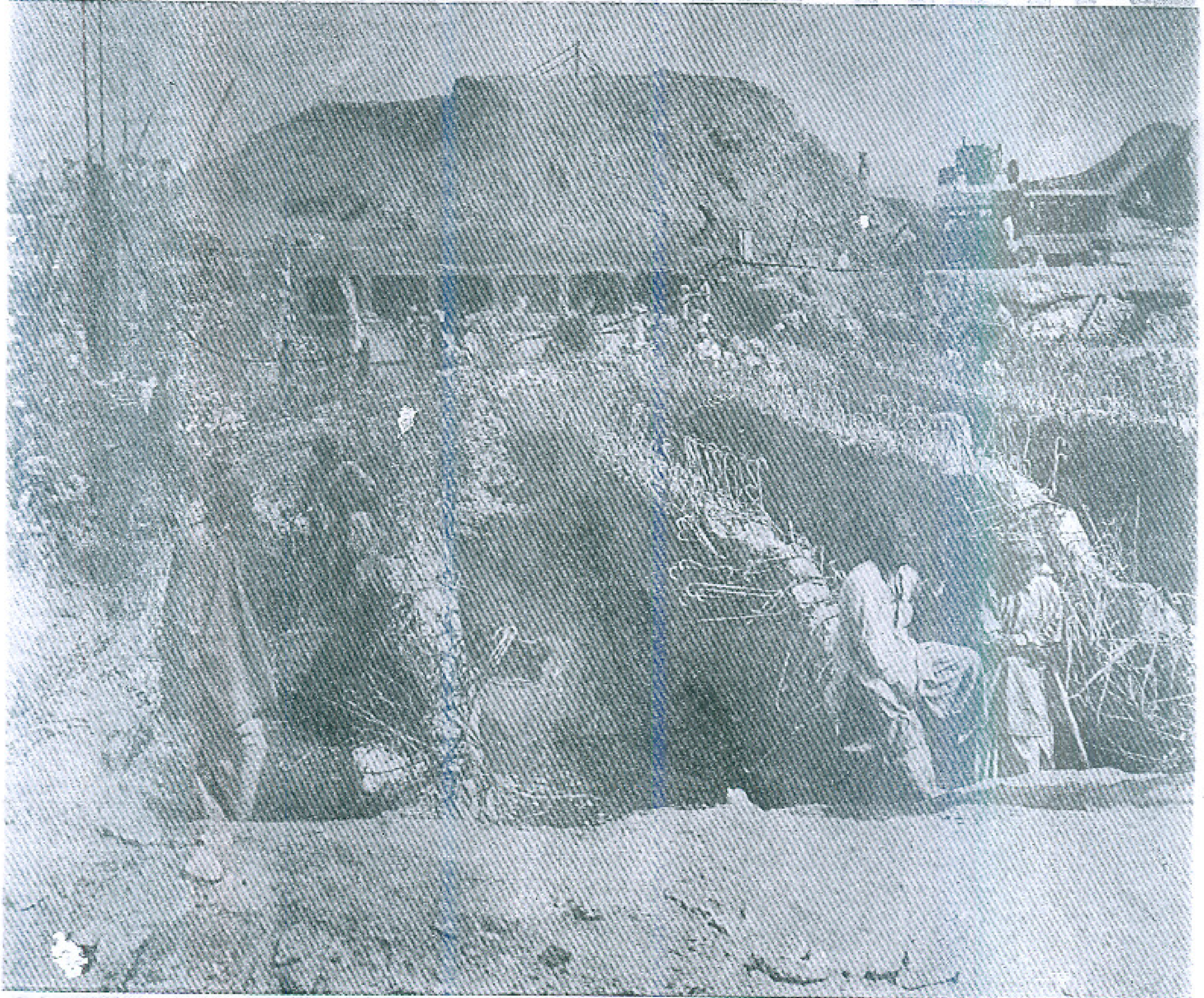
Vol. 16 No. 3
SEPTEMBER, 1971

ENGINEERING NEWS



SIXTEENTH YEAR OF PUBLICATION

ENGINEERING NEWS

Quarterly Journal of the West Pakistan Engineering Congress

Vol. XVI SEPTEMBER, 1971 No. 3

• All communications should be addressed to the Editor, *Engineering News*, P. W. D. Secretariat, Lahore (W. Pak.)

