caused a sudden rise in the lake level but officials found the water about two metre below crest level and no signs of leakage any where.

The six hours after that there was heavy rain storm and finally the dam failed. A preliminary repot on possible causes of failure, prepared for the President of U.S.A. showed, overtopping as main cause of failure.

OROS DAM

On 25th March, 1960, the earthfill dam at Oros in the State of Ceara in Brazil, which was still under construction, was over-topped by a flood and in 15 hours the major part of the embankment was washed away. Between 21st and 25th March the heavy rains caused the flow of river Taq to rise to 80,000 cusecs (ft^3/sec) of which only one fifth could be dealt by the diversion channel. A critical situation arose and it became impossible to raise the dam with necessary speed. Events soon became out of control with the result that water flowed over the top of the embankment. Over 1 million cubic yard of fill was washed away in a few hours leaving a gap of 660 ft wide. The flood waters were discharged down the Taquaribe Valley and in three days reached the Atlantic 210 miles away.

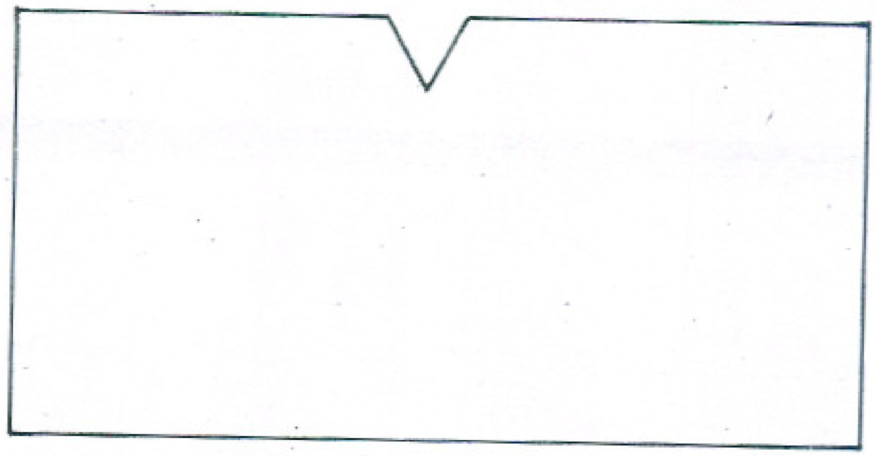
The causes of failure were overtopping and slow speed of construction. It was also discovered that wrong estimate of max future flood was made.

1.2 FAILURE OF DAMS DUE TO SLIDING

Slides, which are one of the frequent causes of dam failure, occur in earth dams in the same

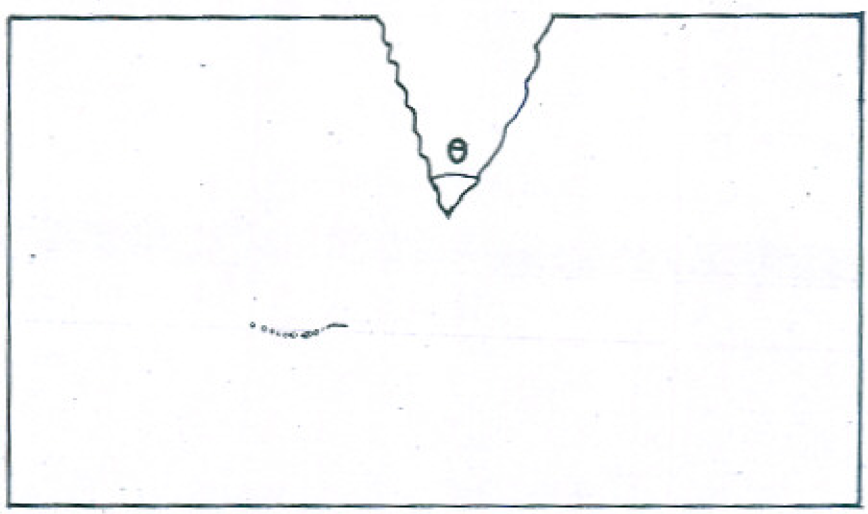
BREACH SHAPES OF EARTH EMBANKMENTS RESULTING FROM OVERTOPPING

(a)



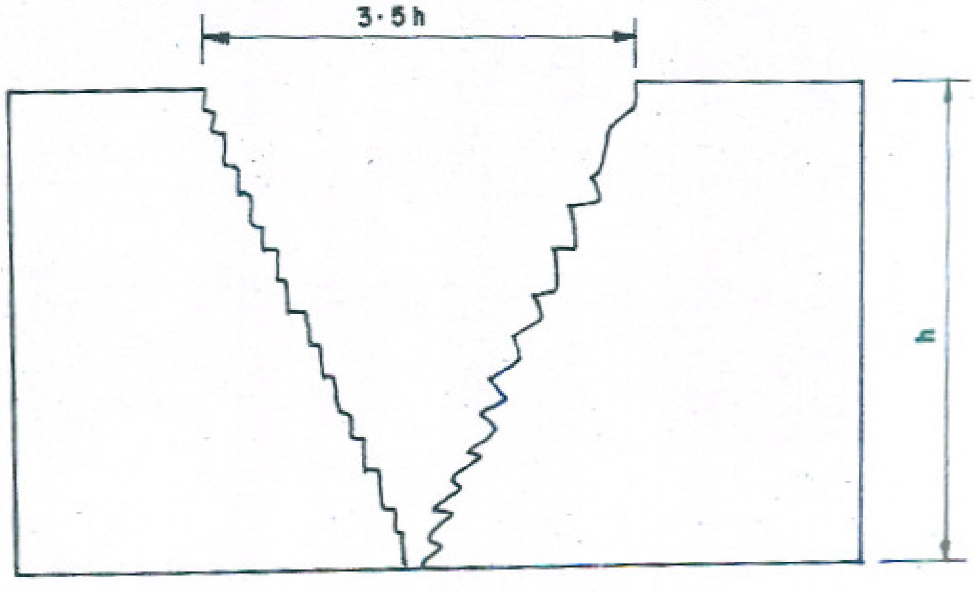
INITIAL NOTCH FORMS

(b)



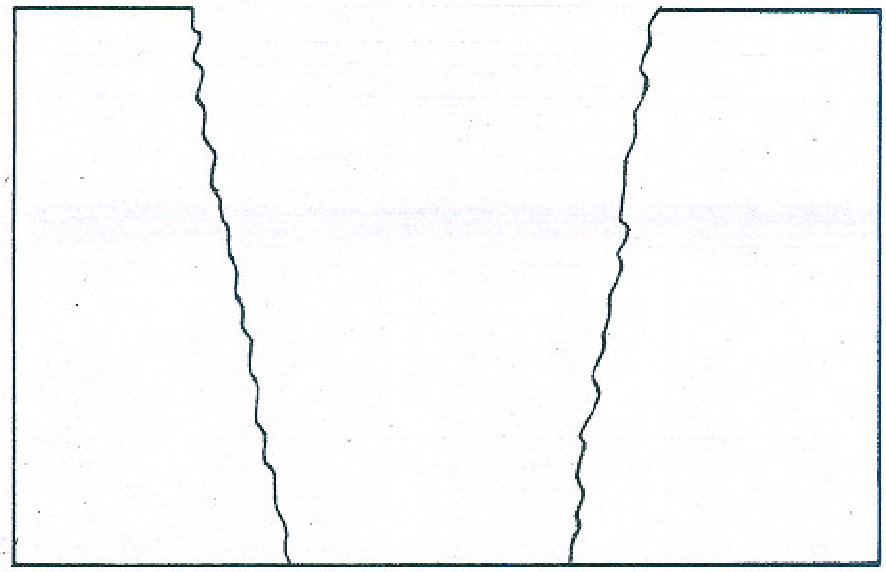
EROSION CONTINUES WITH θ CONSTANT

(c)



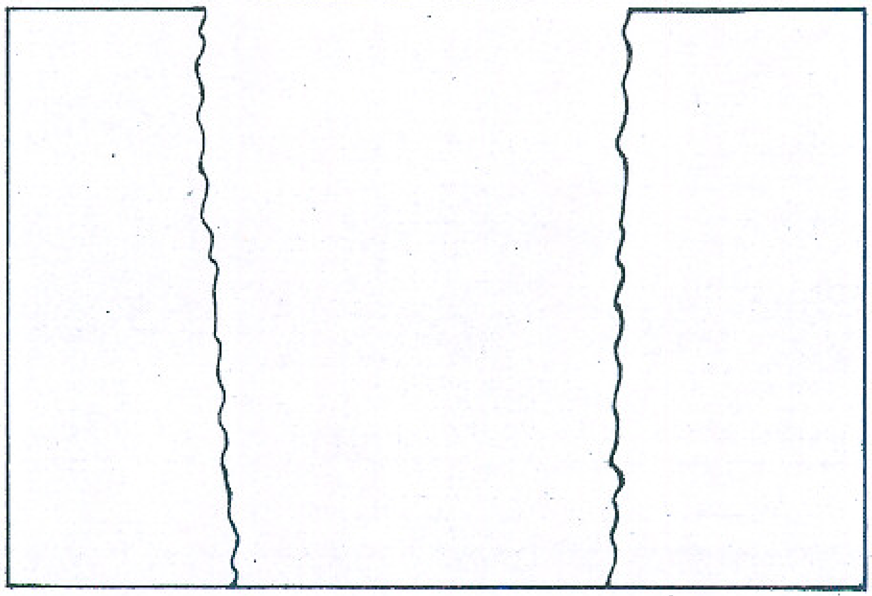
FOUNDATION LEVEL IS REACHED

(d)



ERODSION PROCEEDS LATERALLY

(e)



ULTIMATE FAILURE SHAPE

way that land slides develop in natural earth slopes when the average stress along any potential sliding surface becomes greater than the average strength. Middle brook's summary of records of earth dams with unsatisfactory performance, give an Instructive statistical view of the chronology of slides. As seen from the Table-1, nearly one-third of all the slides developed within the first year after construction, and more than half took place within 5 years. Table-2 shows that 94% of the slide failure in the study occurred before 1940. Because a large proportion of existing earth dams have been built since 1940, it is apparent that present design and construction practices have almost eliminated the likelihood of slides.

Most of histories available are at least 20 years old, although there have been a few recent failures by sliding in poorly constructed dams, dams with very soft foundations, and dams built number of years ago.

Slides in dams can be grouped into three categories'

i. Slides during construction involving the upstream or down stream slope (or both)
ii. Slides on the down stream slope during reservoir operation.
iii. Slides on the upstream slope after reservoir draw down.

SLIDES DURING CONSTRUCTION

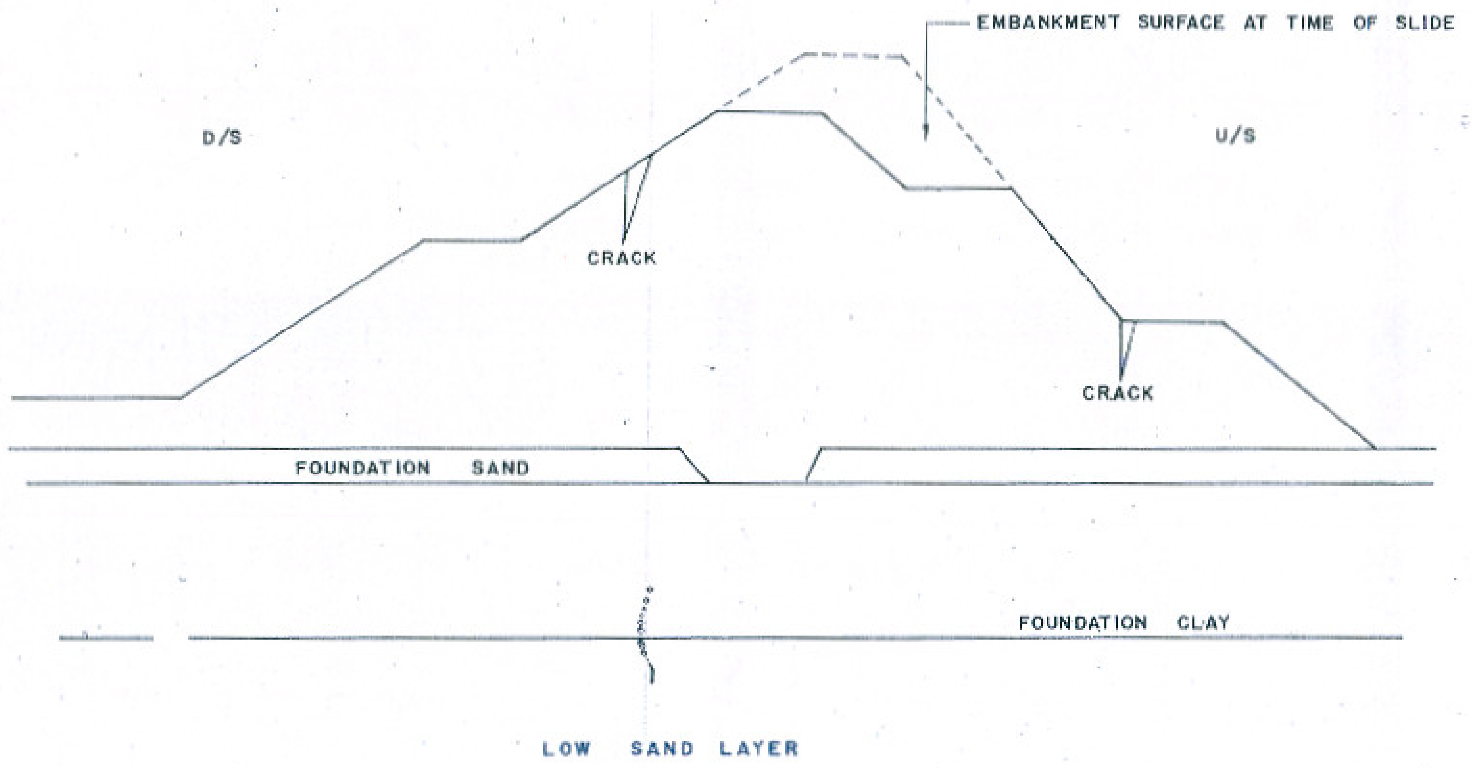
In every case of slide during construction, the dam was underlain by a foundation of either soft, brittle, or sensative clay, usually of high plasticity, and a large portion of the sliding surface passed through the foundation. It is known that there are two types of slides that occur during construction of dam.

In the first type (SLOW SLIDE), the movements starts gradually and continues at the uniform rate for a period of one or two weeks. The total horizontal and vertical component of the moement are usually a small percentage of the height of the dam (often between 5 and 15 percent), while the sliding never completely stops, the movement slows down at the end of the initial period to an un-important rate in a creep like action.

The second type of slide (Rapid slide) takes place suddenly, and the magnitude of the movement is usually equal to one half of the total height of the dam. The major part of sliding is over in a few minutes, and the movement stops.

The difference between the two types is due to the result of differences in the foundation clay. Slow slides occur when the foundation is more or less homogenous deposits of soft clay which is not sensitive; that is, which does not lose its strength under the shearing movement. Rapid slides results when the foundation clay contains horizontal bedding planes, layers of coarse silt or fine sand through which the high pore water pressures developing under the crest of the dam can be transmitted outward toward the more lightly loaded areas under the toe of the dam. The failure occurs abruptly. A typical slow slide during construction occurred at the North ridge dam in Canada in Sept. 1953, when the dam was nearly completed. The first indication of the slide was the appearance of cracks, a slight bulging of the fill, and an overthrust at the downstream toe. The widest cracks, which opened to the maximum of about six inches at the top, were 11 feet deep along the upstream slope and 22 feet deep along the down stream berms at the locations shown in the fig. 2. As soon as the slope movement was noticed, fill construction was immediately stopped and berms were added at both sides. The dam was successfully completed in 1956 to its full height with no further modifications.

FIG. 2



SLOW SLIDE AT NORTH RIDGE DAM, CANADA

TABLE - 1

Relation of occurrence of slide to age of dam (in % of total of dams on which slides occured).

No. of Years After Completion	Percentage of Slides Occurring
0–1	29
1–5	24
5–10	12
10–20	12
20–30	12
30–40	11
40–50	0
50–100	0

TABLE - 2

Chronological Distribution of Failures

Calender Year	% of Total No. of Slides
1850–1860	0
1860–1870	0
1870–1880	0
1880–1890	3
1890–1900	3
1900–1910	16
1910–1920	23
1920–1930	26
1930–1940	23
1940–1950	3
1950– –	3

MARSHALL CREEK DAM IN KANSAS:

The failure of this dam occurred in 1937 when the embankment was within the 10 ft. of the crest. The watchman, riding on the running board of one of the cars, noticed a large crack in the roadway near the centre of the dam. A hasty inspection revealed that the crack was increasing in width and a rumbling noise was being produced. The cause of the failure was silty clay soil on which the dam was resting, as it transmitted the pore pressures horizontally. The cost of putting every thing is perfect condition was several thousand dollars.

DOWN STREAM SLOPE SLIDES DURING OPERATION

Two types of downstream slides have occurred in Earth Dams, deep slides, which generally pass through the clay foundation, and shallow surface slides. Deep slides take place during full reservoir and frequently reduce the free board by extending further upstream the upstream edge of crest. The internal pore water pressures which cause the deep slides are result of seepage from the reservoir through or under the dam. Many dams have been saved from complete failures after the downstream slides only by around-the-clock emergency action. Deep downstream slides generally move at about same rate, or some what faster, than slow slides during construction. A typical down stream movement might be 3 or 4 ft./day in the first day or so and then approximately 1 ft./day for several weeks.

FRUIT GROWERS DAM:

At the fruit growers dam in western Coloraddo, which was constructed to a height of 32 ft. in 1910, a deep downstream slide occurred in 1937 when the reservoir was at the maximum elevation. The sliding soil buried the down stream end of the outlet conduit, making it impossible to lower the reservoir.

extends only to the upsteam edge of the crest. Some upsteam slides have cut as far as the middle of the crest, and a few have reached to the down stream edge of the crest.

In some dams constructed for hydroelectric power, where the reservoir remains eseentially full all the time, the riprap wave protection is placed only at the top of the dam in the range of elevation over which the reservoir will operate. If the upsteam slope is relatively steep, trouble some surface slides may develop and become progressive as the reservoir is being filled and water level is still below the bottom of the riprap.

1.3 FAILURE OF DAMS DUE TO PIPING

Before 1900 the percolation of water through and under earth embankments and the stability of earth dam slopes were problems about which very little was known. The continuing poor performance of earth dams eventually put pressure on the engineering profession to take up these matters in the twentieth century.

Piping results from water entering the compacted clay and carrying with it the small particles of material. Eventually in this way a free path through the dam is made, which gradually increases in size, until the callapse of dam occurs. The events which take place in the case of piping failure are shown in fig.4. The nature of breach so formed varies greatly, depending upon the size of the dam and the type of clay used in constructing the embankment. The origin of the failure is usually low down on the dam, where the hydrostatic pressures are high, although this effect is balanced by increased thickness of the dam. The major obstruction to the flowing water is the core of the dam commonly made from puddled clay. It is to be noticed that any defect during construction can produce a weak spot and hence the piping can proceed through the core as shown in Fig. 5.

In Great Britain the earth dam has enjoyed a better safety record than in most other countries. The fact remains, however, that a few failures have occurred, and one was the worst dam disasters ever experienced in Great Britain.The crucial year was 1864.The Dale Dyke dam was built in 1858 to supply water to sheffield. It stood to the greatest height of 95 feet and was 1250 ft. long. Its thickness at the crest was 12 feet and at the base in the centre of the valley it was 500 feet thick. The slope of two faces were 1 in 2½ and the core wall was made of puddled clay and was 4 feet thick at the top, 16 feet thick at the base of the embankment, and below this level extended 60 feet into the foundation. The upstream and downstream parts of the embankment were composed of a mixture of shale and rubble excavated from the bed of the reservoir. These materials were dumped loosely into place without proper compaction and the dam's water face was covered with the layer of rough masonary Blocks. The outlet consisted of 15 inches cast iron pipes through the base of the embankment. The dam had an overflow spillway 24 feet wide and 11 feet deep. In spring of 1864 the outlet valves in the Dale Dyke dam were closed in order to raise the water level in Bradfield reservoir. This was the first time it was filled right up, and it was to be the last. On the afternoon of 11 March, Mr. John Hunson, an engineer of the sheffield water works and the man incharge of the dam, went to inspect the full reservoir and found that all was well. In the evening, a crack was observed which gradually widened and at 11.30 PM the dam failed completely. Nearly a quarter of the embankment near the middle collapsed, and an estimated 200 million gallons of water poured out of the reservoir. The wave of water was released carrying 40,000 cubic feet of water per second and bore down on the town of Sheffield, seven miles away, at a speed of 18 miles per hour. Between the dam site and Olerton valley opens out, and this allowed the water to disperse sideways, thereby saving sheffield from the worse fate. Even so some parts of the city were flooded to a depth of 9 feet and large areas were left covered with thick layer of wood, mud, sand and stones, when the water subsided. The failure of Dale Dyke dam killed 250 people, destroyed 798 houses and seriously flooded more than 4,000 others. Dozens of mills, factories and workshops were totally or partially destroyed and hundred of horses, donkeys and other animals were drowned.

It was discovered later on that the structure was a poor piece of work, the core wall was too thin, the embankment was loosely built and was made of wrong material. It also seem likely that dam failure was initiated by faulty positioning and laying out of the outlet pipes. As the heavy bank settled the pipes were probably displaced, and water percolating through the pervious embankment began to wash away the puddled clay surrounding the pipes. This undermined the core wall at the point where the pipes passed through it, and very soon considerable amount (volume) of water was eroding away the interior of the dam and collapse was inevitable.

After that failure, in a official report a recommendation was made to the effect that all dams and reservoirs in Great Britain should, by act of parliment, be subject to frequent, sufficient and regular inspections. This suggestion was not however taken up, and it took the dolgarrog disaster of 1925 to bring about reservoirs act (Safety provision act) in 1930.

DOLGARROG DISASTER

In Great Britain the Aluminium Corporation built two dams. The first stage of Eigian dam was built on the slopes of Caruedd Llewelyn in 1908, and three years later the structure was raised to a maximum height of 35 ft. with crest length of 3253 feet. It impounded 160 million cubic feet of water to drive the pelton wheels of the Dolgarrog Power Station and Aluminium works. In 1924, inorder to provide more storage space, a second dam was built two and a half miles below the Eigian dam and about a mile to the west of the power station. The Coedty dam was an earth structure made from locally excavated material and equipped with a Central core wall of concrete. Its maximum height was 36 feet and it was 860 feet long, the reservoir had a capacity of 11 million cubic feet.

On the evening of 2 November 1925 the Eigian dam failed, apparently as a result of piping caused by bad construction. The concrete itself was of poor quality, and the foundations of the dam were very inferior. Through out its length the structure was founded on a deep layer of blue glacial clay, and in places the base of the dam penetrated only a few feet into the top soil and only inches into clay.

At the point where failrue began, the concrete base was only 2 feet below the top of the clay and water percolating under the dam is believed to have led to piping, perhaps encouraged by the presence of low-level out let pipe and the fact that during preceding summer, when the reservoir was emptied, the clay had partially dried out. Any way, after seventeen years of trouble free use a section of clay under the dam was washed away leaving a channel 70 feet wide and 10 feet deep, the dam itself remained intact. The breach was estimated to have been large enough to release 50 million cubic feet of water per hour. The flood swept down the steep valley and poured into the coedty reservoir. There is nothing to suggest that Coedty dam any thing but sound piece of work, but its spillway was designed to deal only with flood arising from rainfall and not with the whole contents of Eigian reservoir. The Coedty reservoir being relatively small, was rapidly filled, spillway overwhelmed and over topped. The down-stream half of the embankment was washed away, the core wall collapsed and a breach, 200 feet at the top and 60 feet at the base, released a second wave of water. Without doubt this flood was more serious because, whereas the Eigian reservoir emptied at a rate of about 50 million cubic feet per hour, the whole contents of Coedty reservoir, 11 million cubic feet, was released at once. The water rushed down the river bed past the power station and swamped the village of Dolgarrog one mile below in the coreway valley. A great deal of damage was done to the power station and in the village. Fortunately a large number of villagers were attending the weekly film show when the flood wave arrived; and as this was held in the assembly hall on high ground, they were unharmed. Otherwise the death toll might well have been more than sixteen. Hundred of houses were destroyed and thousand of people were left homeless.

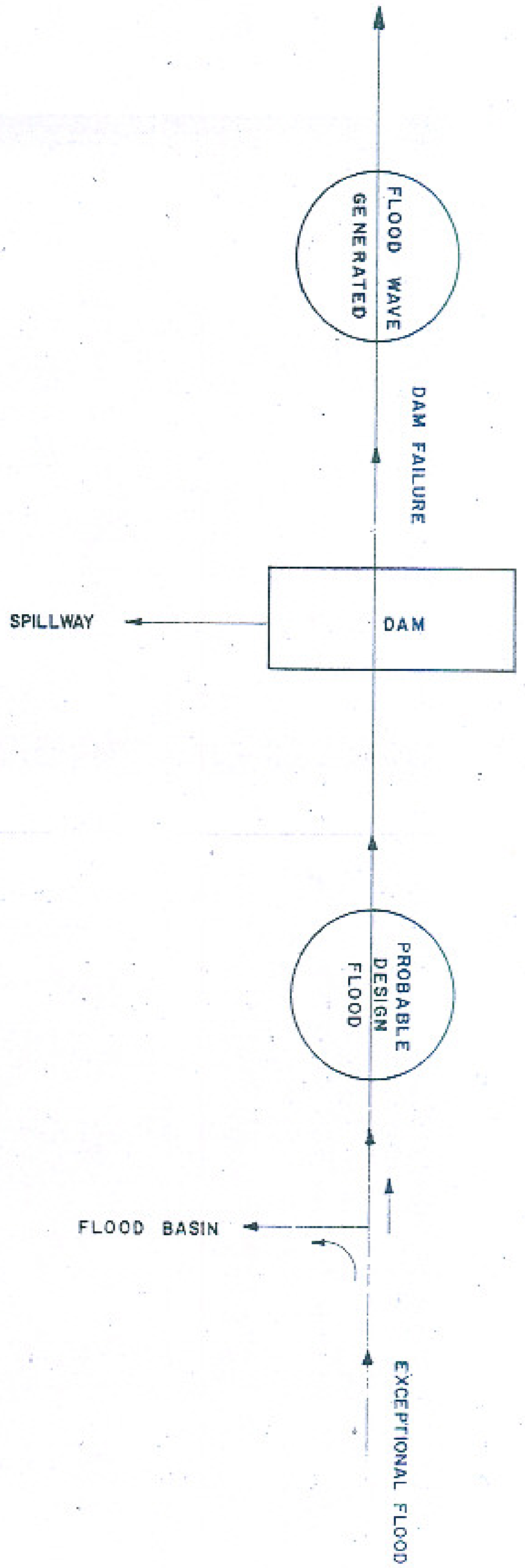
2. FLOOD ESTIMATION, AND DAM COLLAPSE WAVE

For designing a safe dam the engineer must know the maximum probable flood which can be expected in the life of dam and the discharge and route of flood wave in the case of the dam failure. Fig. 6 shows the whole thing in sketches.