• Price Rs. 2.50 per copy. Rs. 10.00 a year in advance. Free to members of the West Pakistan Engineering Congress. Change of address should be intimated promptly giving old as well as new address along with membership number.

• Contributions to this journal in the form of articles, news of engineering works, news about engineers, photographs and technical data etc. are cordially invited.

• Reprints from this journal be made on condition that reference is given to the *Engineering News*, its Vol. No., and the author.

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Price of this Issue : Rs. 2.50

In this issue

TITLE COVER

A view of dismantling of the damaged syphon at R. D. 121560 *B. S. Link I.*

Printed by Mirza Muhammad Sadiq at Ripon Printing Press Ltd., Bull Road, Lahore.

CREATE COMPETENCY AND APPLY IT HONESTLY

Corruption is the cancerous malady that is eating the very vitals of this nation. It is devastatingly rampant in all facets of national life. How tragic it is that a community claiming to be the follower of the divine morality of the revealed religion is crowded with acquisitiveness, rapacity, double standards and ruthless pursuit of an ignoble objective to accumulate the worldly gains. What for is all this degradation—just for the sake of some unearned comfort of life or for the vain effort of capturing those fleeting moments of sensual pleasure or to attain that illusory objective of a high standard of living. Let us search our hearts for a forthright answer to a question if the degradation that we suffer to our soul and mind is adequately compensated by a little more butter on a few more crumbs of bread. Every time this "masterpiece of creation" succumbs to the temptations of baser passions, the humanity gets diminished. We thus commit a crime against humanity. Why do we indulge in it? Is that we have no nobler objective to sustain us in abstention or we have no faith in the unalterable moral values revealed by the Creator which are to purify our mind and soul to equip us to fulfil the objective of creation. The agonising questions are more than answered by our conduct. Yes we are not familiar with the pleasure of devotion to a higher ideal. We have not learnt to practise the virtues of living for the whole of community—for the whole of humanity. That is the short-coming of our education and more so of our training. We have not been introduced to the charm of living for values not measureable by gold or gun. The whole community seems to have drifted that way in an atmosphere of complacency. Is that the normal posture of any living nation? No, it can not attain the status of accepted norms. History is replete with the story of nations meeting an ignominous end for persisting in corrupt practices of mind and body.

Some group somewhere, sometimes has to assume the role of pioneers to break this vicious circle. It is an unnatural state of affair and it must end as it is not in conformity with the Will of God. Allah has His way of honouring some individual or a group by choosing it to be an instrument in His hand to fulfil the ordained.

We the engineers have chosen a profession for ourselves which stands dignified by virtue of its service. The worthiness of this profession lies in employing high ethical standards of conduct in the service of nation, as is also true for all other professional groups. The obligation for this group to come forward at this critical juncture of national life is more than obvious. It is they who have the strength of confidence because they have the knowledge of natural laws to sustain them to steer a course of their own, through stormy waters of confusion. It is an onerous responsibility on them, but they have no option but to face the challenge as otherwise they will perish. We have to equip ourselves with technical competency as this is our armour in the nation's struggle to march forward. There is no substitute for this competency as not to have attained the proficiency is not to have laid the foundations. On this foundation we raise the edifice of magnificent service through the most honest practices based on truthfulness and sincerity. Do we pledge ourselves to do that?

West Pakistan Engineering Congress Council 1971-72

The members of the West Pakistan Engineering Congress elected unanimously Mr. I. A. S. Bokhari, Managing Director (Power), Wapda, as the new President for 1971-72. Five Vice-Presidents and other office-bearers were also elected by the members. At the first meeting of Executive Council, Malik Mohammad Ashraf, Director (Grid System Construction), Electricity Wapda was nominated as Honorary Secretary. The brief biographic sketch of Mr. Bokhari, the President, Secretary and two Vice-Presidents is being published in this issue of 'Engineering News'. The remaining office-bearers will be introduced in the coming issues.