2.1 FLOOD ESTIMATION

The occurrence of floods of exceptional magnitutde may give rise to serious disaster,

Fig. 6



and it is therefore important that the civil engineers responsible for designing works to deal with them should be able to make sufficient estimate of maximum flood intensity which might occur in any particular case. The subject of flood estimation occupies a prominent place in technical literature, both in regard to the records of actual floods and also to the methods of estimation.

Floods undoubtly follow certain natural laws, but the number and complexity of the factors involved and interplay between them make precise estimation impossible, and the subject of flood prediction has long been one of the perennial problems of civil engineering.

The natural laws governing floods are both recognisable and generally aprpeciated, the difficulty lies in their application. Where they are available, actual records of floods are naturally preferable to theoretical computations. The engineer is generally concerned with ascertaining either the worst flood conditions that may be anticipated from a given catchment, or the frequency with which what may be described as a normal flood is likely to occur.

Floods may occur yearly or even more frequently, but their intensity will vary and one of the exceptional severity may possibly be met with once only in hundred years. Complete security can only be otabined by ascertaining the maximum possible flood and making due provision for this. The ideal data would be the detailed records of all major floods from the catchment over the long period, this is seldom available. Except in certain special cases, the magnitude of a flood depends on the intensity and the distribution of the rainfall and on the characteristics of the catchment. In most countries rainfall data are more extensive than river flow data, and long period rainfall records will give a clue as to the frequency of the floods and as to the relation any recorded flood bears to the maximum flood probable. The method adopted for predicting the probable worst flood condition will necessarily depend on what data are available. Generally speaking, the various methods are as follows:

1) Where an extensive record of floods exists and also rainfall records, flood intensity and frequency may be determined from the study of these.
2) If no such records exist for the given catchment, but are available for the adjoining catchment, it may be possible to assess the flood condition by analogy.
3) If the records of several floods are avilable which can be correlated with rainfall, then a curve of flood intensity to the rainfall can be plotted and this can be extended to obtain an estimate of intensity for higher rainfall.
4) The records of several floods correlated with the flood level can be plotted as a rating diagram for the river. If the highest historical level of the river is known, then max flood can be deduced by extending this curve.
5) The cross-section of the river in the middle of the straight reach, with the average slope of the bed and the highest flood level, will enable an approximate estimate of the maximum flood to be made.
6) The maximum flood can be computed by means of formulas.

Of the above mentioned methods, number 1 and 2 are most reliable if the data are available, which is seldom the case. Method 3 is more likely to be possible, and gives fairly reliable results. Method 4 is depended on information as to the flood levels being available, high flood marks soon disappear and local reports of flood levels cannot always be relied upon. This method will be unreliable in an unstable river whose channel is subjected to scour and silting. Method 5 admits considerable room for error. The surface slope of river in flood is required rather than the bed slope, and if this can be determined during the period of flood or by the examination of the high flood marks immediately after, a more reliable estimate will be obtained. The last method, the use of formulas, to be of practical value, demands that the formula used should provide not only for the intensity of the rainfall but also for the characteristics of the catchment. This involves the use of co-efficients which must be determined by analogy and experience.

If must be noted that the accurate measurement of a flood itself presents considerable difficulties. Apart from these, complications arise if the bed is unstable, it may scour out at the height of the flood and silt again as the flood falls. In such a case the cross section of the river taken before or after the flood will not be the true section applicable to the flood at its height, and its use may lead to serious under estimate of the maximum flood intensity. Hence, unstable rivers present peculiar difficulties as regards the flood measurement, in that the relation between flood intensity and height of water becomes an in constant one if the bed is shifting.

Floods are primarily caused by the incidence of heavy rainfall on the catchment, a special case occurs where the rainfall has already accumulated in the form of snow, the melting of which gives rise to or increase the flood. The rainfall may vary in intensity over the catchment, some parts of which may receive no rainfall at all. It may also vary during the period of the storm. If the rainfall is continuous, uniform and evenly distributed over the whole catchment, then the point of concentration (or outlet of catchment) will receive an even increasing amount of water, as the more distant parts contribute their quota, until a maximum is reached when the water from the furthermost part is arriving and the whole catchment is thus contributing to flood. It allows, therefore, that if the rainfall is uniform and evenly distributed over the catchment and the duration of storm is sufficient, the flood intensity and discharge will increase with the area of catchment and the intensity of rainfall. But if the duration of the storm is in-sufficient to enable the whole catchment to contribute at the same time, then while flood discharge or runoff will increase with the area of catchment, flood intesity may not necessarily do so. The time factor thus becomes very imortant, particularly in regard to the relation between the duration of storm and the period of concentration, that is to say, the time taken by the water from furthest point to reach point of concentration.

It has been well established that greater the area of the rainfall, the less will be its average compared to its maximum intesity. It is further well established that greater the duraation of storm, the less will be the average intensity of rainfall. The average intensity is thus some inverse function of the area and the duration of storm. Of the actual rainfalling on the catchment, part will be lost by absorption and other causes, and some fraction only will reach the point of concentration. This fraction, called the run off factor or co-efficient, is dependent on the number of conditions, such as the nature of the soil and the vegetation.

The steeper the slope of the catchment, the more quickly the water will flow off it and the shorter will be the period of concentration.

If the catchment is long and narrow, the distance from the point of concentration to the further most point will be greater than in the case of shorter and wider catchment of the same area and consequently its period of concentration will be greater. Finally, if the catchment is already wet when the storm occurs, the water will flow more rapidly to the point of concentration.

There are thus a number of factors which tend to affect the flood and which group themselves under the two main categories of rainfall and characteristics of the catchment. These factors are as follows:

A. i. Rainfall Intensity
   ii. Rainfall distribution
   iii. Rainfall duration

B. Characteristics of catchment area.
   iv. Characteristics of the catchment shape.
   v. Characteristics of the catchment slope.
   vi. Characteristics of the catchment nature as affecting the run-off factor.
   vii. Characteristics of the catchment initial state of wetness.

These seven factors may be described as the major factors affecting floods. Some of them are inter connected and are not constant either in time or space. Two further factors may be mentioned: temperature, the effect of which will be mainly reflected in the run-off factor, and wind, which will affect the distribution of rainfall.

It is clear that in the factor 'A' mentioned above, rainfall, the possible variations of intensity, distribution and duration are unlimited. What one is primarily concerned with is to ascertain what conditions within the limits of those probable in the particular district are likely to produce the worst flood. The factors area and the slope of the catchment are constant. The slope of the catchment may vary. The run-off factor will depend on the nature of the catchment, particularly in regard to soil and vegetation and may not be uniform through out the catchment.

The last factor is the initial state of wetness, and is important not to underestimate flood possibilities, and it is better therefore to assume that the catchment is initially wet when the storm occurs, an exception might be made in normally arid region subject to occasional storms. If there are lakes or marshes within the catchment area, or low lying ground which may become in-unduated, then the part of the flood water will be temporarily held up and flood peak will be damped down. It may be possible to take this into account in assessing the run-off factor, but if such areas are wide spread, it is probably better to estimate their damping effect seperately.

FLOOD FORMULAE

Many formulae for estimating flood intensity have from time to time been published; some are based on theoretical considerations, other are purely emperical and derived from records of actual floods, some again are based on combination of both these methods.

DICKEN'S FORMULA

This is an emperical formula based on the records of a few floods and expresses the flood intensity as a simple function of catchment area. $Q = 825\, a^{3/4}$ where Q denotes the maximum intensity of the flood in cusecs (ft.3/sec) and a denotes the area of catchment in square miles. This formula, partly no doubt on account of its extreme simplicity, has been widely used and probably to a much greater extent than contemplated by its author, who derived it for Northern India and considered it as approximately correct for districts having an annual rainfall of from 24 to 50 inches.

As it takes only one variable factor, area, into account, the remainder must be considered as uniform, and the formula is therefore only applicable to a services of catchments of uniform characteristics other than area. A variation of any of the other factors would affect the coefficient. Various values for the coefficient in Dicken's formula have been adopted with the view of making the formula applicable to other parts of world. Beale used 1600 when the rainfall is very large.

RYVE'S FORMULA

$$Q = C.a\ 2/3$$

C = Coefficient. He adopted a variable value of coefficient C as follows.

C = 450 for areas within 15 miles of coast
= 563 for areas between 15 and 100 miles from coast.
= 673 for limited areas near hills.

there is thus only one variable factor area of catchment. The other factors are assumed to be uniform and are covered by a single coefficient, for which, three alternative values are given according to the location of the catchment.

INGLIS FORMULA

Sir Claude Inglis prepared a formul $Q = \frac{7000A}{A+4}$ where Q denotes flood intensity in cusecs and 'A' the catchment area in square miles. He derived it for India and considered that it covered almost all catastrophic floods recorded in that country. His view is that the area is the preclaminent factor. The formula is applicable to catchments whose main characteristics except area are similar.

CHAMIER'S FORMULA

This formula published in the paper presented to the I.C.E. vol 134 is as follows $Q = 640\ iKa^{3/4}$ where Q is max flood intensity in cusecs.

i. - Average rate of rainfall anticipated in inches per hour for such duration as will allow of the flood water flowing to the outlet from the furthest point of the catchment.
k - the coefficient of run off.
a - area of catchment in square miles.

The difficulty is the application of this formula and lies in the assessment of i. This is dependent on the period of concentration which is an unknown quantity. Chemier detemined this period by assuming velocitics for stream flow and surface drainage.

UNIT HYDROGRAPH METHOD

This method of flood prediction, developed in America, is one based on the analysis of river flow records which must be extensive for its application. The application of the method is as follows.

i. From a study of the recorded hydrographs of stream flow and the corresponding rainfall records, a hydrograph is prepared showing the run off from the catchment given by a storm of one day's duration (or other unit of time selected). The base or ground water is deducted, giving a hydrograph of a surface run-off due to the storm (hydrograph seperation techniques are used).
ii. Expressing the daily run-off as the percentage of the total run-off of the storm, a distribution graph is prepared which shows the daily percentage distribution of the runoff of a storm. A number of these distribut-

tion graphs are drawn and the average distribution graph for the catchment is obtained.

iii. The total rainfall of a one day storm (or other unit of time taken) is then distributed over the several days run off according to this diagram and the figure obtained represents the hydrograph of the one day storm with 100% run off factor. This diagram is then adjusted to suit the observed factor of run-off and gives the hydrograph of net run-off for a one day storm. This hydrograph can be drawn in terms of cusecs or of inches of run-off from the catchment area.

iv. If the storm has a duration of several days, then each day is treated separately and the one day hydrographs are super imposed in their correct phase and combined to give a hydrograph of run off for the whole storm. Similarly a combined hydrograph can be obtained for consecutive storms. This method enables the hydrograph to be drawn for a given storm or a hypothetical storm, and to obtain the maximum flood of which a catchment is capable.

2.2 DAM COLLAPSE WAVE

Despite of substantial safety factors incorporated in the design of important dams and careful prediction of maximum expected flood, the probability of their failure, sudden or gradual, cannot be ignored. A prior knowledge of flood resulting from dam failure can be used as the basis for rational zoning in the valley down stream of the dam, as well as in the preparation of preparedness plans, well ahead of any possible catastraophe.

The wave, generated due to failure of the dam, may have either gradual variations in flow rate and depth of water (over major portion of the wave) if the breach is gradual or rapid variations if the breach is sudden. In the former case, flow is termed as gradually varied unsteady flow. The general equations governing unsteady flows derived from the principle of conservation of mass and momentum are called saint venant equations and are as follows:

$$\frac{y}{x} + v/g\,\frac{y}{x} + \frac{1}{lg}\,\frac{v}{t} = SO - S_f \quad (1)$$

Momemtum equation

$$D\,\frac{y}{x} + V\,\frac{v}{x} + \frac{v}{t} = 0 \quad (2)$$

-Continuity equation -

$$dy = \frac{y}{x}\,dx + \frac{y}{t}\,dt \quad (3)$$

- (Total change in depth)

$$dv = \frac{v}{x} + \frac{v}{t} \quad (4)$$

(Total change in velocity)-

where

$\frac{y}{x}$ = slope of watersurface

$\frac{y}{t}$ = Change of depth with respect to time.

$\frac{v}{x}$ = Change of velocity with respect to distance

$\frac{v}{t}$ = Change of velocity with respect to time.

S_o = Channel slope

S_f = friction slope $= \frac{n^2}{2.21R\ 3/2}\ {-4/3}$

n = Manning's roughness Co-efficient

R = hydraulic radius = area / wetted perimeter

D = hydraulic width $= \frac{\text{area}}{\text{width}}$

dy = total change in depth

dv = total change in velocity

g = accleration due to gravity.

The above equations are manipulated to yield dimensional or dimensionless equations of characteristics, which can be solved graphically or by a programme simulated on a computer. The sudden total callapse of a dam which is a worst case, holding back a reservoir of water leads to the following sequence of events.

i. Water discharges from reservoir under negative surface wave at rate dependent on intitial depth of water behind the dam.
ii. The flow at dam site is near critical.
iii. The surge of water released by reservoir progresses down the valley at the speed dependent both on intitial depth of water behind the reservoir and surface roughness of valley.
iv. The water velocities are super critical in early stages. Resistance effects may eventually cause passage to sub-critical conditions.

Normally due to sudden collapse of a dam, with dry bed conditions downstream, the shape of the wave profile is as on fig. 7. Barre' de Saint Venant developed the following relationship for the profile of a wave resulting from instant failure of dam, with dry downstream channel bed, zero slope of channel and no frictional effects which represents a parabula with

$$\frac{X}{t\sqrt{gy_2}} = 2 - 3\,(y/y_2)^{1/2} \qquad (5)$$

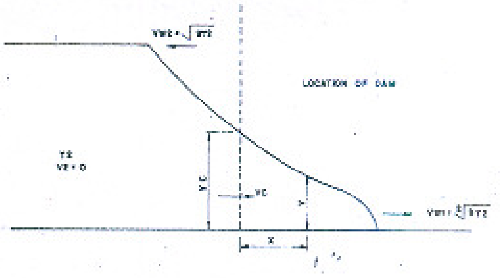
with vertical axis and vertex at the bed, x being the distance from the dam. Considering continity, it was shown that

$$V_c = 2/3\,\sqrt{gy_2} \qquad (6)$$

$$y_c = \frac{4y_2}{9} \qquad (7)$$

$$q = \frac{8}{27\sqrt{g}\,y_2^{3/2}} \qquad (8)$$

FIG. 7



WAVE GENERATED ON SUDDEN DAM FAILURE

where q is dischange per foot width.

The theoretical results presented by Saint Venant, were verified experimentally by A. Schoklitsch and later by U.S. Corps of Engineer (1960-61). The latter also considered the base flow prior to the failure of dam. The experimental data permitted the computation of maximum discharge due to partial breaches of width and depth. The equations presented by A. Schoklitsch are:–

i. Partial width and full depth.

$$Q_{max} = \frac{8}{27} Wb \left(\frac{Wo}{Wb}\right)^{1/4} \sqrt{g}\, yd^{3/2} \qquad (9)$$

ii. Partial depth and full width

$$Q_{max} = \frac{8}{27} Wo \left(\frac{Yo}{yd}\right) 0.33 \sqrt{g}\, yd^{3/2} \qquad (10)$$

iii. Partial width and partial depth

$$Q_{max} = \frac{8}{27} Wb \left(\frac{Wo}{Wb}\right)^{1/4} \left(\frac{yo}{yd}\right) 0.33 \sqrt{g}\, yd^{3/2} \qquad (11)$$

Where

Q_{max} = maximum discharge in cusecs.

Y_o = depth of water in reservoir before breach in ft.

Y_d = distance from original water surface to bottom of breach in ft.

W_b = width of breach at original water surface in ft.

Wo = width of dam at initial surface level in ft.

g = Acceleration due to gravity in ft./sec.

The experiment results by U.S. Corps of Engineer (1960–61) showed that equation 11 is valid for

$$1.0 \leq \left(\frac{Wo}{Wb} - \frac{Yo}{Yd}\right) \leq 20$$

A limiting value of downstream depth (Y max) was also shown to be

$$Y\,max \angle Yd' \frac{Wb}{Wc}$$

where Yd' = maximum depth passing through the breach during initial time intervals.

W_c = width of downstream channel.

In the absence of any evidence, it is a matter of conjecture to what extent these relationships hold for actual field conditions.

G.H. Keulogan (Engineering hydraulics, edited by H. Rouse) has indicated that for a rough approximation, Constant rate q at a given section of originally dry channel will produce a surge front of height "Y" which travels approximately with velocity.

$$V_w = 1.5\,(gq)^{1/3} = 2\sqrt{gy}$$

The computations of downstream stages (to get dam failure hydrograph) require a precise knowledge of topographical characteristics of the valley. This may consist of longitudinal profiles and cross-sections at general and typical locations, such as bridges, contractions, etc.

The whole procedure for computing down stream stages is as follows.

We know the distance of the cross-section of the valley where we are interested in getting the height of wave.

1 Elevation	2 Area A	3 Hyo-Radius	4 AR $^{2/3}$
x		–	
x		–	
x		–	
x		–	
x		–	
x		–	
x		–	

Column 1 – Assume different elevations at the cross-section under consideration.

Column 2 – Area of cross-section with assumed elevations of the column 1.

Column 3 – hydraulic radius R = A/P, where P is wetted perameter.

We can get Q_{max} from equation 9, 10, or 11, as appropriate.

The slop of bed channel S, known and also manning's roughness co-efficient n is known calculate AR 2/3 = $\frac{Q_{max}\ n}{1.49\sqrt{S}}$

Compare this calculated value with the values of column 4 and get appropriate value of elevation from the column 1 corresponding to above calculated value. Hence wave height can be obtained

$$Y_{max} \leq \frac{Yd_{max} \quad W_b}{W_c}$$

3. CONCLUSION

It has to be recorded that disasters have overtaken dams which were built by those who were in their day acknowledge master of their Profession. Inspite of the inherent soundness of original design it is not always possible to recognize potential weaknesses and once such weakness has been discovered it may be too late to remedy.

It has been pointed out that something like one quarter of recorded dam failures has been attributed to in-sufficient spillway capacity. In America a task committee was set up to look into the adequacy of existing dam spillways, because usually it is the inadequate capacity of the spillway which brings the failure of dam. What task committee discovered was as follows:-

i) Dam designers do not carry out analysis of risks associated with different designs.
ii) They do not consider flood reporting network and forecasting system.
iii) They do not consider any form of warning and evacuation system for possible disasters, which is a great risk.

We will have to accept the fact that dams, like any other structure have a definite probability of failure but it is to be considered in mind while designing a dam that dam failures cannot be tolerated. It is recommended that engineer must face upto estimating degrees and consequences of failure, due to different flood sizes. The probable maximum flood to be "considered" to be practical upper limits for analysis, with an arbitrary assigned average return interval of 1,000 years.

Recent advances in methods of estimating design floods have been accompanied by changes in the method of providing their safe passage; Where conditions are suitable the total design flood can be divided into two parts. A normal type of spillway can be used to deal with the floods of order of those recorded in the past, while other type of tunnel or pipes can be provided to act as 'fuse plugs' to provide additional relief in the event of catastrophic flood.

In Soviet Union the method adopted is to put tiles on the embankment and allow overtopping.The deficiencies in the science of dam design and construction led to the Foundation of the International Commission on large dams in 1948. The important remarks made by speakers in one of the meetings are as follows:

1) The evaluation of dam design should be directed towards making more effective use of materials of construction.
2) An Italian speaker suggested that deep study of problem would assist in reducing the areas of ignorance which surrounded them and so lead to further economics of design.
3) Another speaker suggested that for any given site the type of design adopted should represent the best which could be chosen in the light of engineering and scientific knowledge available. He also drew attention to the fact that all dams are subject to gradual deterioration, and that because of lack of homegeneity in the foundations the factor of safety might in fact be lower than the value assumed in design. Apart from this International Commission many Commitees were set up in England and America for the safe construction and inspection of dams, for example:

i) Dam safety legislation
ii) National Dam Safety Programme
iii) National Dam Investigation Programme
iv) Re-evaluating existing dams

In 1877 Swiss federal parliment introduced legislation which brought the control of water courses under federal supervision. The act was revised as result of the destruction of Mohne dam, and it was again amended in 1953, to provide compensation for those who might suffer from the destruction of dam, whether as the result of natural cause or enemy action. A further amendment in 1957 brought within the scope of federal control all dams more than 33 ft. in height, and all reservoirs with volume in excess of 39 acre ft. the Act in its present form provides that both design and construction should be properly supervised and stipulates the precautions which shall be taken in the event of dangerous situation arising. Todate, however, Swiss Regulations make no provision for the periodic inspections of the work once it has been completed.

In 1930, British Parliment passed the reservoirs (safety provision) act. The act provides that the owner of the reservoir containing more than 5 Million gallons shall obtain at interval of not more than ten years a certificate from duly authorised engineer as the evidence of the safety of reservoir.

In 1933, Germans prepared a set of draft regulations covering the design, construction and operation of dams. In 1953, these draft regulations were superseaded by DIN 19700. While this code possesses no authority, it has never the less proved of value as a means of reminding the owners of dams of their duty to the public. The code was drafted by a team of 69 specialists from the field of dam construction and design. Regulation 5.2 of Code requires that in the event of danger the custodian of the dam shall not only inform the owners immediately but shall issue a warning to the inhabitants of the area which might be effected.

The federal government of USA exercises no authority over the design and construction of dam. Instead the responsibility lies or is delegated, as may be appropriate, to the U.S. Bureau of Reclamaion, Tannessee Valley authority and U.S. Corps of engineers, or if the dam serves a farming community the resposibility lies with the department of agriculture. On comparing the various sets of regulation it is found that considerable diversity exists between different authorities. For example, those issued by the Corps of engineers specify higher factor of safety then those of the Bureau of Reclamation, possibly implying the recognition of greater latitude in the quality maintenance to be expected.

If a company or a public authority in U.S.A. wishes to build a dam then the design have to be submitted for approval to one of the authorities mentioned above. The Water Code on the supervision of dams issued in 1960 by the department of water resources in U.S.A. States that "if is found that unsafe conditions exist, the department shall take action as it is necessary to render or cause the condition to be rendered safe".

It is further concluded that our present knowledge of the forces which lead to the deterioration of dam is incomplete and much help can be obtained in filling the gaps in our knowledge by making a close study of disasters when they occur.

Every dam which impounds water presents a potential danger which should neither be under nor over estimated.

Man's dependence on dams as the means of harnessing and controlling rivers is not likely to diminish in the near future. The benefits to be derived, especially in under developed countries, are so obvious and worthwhile that the various political, financial and social problems will be over come if humanly possible, and the strategic risks will either be lived with or ignored.

There will be technical problems in building safe modern dams, but solved they will be, that is in the very nature of engineering.

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RURAL WATER SUPPLY IN DEVELOPING COUNTRIES

By
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INTRODUCTION

It is difficult to imagine a clean and sanitary environment without water. Invariably, the progress of sanitation throughout the world has been closely associated with the availability of water; and the larger the quantity and the better the quality of water, the more rapid and extensive has been the advance of public health.

The realization of importance of water is a recent development. This knowledge, even today, is not complete, particularly with regard to the relationship which apparently exists between the quantity of water available per person and the incidence of certain communicable diseases.

A supply of clean potable water to the rural areas is one of the factors needed to improve conditions in the rural areas, thus reducing the social irregularities existing between rural and uraban living conditions.

According to an estimate, over 1,000 million people living in rural areas do not have an adequate water supply and the majority of these people live in Asia. The rate of extending water supplies to these areas is slow, and does not keep pace with population growth. The water-borne diseases, in these areas, are among the major causes of sickness and death.

Governments of developing countries are becoming increasingly concerned with improving the life of the people in rural areas. Many health experts and, in particular, the World Health Organization beleive that the provision of a safe and convenient water supply is the single most important and cost-effective activity that could be undertaken to improve the health of rural populations.

THE SITUATION IN 1970 - TARGETS

According to a survey carried out by WHO in Dec. 1970, only 14 percent of the rural population in developing countries had reasonable access to safe water. The survey obtained information on the water supply situation in 91 developing countries; the population of these countries was 1700 million out of which 73 percent lived in rural areas (the population figures are indicated in Appendix I). The situation in urban areas appeared far better; about 68 per cent of the inhabitants had access to a piped water supply (a summary of the percentage of population with reasonable access to safe water is shown in Appendix II).

Expressed in figures, 144 million people had no service in urban areas, and in rural

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areas 1076 million people - about one-third of the total world population, were without reasonable access to safe water.

To improve the situation, the United Nations set goals for the global improvement of water supply during 1971–80. These goals were to supply safe water to 100 percent urban population (60 percent through house connections and 40 percent through public hydrants) and to 25 percent of the rural population in 1980.

To achieve the said goals, the estimates of the investment needed were:

\$ 11,000	million for urban water development.
\$ 3,000	million for rural water development

The estimated per capita costs for providing extension water service of these areas were:

\$ 16 – \$ 120	for urban supplies through house connections.
\$ 9 – \$ 28	for urban public hydrant supplies
\$ 6 – \$ 24	for rural supplies.

GENERAL PROBLEMS

The most common problems encountered in rural development programs are:

1. Institutional

- Lack of rural water supply policy forming part of a national water supply policy.
- Existence of several Government agencies whose responsibility overlap.
- Lack of water organizations at the local level.
- Lack of trained manpower at every level.

2. Financial

- Per capita cost increases as village size decrease.
- Relatively low income of rural inhabitants and limited financial resources.
- Lack of village motivation, lack of public health education. The rural inhabitants are unaware of the potential benefits of improved water systems and are not willing to pay for them.
- Lack of a policy to obtain maximum support i.e. financial support from areas to be served.
- Seasonal availability of water from ponds, streams, shallow wells or other sources of questionable quality. The villagers may return to these reasources if high charges for piped water are imposed.

3. Technological

- Short operating life for equipment, poor maintenance and many project failures.
- Problems of communication between remote rural systems and their support organisations in areas with poor telephone service so that system breakdowns are not reported promptly.
- Difficulty in obtaining spares due to lack of money, scarcity of foreign exchange, combersome procurement procedures.
- Difficulty in providing sufficient repair staff and transport to attend promptly to breakdowns in widely spread rural systems with poor road links.

Out of the above,, the institutional and financial problems are the most crucial.

TECHNICAL ASPECTS

Factors that affect the type of rural water supply system include, the level of service, water quality and quantity and the nature and location of water sources. The general principles that may be applied to most rural water program s are as:

- Ground water from springs, wells and boreholes, which require little or no treatment to make it safe is preferable to surface water.
- Systems must be designed for simple, trouble-free operation, and be capable of being operated and maintained by local technicians.
- Replacement parts of the equipment must be readily available.
- The equipment must be able to withstand hard usage.
- Standard designs which can be slightly modified to meet local conditions should be developed and used for cost estimation, procurement and construction.

Level of Service

In rural water supply systems, various levels of service can be provided. They are:

- No distribution system. One or more water points such as a protected spring or a well with a pump.
- A simple distribution system. Few public hydrants supplied with water from a single source.
- More elaborate distribution system. Serves a substantial number of public hydrants and some house connections.
- Systems with a substantial number of house connections and few public hydrants.

The capital costs as well as the associated operating costs of the facilities will rise with the level of service. The first two levels are likely to be very simple systems, using a hand-pump or gravity supply, whereas the latter ttwo which require large quantities of water will require motorized pumps and some treated water storage facilities to meet peak demands and guard against breakdown.

Quality

Quality standards for rural water supply are principally concerned with ensuring that water does not contain any matter either chemical or biological, which would affect its safety or acceptability. Standards which have little bearing on health, such as hardness of the water, or the presence of iron, manganese or chlorides, can be relaxed unless this could cause technical problems such as encrustation or corrosion, and so long as the rural inhabitants find the water acceptable. The International Standards for drinking water as suggested are reproduced below. It may be pointed out that due to wide variations in the chemical composition of water in different parts of the world, rigid standards for chemical quality cannot be established. The following limiting concentrations are indicative only and can be disregarded in specific instances:

Substance	Permissible	Excessive
Total solids	500 mg/l	1500 mg/l
Color	5 units	50 units
Turbidity	5 units	25 units
Taste	Unobjectionable.	–
Odour	Unobjectionable.	–
Iron (Fe)	0.3 mg/l	1.0 mg/1
Managnese (Mn)	0.1 mg/l	0.5 mg/l
Copper (Cu)	1.0 mg/l	1.5 mg/1
Zinc (Zn)	5.0 mg/	15 mg/l

Calcium (Ca)	75 mg/l	200 mg/l
Magnesium (Mg)	50 mg/l	150 mg/l
SO_4	200 mg/l	400 mg/l
Chloride (C1)	200 mg/l	600 mg/l
pH range	7.0 – 8.5	Less than 6.5 or greater than 9.2
Magnesium + Sodium sulphate	500 mg/1	1000 mg/1
Phenolic substances (as phenol)	.001 mg/1	.002 mg/1

There are certain substances which, if present in supplies of drinking water at concentrations above certain levels, may give rise to actual danger of death. A list of such substances and of the levels of concentrations which should not be included in communal drinking water supplies is given below:

Substance	Max. allowable concentration
Lead (as Pb)	0.1 mg/l
Salenium (as Se)	0.05 mg/l
Arsenic (as As)	0.2 mg/l
Chromium (as Cr)	0.05 mg/l
Cyanide (as CN)	0.01 mg/l

Quantity

The quantity of water consumed depends uponseveral factors of which the most important is convenience. If there is a supply in the house, the consumption may be 5 or more times greater than if water has to be fatched from a public water point. The climate and cultural patterns of bathing, laundering and food preparation are also important factors. Public bathing and laundering facilities, if provided, may increase demand considerably. Waste water may be a major problem unless public hydrants are properly designed to prevent faucets from being left to run continuously or measures are taken to control and supervise their use.

The WHO surveys give the following data for average daily consumption in rural area.

Region	Liters per capita per day (lcd) Minimum	Maximum
Africa	15	35
Southeast Asia	30	70
Western Pacific	30	95
Eastern Mediterranean	40	85
Europe (Algeria, Morocco and Turkey)	20	65
Latin America and the Caribbean	70	190
Average	35	90

STANDARD DESIGNS

To reduce both the engineering costs and project preparation time sets of standard designs should be developed with corresponding standard costs. Sufficient attention must be given to the cost effectiveness of the designs and materials selected.

To the greater extent possible, the standard designs should use local materials and technology and be suitable for construction with unskilled village labour.

No unnecessary expensive decorative work, which would not contribute to the efficiency of the plant should be included. Block work and masonry can be substituted for concrete, locally fabricated asbestos cement or polyvinyle chloride pipes for imported cast iron or steel.

The technology involved should be kept as simple as possible, so that local operators will be able to operate and maintain the system for long period of time without the assistance of a trained operator from a central agency or of a qualified engineer.

Treatment plant should be simple and automatic equipment should be generally avoided in favour of manually operated equipment as the maintenance and operating of spares of the former may be difficult.

PER CAPITA COSTS

The per capita costs of rural water systems may vary widely from country to country and from one area to another, depending on local conditions and the type of system installed. The following are the important features:

- Where groundwater is available at moderate depth, constructing a number of wells fitted with handpumps is the cheapest means of providing a good water supply.
- Use of surface water which requires full treatment may be several times as expensive as using ground water.
- Providing a high level of house connections may at least double the per capita cost of the system.
- Distribution system costs are a high proportion of total system costs. For systems serving poor regions, considerable saving may be made by omitting the distribution network and delivering the water through overhead storage tanks to a few central public points.
- There are considerable economies of scale in piped village water system. For similar systems, per capita costs for a project in Tanzania fell from $ 27 to $ 16 as the population served increased from 1750 to 5000. In general one can say that for similar systems the per capita cost of a system for a village of 10,000 population may be only 40 percent of that for a village of 1,000 population.

BENEFITS

Ideally, decision to invest in rural water supply should be based on cost-benefit analyses in which both costs and benefits are quantified. However, no satisfactory method has yet been developed for quantifying all the benefits of improved water supply. Nevertheless, the strongly held opinion of Public Health officials, and in particular of WHO is that provision of safe water is of prime importance to public health and in combination with other sanitary measures, is an essential prerequisite to control waterborne diseases.

In some cases the benfits may be directly measureable and quantifiable: for example, an improvement of the water supply may allow processing of produce, fish freezing or yarn dyeing. But in most cases the benfits cannot be measured adequately; in this respect the investment in rural water supply resembles investment in many other social sectors such as education. The most important of the unquantifiable benefits are improved public health and greater convenience both of which may increase productivity. Indirect benefits commonly cited are a slowing down of rural-urban migration, redistribution of real income in favour of the rural poor, a better standard of living and the development of village institutions.

Public Health Benefits

Numerous studies have identified contaminated water as the principal agent in the transmission of typhoid, cholera and bacillary dysentry. Lack of safe water for drinking is also an important factor in the spread of other diarrhoeal diseases the most common cause of death in infants in the developing countries. To break the chain of transmission of certain diseases, improved excreta disposal methods must be provided together with improved water supply. The combination of these two measures will frequently be found to be the most effective means of control. Public health education will almost always be necessary to achieve full length benefits.

Productivity Benefits

Improving rural water supply may be an essential step in the development of rural industries such as fish processing and freezing, fruit and vegetable production, or cloth dyeing. The benefits from these activities can be measured directly. The productivity of the inhabitants can be increased in two ways: reduction in the time and effort spent fetching water and increasing output through improved health.

Slowing of Migration

Most developing countries are experiencing a high rate of migration from rural to urban areas which strains their social and economic infrastructure. At an individual level, a better water supply reduces the "push" component of migration from villages to the towns, on the other hand, it does nothing to reduce the "pull' component (better jobs, higher incomes, greater educational opportunities). For the real control of migration, rural water supply should be coupled with rural development (other programes) to encourage people to remain in their areas.

Income Redistribution Effects

Income redistribution from more prosperous urban areas to less prosperous rural areas is a common feature of rural water supply projects, since most rural projects are not financially viable and need support whether from Central Government revenues or from a national water authority. Care should be taken that richer farmers do not benefit at the expense of the urban poor.

Improvement in Village Institutions

Many rural areas in developing countries lack an organization of community leaders capable of dealing with present day problems. The Community Water Supply Project is one way of encouraging the emergence of such leadership.

Also as the rural inhabitants are required to pay for a valued service such as water supply, it will develop a "habit of payment" for other worthwhile goods, and that this willingness to pay will indicate to the planners that the village should be selected for further development.

Lower Per Capita Costs

Village water systems serve more people for a given investment than urban systems, because systems in rural areas normally provide a lower standard of service than those in urban area.

Fire Protection

In rural areas fires are fought using water brought in containers from neighbouring public points. If water is readily available, many fires can be extinguished before they cause much damage.

The benefits of improved fire control depend on a number of factors, including the material used in house construction, housing density and distance to the pump or public hydrants. These benefits are difficult to quantify. In economic terms, these benefits may not be large but the increased security from fire is an important factor in villager's desire for an improved water supply.

ORGANIZATION AND A MANAGEMENT

Institutional weakness is probably the most important single problem in rural water supply. Many countries do not have a national policy for rural water supply. Numerous agencies are normally involved in the management of rural water schemes. This leads to uncoordinated or inefficient planning and extension of projects.

Most agencies suffer from inadequate staffing usually because the conditions of service are unattractive compared with those in the private sector or in agencies working in metropolitan areas. To attract better staff in sufficient numbers, salaries and other benefits should be improved where possible. In addition, training is required at all levels from operators and technicians to professional staff.

APPENDIX I

POPULAION OF COUNTRIES SURVEYED IN 1970

(MILLIONS)

Region	Urban	Population Rural	Total	Percentage rural population
Africa	31	152	183	83
Latin America and the Caribbean	156	118	274	43
Eastern Mediterranian	65	169	234	72
Europe (Algeria, Morocco and Turkey)	24	42	66	64
Southeast Asia	158	693	851	81
Western Pacific	38	75	113	66
Total	472	1249	1721	73

APPENDIX II

PERCENTAGE OF POPULATION WITH REASONABLE ACCESS TO SAFE WATER

Region	House connections	Urban Public Hydrants	Total	Rural	Total
Africa	29	39	68	11	21
Latin America and the Caribbean	59	17	76	24	54
Eastern Mediterranean	58	26	84	18	33
Europe (Algeria, Morocco and Turkey)	50	23	73	44	55
Southeast Asia	36	17	53	9	17
Western Pacific	65	10	75	21	40
Weighted Average	49	19	68	14	29

PREVENTIVE MAINTENANCE AND REPLACMENT STRATEGIES — A SURVEY

By

DR. AMJAD PARVEZ SHEIKH*

1.0 INTRODUCTION

The renewal theory analysis is particularly useful in evaluating the performance of maintenance and replacement strategies, and in predicting their performance. When items fail due to sudden stochastic failure, they need to be replaced (32). If the time to failure can be represented by a random variable and each replacement is made with an item of a similar type, then the replacement constitutes a renewal process.

Renewal theory technique has helped in solving many industrial problems. Although, renewal theory has been used to solve problems in diversified fields, the selection of its application in maintenance and replacement strategies as a basis of this paper, is due to the fact that this area is believed to be one of the major areas of application of this technique.

This paper is the first of a series of articles in this area and the result of the author's research in this field of interest over past few years during his stay in UK and in subsequent years in his home country, Pakistan. In this article, an attempt has been made to cover a thorough survey of the preventive maintenance strategies. The next article consists of the application of renewal theory to obtain the operating characteristics of these strategies. Frequent use has been made of the work of the major contributors in this field. A fairly detailed bibliography has also been provided for the further interest of the reader.

2.0 TWO MAIN CLASSES OF MAINTENANCE AND REPLACEMENT THEORY.

Most of the earlier literature on the maintenance theory has been confined to the determinstic problems, that is, the problems in which the outcome of every maintenance action is non-random. As distinct from this case, most of the work done on maintenance and replacement strategies for the stochastically failing equipment is relatively recent.

2.1 MAINTENANCE POLICIES FOR STOCHASTICALLY FAILING EQUIPMENT.

In such problems the out-come of every maintenance action may be random. Problems pertaining to complex electronic equipment like missiles, mo- dern aircraft, spacecraft equipment, communications, and computers, etc., have provided a large impetus for solving the problems of maintenance and replacement strategies for stochastic situations. A monograph on maintenance theory proposed by Jorgenson, et al (16) gives special consideration to topics such as preventive maintenance policies, due to the extensive work carried out by the authors at the RAND Corporation California.

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The state defining each piece of stochastically failing equipment, depends on how well the items meet specified performance characteristics. Extreme states are 'good' and 'failed', although it is helpful to distinguish the additional states between these extremes.

We have further to assume that:—

a) Whether there are two states or many; Transitions from one to another are assumed to be governed by a probabilistic mechanism. This means that changes of state are stochastic in character;

b) there is zero probability of transition from failure to any other state without some kind of maintenance action; this means that failure is an absorbing state, this property being called the minimal characterisation of such a mechanism.

Although the stochastically failing equipment in the simplest physical situation can take either a good or failed state, the probability law describing the changes from good to bad state can be represented by the equipment's reliability function. Otherwise, this probability law can also be represented by the distribution of time between equipment installation and equipment failure. In other words, this is the equipment's failure distribution because time to failure is a random variable.

2.1.1 TWO CASES OF UNCERTAINTY DURING THE MAINTENANCE OF STOCHISTICALLY FAILING EQUIPMENT.

PREVENTIVE MAINTENANCE AND PREPARENDNESS MAINTENANCE MODELS.

There are two cases of uncertainty during the maintenance of stochastically failing equipment. In the first case we are uncertain whether failure will occur or not, that is, whether transition from one state to the other will occur. In the second case, we are uncertain about the actual state of the system, unless some definite action is taken. The preventive maintenance model and the preparendness maintenance model are applied respectively for the two corresponding uncertainties.

A) PREVENTIVE MAINTENANCE STRATEGIES.

In these models the states of equipment is always known with certainty. We can assume that the equipment has a continuous operation which ensures the knowledge of the state of the equipment with certainty. Otherwise, it may be assumed that the equipment is kept in storage for use in emergencies, but is under continuous surveillance. It is also assumed that the replacement is made with an identical component, with the replacement costs depending on the state of the equipment when it is replaced. It is presumed that replacement costs after an operational failure are higher than when equipment is still good. We have said previously that in these policies, the equipment's failure cannot be predicted with certainty. This uncertainty, however, does not preclude replacement at some time prior to an actual failure. This is however, valid only for equipment with increasing failure rate. The failure rate is increasing only if the second derivative of ln Ψ (y), is non-positive, i.e. only if ln Ψ (y) is concave, where Ψ(y) denotes Survivor mortality.

i) THE AGE MODEL

A policy where the decision to replace an item depends only on the age of the component was first of all investigated by Senju, et al (25-28). They have concenterated their attention on control limit policies, these policies are characterised by the rule - replace the component when it fails or if its age exceeds a fixed limit. Assuming the life of machine parts to be stochastic variables, some formulae for determining the optimal time of parts replacement were developed, in order to minimise the total cost from the point of view of probability theory. This was done for three replacement models (Fig. 1). The first case is the one where every machine part is replaced by a new one after its natural life expires. The second case is the one where a machine part which survives a time interval y_p, is replaced by a new one without waiting for its natural life to expire. In the third case which occurs when a large number of machines of the same type are in a factory, it is better to replace all parts together at regular intervals, regardless of the time duration since their last renewal.

Woodman (36) has dealt with the age model very thoroughly, and for the first time contained the necessary and sufficient conditions for the optimality of control limit rules. According to him Barlow, et al (5) having shown that there is an optimal non-random control limit rule amongst the whole set of possible control limit rules, do not consider whether there may be better rules which do not belong to the class at all. He has, therefore, obtained the conditions underwhich the control limit policies are optimal. This, he has done by examining how control limits can be derived, and by studying the statistical aspects of the policy. The dynamic programming formulation is inserted by defining C (a,y) as the expected cost of operating an optimal replacement policy, starting with a component of age 'a' when there is time y to go to system termination.

Barlow and Hunter (3) considered two type of models. They used renewal theory to obtain them. The optimum policies were determined, in each case, as unique solutions of certain integral equations depending on the distribution. One policy was useful in maintaining simple equipment and the other was useful in maintaining large, complex equipment. A complex equipment such as computers, normally requires a preventive maintenance policy which is commonly scheduled after a certain number of operating hours have accumulated. Between maintenance periods, failures are repaired as quickly as possible. For less complex equipment, repair at the time of failure (or replacement) may actually correspond to general overhaul. Obviously each equipment will have a different preventive maintenance policy. These two polcies are defined as follows.

Type I Policy: Perform preventive maintenance after y_o hours of continuous operation without failure. The possibility of y_o being infinite, and consequently no preventive maintenance taking place, exists. If the system fails before y_o hours have elapsed, perform maintenance at the time of failure. Preventive maintenance is then re-scheduled. For this policy, it is assumed that the system is as good as new after any type of maintenance (or replacement) is performed.

Type II Policy: Perform preventive maintenance on the system after it has been operting a total of y^* hours regardless of the number of intervening failures. Again, the possibility of y^* being infinite exists. It is assumed that after each failure only minimal repair, is made which does not disturb the system failure rate at all. Also it is assumed that the system is 'as good as new' after the preventive maintenance is performed. Of course, this assumption is clearly no restriction if the system is actually replaced.

It was proved that under certain reasonable restrictions, Types I and II strategies have unique solutions which can be computed and their efficiencies can be compared. The optimal y_0 and y^* depend upon the ratios Y_e/Y_s and Y_m/Y_s, respectively, where

Y_e = the expected time to perform emergency maintenance

Y_s = the expected time to perform scheduled maintenance

Y_m = the expected time to perform minimal repair.

When average tir 1es to repair are replaced by costs to repair, the models give the minimum cost solutions. If $Y_e = Y_s$ and $Y_m > Y_s / \mu Z(\mu)$, then Type I policy is better than Type II policy. If $Y_e = Y_s$, Φ is N $(\mu \theta)$ and $\mu / \theta > 2\pi$. $Y_s/2\, Y_m$ then Type II us better than Type I. If Φ follows the third asymptotic distribution of the smallest extremes, then it can be proved that for $Y_e = Y_s$ and $Y_m > Y_c$, Policy I is best. For $Y_s = Y_s$ and $Y_m < Y_c$, Policy II is best (3).

Besides considering the strictly periodic and random periodic policies, Barlow et al also investigated the sequentially determined replacement model (4) . The basis for the development of these three models was the renewal equation (A.4) and the following equations.

$k(y) = k_1\ E\ [N_1\ (y)] + K_2\ E\ [N_2\ (y)]$ for a finite time span . . . .(1)

and

$\lim\limits_{y \to \infty} \frac{(y)}{y}$ for an infinite time span . . . . . .(2)

where,

$k(y)$ = expected cost during $\overline{o,y}$

k_1 = cost that includes all costs from a failure

k_2 = a cost which is less than k_1, which includes all costs incurred for each non-failed item that is exchanged

$N_1(y)$ = number of actual failures in $\overline{o,y}$

$N_2(y)$ = number of actual exchanges in $\overline{o,y}$

The problem is to characterise the replacement procedure which minimises Equations (1), or (2), over the class of sequentially determined replacement procedures. Since it may not be possible to follow an optimal sequential policy, the quantities of interest for any random periodic replacement procedure should be obtained.

Optimal policies for both the strictly periodic and the random periodic policies exist.

Renewal equations for the computation of various quantities of interest such as the expected cost, the expected number of actual failure, and the expected number of planned replacements, etc. can be calculated for the random periodic replacement models. It can be shown that the cost under the optimal random periodic policy will improve by a simple conversion to sequential policy. A random period policy differs from a periodic policy (Type I) in that y is a random variable having a distribution Φ. On the other hand, when the lifetime of the investment process is 'finite', a sequential preventive maintenance policy is always advantageous. This policy is the one in which the replacement interval whether random or not is determined at each removal, in accordance with the time remaining to the time span. Since, the mandatory replacement age is recalculated after each replacement, this replacement age is chosen to minimise the expected cost of equipment operation over the remaining lifetime of the investment process (8,15).

Hosford (15) provided in equation to determine the probability that the system is operable at the start of an interval, if the system repeats the series of different intervals (duty, off, warming up, etc.) in identical form.

Morse (24) too considered a policy which was similar in nature to Barlow and Proschan's (3,4) strictly periodic replacement policy. He dealt with the infinite time span case. Failed items were repaired and restored to their normal operating condition. Preventive maintenance was performed if y hours elapsed without a failure. The objective function was the expected fractional operating time. This is similar to Barlow and Prochan's case which consisted of a solution to a replacement problem for an infinite time span if mean time to perform a repair is replaced by k_1 and mean time to perform preventive maintenance is replaced by k_2.

Weiss (34) presented a more general shape of the age model for infinite time spain. In addition to the effects of failure and replacement, the effects of repair (specially the time, considered random, and the cost of repair) were considered. He presented several simplified examples of the calculation of optimal replacement policies, but showed that an optimal policy need not always exist. In another paper (32), he studied the reliability function and moments of a machine which is periodically replaced in order to increase the mean time to failure. The two cases considered were those of strictly periodic replacement and randomly periodic replacement.

A general model for the reliability analysis of system under various preventive maintenance policies was dealt with by Flehinger (13). Many statistical results were obtained when the control limit is defined by a random variable. Besides the age model, the 'block replacement', system checkouts' and 'marginal testing' cases were also considered. A wide use made of the results of renewal theory and the theory of regenerative stochastic processes, to explore the asyimtotic behaviour of the quantities E $[N_1\ (y)]$E $[N_2\ (y)]$and $\psi\ (y)$ as defined in the Appendix A. The specise assumptions that defined the proposed general model were formulated and the integral equations which determined the significant measures of performance were developed. Smith's results of renewal theory and analysis of equilibrium and cumulative process (28,29) were applied to give the asymptotic values of these performance measures.

So far, we have mentioned the Type I and Type II preventive maintenance policies (3); the former is very useful in maintaining simple equipment and the latter is useful in maintaining large, complex systems. It should also be mentioned here that a Type III policy exists (2) which is a further improvement over Type II policy.

Preventive Maintenance Policy III: Type III policy can be defined by the following characteristics:

(a) Life distribution functions at the time epoch when the overhaul is complete, are not necessarily identical. In the Type II policy, they were identical. The mean life time μ_1, μ_2, $\cdots$ μ_q are positive and finite with probabilities $p_1, p_2, \ldots\ldots\ldots\ldots p_q$.

(b)The life distribution of the rth system can be denoted as usual by $\Phi_r\ (y)$ and its mortality function as $\phi_r(y)$.

Obviously,

$$\Phi_r\ (y) = \psi_r\ (y) = o \text{ for all } y < o \quad \ldots\ldots (3)$$

and if the failure rate is $Z\ (y) = \psi_r\ (y) \,/\, (1 - \Phi\ (y))$, then

$$\mu_r = \int_0^\infty y\, d\,\Phi(y) \quad \text{and} \quad \mu = \sum_{r=1}^{q} p_r\, \mu_r \quad .(4)$$

In order to compare the Type II with Type III Policy, let us recall the following drawbacks of the Type II policy. When $\mu < \mu_r$ the optimal period y^* for rth system must be longer to overhaul in some cases and too slow in the other cases. Also, we know that originally, it is not possible to operate the system for some hours, after the time point of the overhaul schedule in case of Policy II.

These two drawbacks of Type II policy can be removed if it is considered under assumption (a) and (b) above. Thus, we are in a position to define Type III policy as follows:

"For the first n-1 times of failure of the system we perform a minimal repair and the time of nth failure we overhaul it". Further characteristics of this policy have been developed by means of renewal theory (14).

Further characteristics were investigated by Makabe and Morimura (21) and the investigation was extended to the case where the object function is taken as the maintenance cost rate. A combination of Type II and III policies was introduced in the shape of Type IV policy, defined as follows:

Preventive Maintenance Policy IV:

"When the total running time is y^* or when the number of times of failure counted from the last overhaul is n_o, perform an overhaul. Otherwise, perform a minimal repair".

This overhauled system is considered to enter a new generation. Interval reliability of this policy is given by

$$R^{(4)}(n, y^*, z) = \frac{1}{u_{n,y^*}} \sum_{r=0}^{n-1} \int_0^{y^*-z} \frac{(x/\nu)^r}{r!} e^{-((x+z)/\nu)^\beta} dx \quad \ldots (5)$$

by using the key renewal therorem, where Un, y^* is an expected time length of a renewal cycle.

Preventive Maintenance Policy V:

(a) Type V Policy : Let $[a_r]$ denote a pre-assigned of non-negative numbers (allowable the infinity), and at rth failure ($r = 1, 2, \ldots, n, \ldots$), perform an overhaul if $H_r > a_r$, and perform a minimal repair if $H_r \leq a_r$.
This policy would be denoted as policy V_D when $[a_r]$ is a monotone non-increasing sequence and policy V_A when $[a_y]$ is a monotone non-decreasing sequence.

(b) Type V' Policy: If the sign of inequality is conversed, Type V' Policy is obtained. Policies V'_D and V'_A can be defined analogously.

(c) Type II' Policy: putting $a_r = \mathcal{Y}$ in the above definition of policy V we obtian a similar policty to policy II. But, in this case the system may be operated somewhat after H(y) build up to $\mathcal{Y}$.

(d) Tupe IV' Policy: Similarly to the above, putting $a_r = \mathcal{Y}$ for $r < n$, and $a_r = o$ for $r \geq n$, we obtain a similar policy to IV.

(e) Tupe IV'' Policy: Moreover, putting

$$a_1 = a_2 \ldots\ldots = a_{n_1} = \infty,$$

$$a_{n_1} + 1 = \ldots . = a_{n_2} = \mathcal{Y}, a_{n_2} + 1 =$$

$\ldots . = 0$, in policy V, we get an extended policy of policy IV'.

By using the above definitions, an optimal Type V was obtained. Furthermore, it was found numerically that optimal policy of Type III is rather robust (22). In Reference 20 the existence and uniqueness of the optimal policy of Type III were shown for the underlying Weibull mortality. Later, this discussion was generalised to the case of strictly IFR which covers a sufficiently wide class of preventive maintenance problems (23). The argument of the proof of theorem 5.1 given in Reference 21 was cancelled and the optimality was discussed under weaker conditions (23). Another policy called policy III' was introduced in this paper.

(f) Type III' Policy: In policy V, put $a_1 = \infty$ for $r < n$, $a_n = \xi$ and $a_r = o$ for $r > n$ where ξ is a non-negative real number.

The discussion of the existence and uniqueness of the optimal Type III policy in the sense of limiting efficiency, was generalised to the concept of the maintenance cost rate by the following theorem.

Theorem: "If the failure distribution of a system is strictly IFR, then there exists an optimal policy of Type III in the sense of maintenance cost rate. The optimal policy is determined uniquely except for the case in which the succeeding two values of n give the same maintenance cost rate"

Optimal type III' and type IV' were also obtained.

In the case of a finite state space model analogous to age model and discrete time, it was shown by Derman (9) that control limit rules were optimal if the transmission matrix defining the process had an increasing form. Considering the age replacement policy case, it is observed that a component with an increasing failure rate is sufficient to ensure the optimality of control limit rules.

We have already seen that the age Replacement Policy has received maximum attention in the literature. Derman and Sacks (10) obtained the optimal replacement policy for a piece of equipment in which the decision to replace depends on the observed stage of equipment deterioration at specified points in time.

In another paper, C. Derman (8) observed a system periodically and classified it into one of a finite number of states. He then made certain maintenance or replacement decisions on the basis of the observations, enabling the transition probabilities to be made from state to state. Thus, he found the decision rule which maximised the expected length of time between replacements, subject to the side conditions that the probabilities of replacement through certain undesirable states were bounded by prescribed numbers. Linear programming formulation was used.

The earliest documented approach to the planned replacement problems was made by Campbell (7) an early worker in the field of operations research. His results which took the form of complex integral equations were concerned only with mass replacement and not with individual item replacement. He dealt mainly with the replacement of light bulbs. Welker's (35) work was also developed on the same lines and it was not possible to use interpolation with the plotted results. In the field of commercial airlines, Kolner (19) introduced the life characteristics through the failure rate and the mean life of failed engines, both as functions of mandatory overhaul age. Thus, in this manner he developed a workable method for determining optimum mandatory overhaul ages for aircraft engines. A main attraction of his method was the incorporation of the alternative of 'repair' of in-service failures in addition to the more expensive overhaul. He assumed that the life characteristics are linear, so that the method is useful only for an extremely limited range of the overhaul age. However, in Airline operation, these assumptions of limited range of overhaul age and the 'repaired' engines running to overhaul age (which is contradicted in practice) offer no problem, because of FAA ceiling which exists.

We have mentioned the work of Barlow, Hunter and Proschan at various points in this paper; most of their important studies were carried out at Sylvania (2,4). Taylor (31) suggested another criterion for determining whether a specific small increase in mandatory overhaul age is economically justified. He states that life extension is 'profitable' if and only if:

Rate of decrease in total overhaul rate at current life/Rate of increase in average failure rate at current life $>$ Average additional induced cost of a failure/Average cost of an overhaul.

Klein and Rosenberg (18) presented a survey paper to describe the problems relating to the management of goods that deteriorate. They divided this class of items into a 'constant' efficiency' and a diminishing efficiency' group. The problems dealing with the equipment are concerned with the optimal timing of decisions to replace or repair a piece of equipment which deteriorates in some known way or in some probabilistic way.

Many authors have considered the effects of removing components which survive to some predetermined age (6,11,12,33) but the model which includes the fects of marginal testing or of intermittent nponent usage has been neglected.

Kamins and Mc l (17) carried out their studies on maintenance policies for aircraft and missile parts. They found from the failure data that many parts failed exponentially. The optimum replacement policy in this case: Never plan to replace before failure, regardless of how expensive or inconvenient an in-service failure becomes. A substantial portion of the failure data also exhibited an increasing failure rate. In such situation, given aging effect, the higher the relative cost of an inservices failure, the shorter should be the planned replacement interval. Similarly, for a given in-service failure cost, the more severe the aging effect, the shorter should be the replacement interval. Specific replacement policies for the underlying Normal, Log-normal, Gamma, and Weibull distributions, were also obtained. A computational routine was developed to determine the optimum planned replacement interval for parts whose failure characteristics are best described by some of the discrete probability distributions. The gross savings achieved by following a planned-replacement policy rather than a replacement-at-failure policy were computed for the discrete and for the four continuous distributions.

An improvement over age model:

Bansard et al (1) introduced a trigger-off policy as an improvement over age model. In this policy, at the time of failure those parts are replaced which have exceeded an age limit, in order to reduce the frequency of stoppages. He argued that in the age model, replacing parts one by one whenever a failure occurred or when an age limit is reached, would indeed cause frequent stoppages to machinery that consists of a number of components. Such stoppages often resulted in high costs of maintenance, reduced availability, and possibly a loss of income Bansard's model consisted of a stochastic renewal process. It is a continuous Markov process whose state at any moment of time is defined by the age of the various elements of an item of equipment. The trigger-off policy created stochastic dependence between the renewal chain of any particular part and that of all the others. The age limits were decision variables of the policy. There exists an optimal set of age limits satisfying both economic and technical requirements such as: the reliability of each part, reduced availability and/or loss of income, and replacement cost of each part. The solutions were obtained for various distributions which gave a measure of the influence which such a policy will have on the flow of items under repair and on the availability of equipment. Some simplifying assumptions based on the fact that the different renewal chains become less dependent on each other as their numbers grow, were introduced because the general case of more than two components could not have been easily expressed without them. According to these assumptions, a more general policy could be studied which included a new decision variable; namely, the maximum number of components to be replaced at each breakdown.

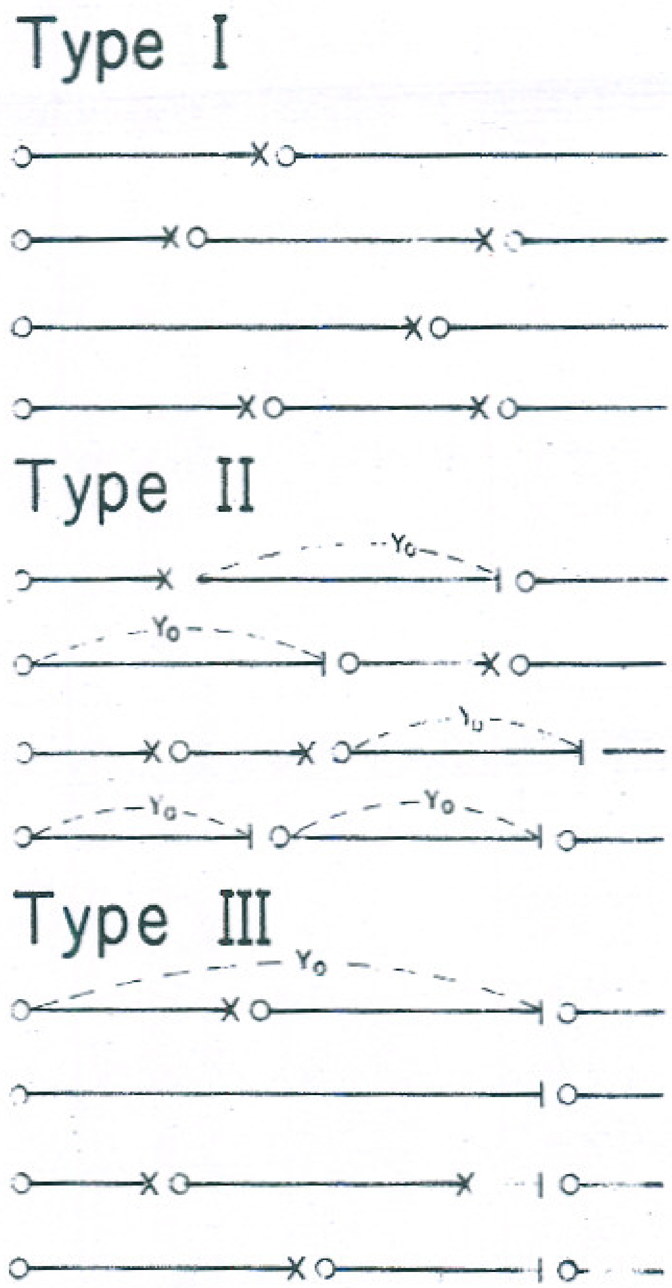

FIG. I Three types of replacement policies

compared by SENJU

APPENDIX A

Definitions:*

1. The Renewal Process:

   If [(Y_1, Y_2, Y_3, ...,] are a sequence of independent, non-negative, identically distributed random variables which are not all zero with probability one, then they are said to constitute a renewal process.

   The epoch of the nth occurence is given by the sum, $S_n = Y_1 + Y_2 + \ldots + Yn$,

   and [Y_1, Y_2, ...] is called a renewal process.

2

2. The time upto nth renewal.

   If φ_n (y) is the mortality and Φ_n (y) is the commulative Mortality of S_n, then, for ordinary renewal process $\varphi_n^*(o)$ (s) $= [\varphi^*(S)]$ . . . .(A.1)

3. The number of renewals in a random time:

   If Y is a random variable independent of [Y_1 Y_2, ...] and N is the number of renewals in $\overline{o,y}$. then

   generating function A (Z) of N is given by

   A (Ƶ) = $\int_o^\infty$ A (y, Ƶ) φ (y) dy . . . . (A. 2)

   where A (y, Ƶ) = probability generating function of N_y.

   and p (y) = mortality function of Y.

4. The number of renewals in $\overline{o, y}$

   N_y.

   Renewal Theory has been defined sometimes in the literature as the study of the random variable N_y where N_y is the bigest value of n for which $S_n \leq y$ subject to the convention N_y = O if $Y1 > y$.

   Also.

   $P [N_y \geq n] = \Phi_n(y) = \Phi_n(n)$

   Therefore, the probability distribution of N_y can be obtained for all values of n, explictly.

5. Renewal function, U (y)

   The renewal function U (y) equals the expected number of renewal epochs in the internal $\overline{o, y}$, the origin counting as renewal epoch. Thus,

   U (y) = $\int_o^y$ [1 + U (y-z)] d Φ (z) .(A.4)

   which is the renewal equation for the mean value function of a renewal counting process.

6. The renewal density.

   The renewal density is given by

   $u\ (y) = \dfrac{d\ U\ (y)}{dy}$

   or $u_{(o)}$ (y) = φ (y) + $\int_o^y$ u O) (y-z) φ (Ƶ) d Ƶ . . . . (A. 5)

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* For detailed analysis of the renewal theory results, refer to

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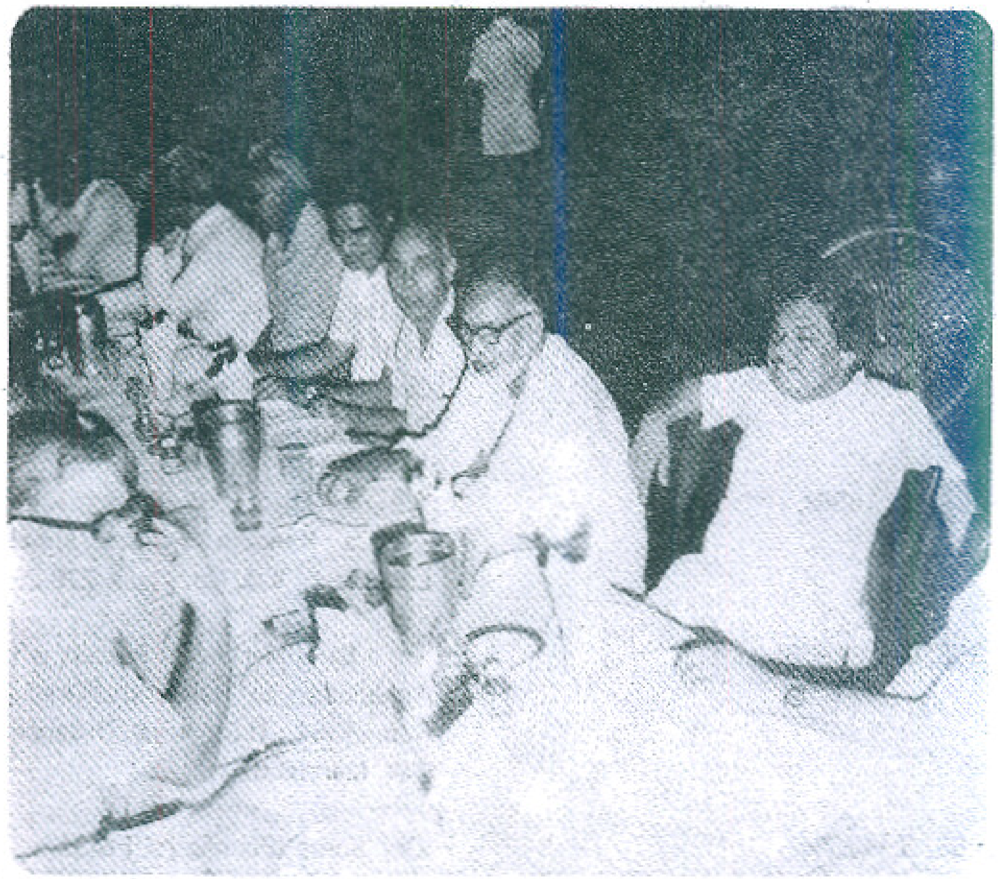

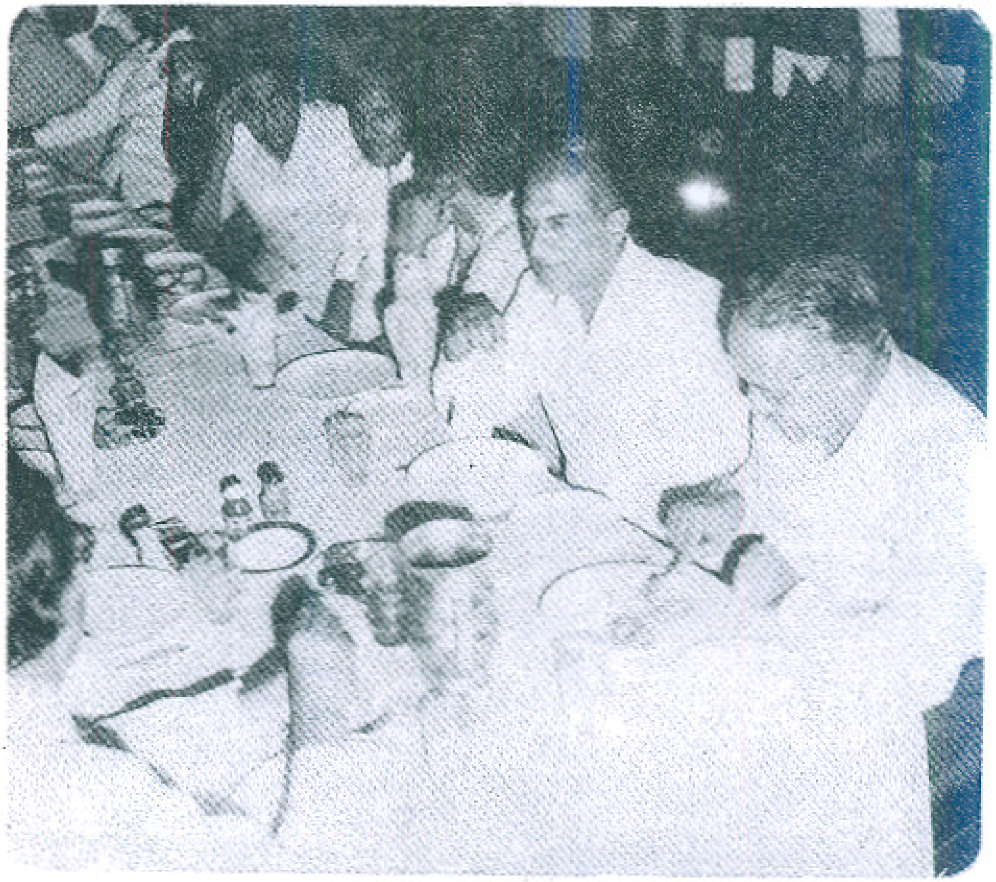

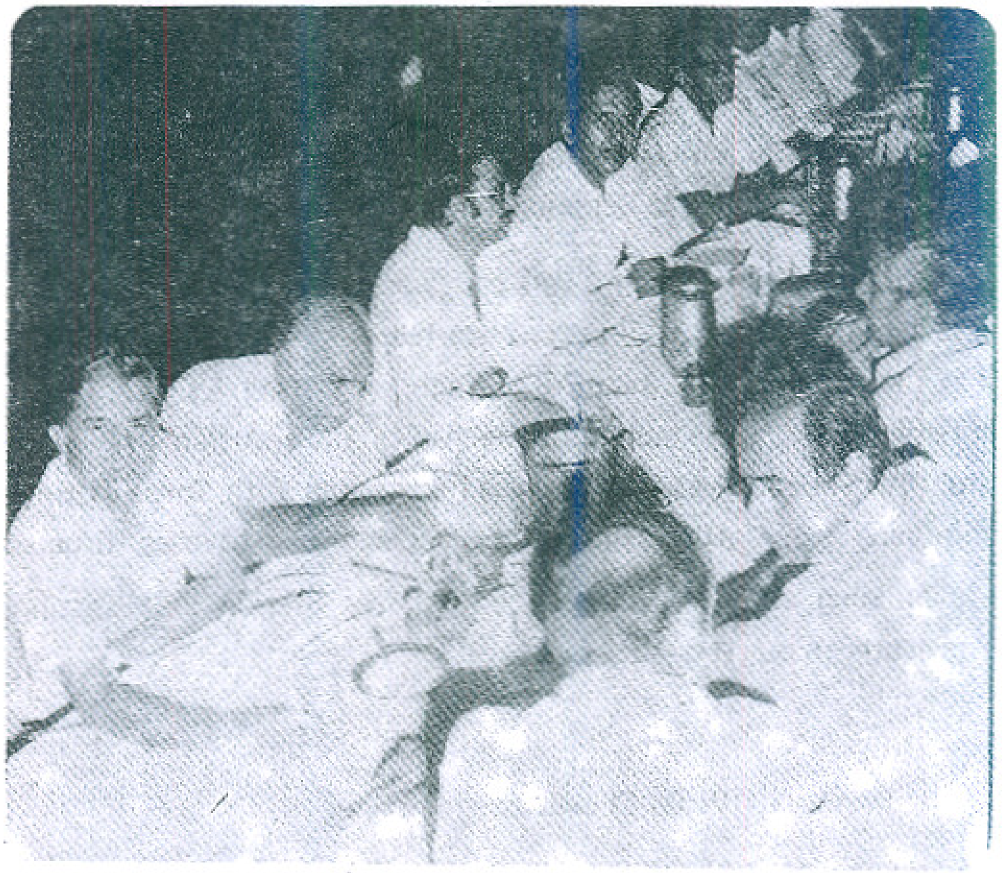

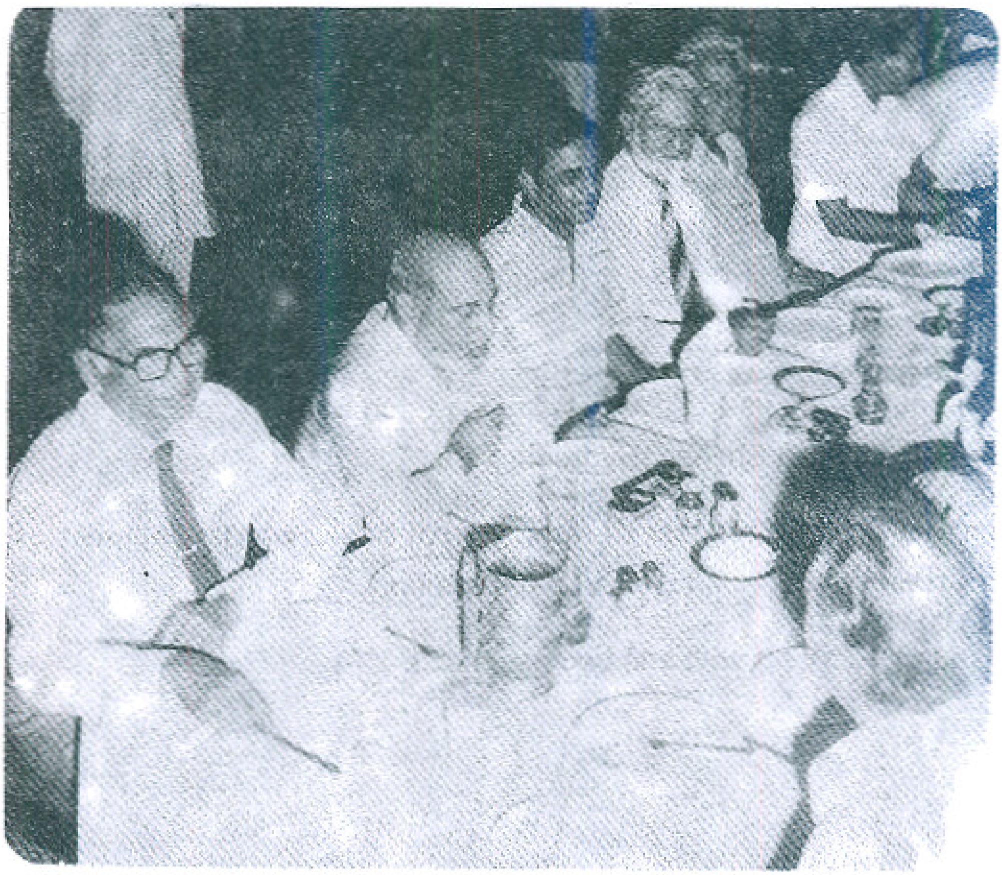

OBITUARY

Arthur Casagrande dies

A. casagrande, a founding father of modern soil mechanics and a protege of Kari Terzaghi died in September 1981, at the age of 79 years.

He was native of Austria and was educated at the Technical Univ in Vienna. He came to U.S.A in 1926 and started work as assistant to Karl Terzaghi at the Massachusetts Institute of Technology (M.I.T.). From 1932 to his retirement in 1973, he taught soil mechanics & foundation engineering at Harvard Univ. For 40 years he remained associated with Corps of Engineers in training army officers in soil machanics. They have constructed casagrande building at their Waterway Experiment Station in Mississippi for housing geotechnical research.

Vente Chow Passes away

DR Ven Te Chow, world famous hydraulician & teacher passed away on 30.7.1981 in Urbana, Illinois, U.S.A. He was Professor of Civil Engineering at the Univ: of Illinois.–

Professor Chow was born in China on 14.8.1919 and received his degree in civil engineering from National Chiao Tung Univ: China & Ph. D. in hydraulic engineering from Univ of Illinois, U.S.A. and came on the faculty of this Univ in 1948.

He was author of several books including Hand book of applied Hydrology, a treatise on Open Channel Hydraulics etc.

He served on large number of international bodies on development of Water resources and won several awards and honours.

He is survived by his wife and two daughters.

۵۔ وَلَا تَأْكُلُوٓا أَمْوَالَكُمْ بَيْنَكُمْ بِالْبَاطِلِ وَتُدْلُوا بِهَآ إِلَى الْحُكَّامِ لِتَأْكُلُوا فَرِيقًا مِّنْ أَمْوَالِ النَّاسِ بِالْإِثْمِ وَأَنْتُمْ تَعْلَمُونَ ۝

"And swallow not up your property among yourselves by false means, nor seek to gain access thereby to the judges, so that you may swallow up a part of the property of men wrongfully while you know". ii : 188

. You shall not abuse your position or power, nor accept illegal gratification of any sort.

۶۔ وَقُولُوا قَوْلًا سَدِيدًا ۝

"And speak straight words." xxxiii : 70

5. You shall express your opinion on engineering or other matters in a frank, open and straightforward manner.

۷۔ اجْتَنِبُوا كَثِيرًا مِّنَ الظَّنِّ إِنَّ بَعْضَ الظَّنِّ إِثْمٌ وَلَا تَجَسَّسُوا وَلَا يَغْتَبْ بَّعْضُكُمْ بَعْضًا

"Avoid most of suspicion for surely suspicion in some cases is sin; and spy not nor let some of you backbite others". xlix : 12

7. You shall not criticise another engineer's work without his knowledge, nor malign, or injure his professional reputation.

۸۔ وَلَا تَقْفُ مَا لَيْسَ لَكَ بِهِ عِلْمٌ إِنَّ السَّمْعَ وَالْبَصَرَ وَالْفُؤَادَ كُلُّ أُولَٰٓئِكَ كَانَ عَنْهُ مَسْئُولًا ۝

"And follow not that of which thou hast no knowledge. Surely the hearing and the sight and the heart, of all these it will be asked." xvii : 36

8. Your professional advice shall be based on full knowledge of the facts and honest conviction, and you shall not write articles or advertise in self-laudatory language or in any manner derogatory to the dignity of the profession.

۹۔ وَتَعَاوَنُوا عَلَى الْبِرِّ وَالتَّقْوَىٰ وَلَا تَعَاوَنُوا عَلَى الْإِثْمِ وَالْعُدْوَانِ وَاتَّقُوا اللَّهَ

"And help one another in righteousness and piety, and help not one another in sin and aggression and keep your duty to God." v : 2

9. You shall help one another in upholding and doing what is right, and shall not associate with those who transgress and those who indulge in unethical practices.

۱۰۔ وَأَمْرُهُمْ شُورَىٰ بَيْنَهُمْ

"And whose affairs are decided by counsel among themselves." xlii : 38

10. You shall decide matters of common professional interest by mutual consultation.

۱۱۔ وَاعْتَصِمُوا بِحَبْلِ اللَّهِ جَمِيعًا وَّلَا تَفَرَّقُوا

"And hold fast by the covenant of God all together and be not disunited." iii : 102

11. You shall strive individually and collectively to enhance the prestige of the engineering profession by ordering your conduct in accordance with this Code of Ethics, and shall not be disunited.