PRESIDENT

Mr. I. A. S. BOKHARI

Born in Lahore in 1919, Mr. I. A. S. Bokhari graduated in Electrical Engineering from the Punjab University and joined the Punjab Electricity Department in October 1940. While in College, he was awarded Dunlop Smith scholarship for his brilliant academic career. During his service, he was selected by the Government in 1946 for advance education and training in United States under the Post-War Reconstruction Scheme. He received his Master's Degree in Electrical Engineering in 1948 with high merits from the Brooklyn Polytechnical Institute, New York and received practical training with the United States Bureau of Reclamation, Denver and Garik Oil and Transformer Company, New York.

He was appointed Project Engineer in 1949 in Punjab in which capacity he continued till 1958, when his services were requisitioned by the West Pakistan Water and Power Development Authority. Mr. Bokhari has been responsible for the planning and designing of electric supply facilities of the public sector in West Pakistan and has been actively associated with the Irrigation Department for the development of Hydro-electric power and execution of the salinity control and reclamation programme. His contribution towards power development schemes in West Pakistan has been singular. On reorganization of WAPDA, Mr. Bokhari was appointed Member and Managing Director (Power) in December 1969.

He was nominated as an Adviser on Federal Capital Commission and was appointed Convenor of the Committee on Gas and Power requirements for the new Capital at Islamabad. He was appointed Member of the Syndicate of the West

Pakistan University of Engineering and Technology. He has also served as a Member of the Engineers Problems Committee appointed by the Government of Pakistan to look into the problems of Engineering Services. He is a Fellow of the Institution of Electrical Engineers (Pakistan) and a Member of Institute of Engineers (Pakistan). He is a Member of the Engineering Congress with a long standing and has been actively participating in the affairs of the Engineering Congress in the past. He has served the Congress as a Secretary during 1963-65.

Mr. Bokhari has visited many countries including United States, Germany, Switzerland, Japan, Soviet Union, U.K. etc. and is widely known in the international engineering circles for all-round experience in power generation, transmission and distribution.

The Engineering News accords a hearty welcome to the new President of the Congress.

VICE-PRESIDENTS

(1) M. S. MINHAS, P.S.E.I.

Mr. Muhammad Saeed Minhas, one of the five Vice-Presidents of the year, is an Engineer with a brilliant service career.

Mr. Minhas graduated with Honours in 1939 from Government College, Lahore and Civil Engineer with Honours in 1942 from Punjab College of Engineering and Technology, Lahore.

He joined the Irrigation Department in 1943 and worked in field in different capacities and had an experience of both construction and maintenance of Canal System in Punjab. Mr. Minhas worked as Under-Secretary to Government of Punjab (Establishment) for two years (1950-52).

He was sent for special training in reclamation of saline and water-logged lands, ground water development, construction of tubewells, selection of pumps and design of tile drainage system. During his stay in the States, he worked with the U. S. Geological Survey and Bureau of Reclamation. For one semester got special training at the College of Agriculture Davis, California.

He remained for some time member, water delegation to the Indus Basin working party and then he worked in the ground water development organization. From 1957 to 1959, Mr. Minhas worked on the design and construction of all the works of Warsak Dam.

He was Director, Irrigation Research Institute, when he took assignment in East Pakistan Wapda, as Director Planning and Design and also Director Hydraulic Research Laboratory, Dacca.

In April, 1962, Mr. Minhas was sent on deputation to North Nigeria as Chief Irrigation Engineer. Till July, 1966, he was responsible for planning of water resources in Sokoto Rima Valley for Irrigation and Hydel Power Development. He organized Hydrological stations and arranged collections and completion of Hydrological data.

Mr. Minhas joined A. D. C. as Deputy Chief Engineer, small dams organization and then from 1960 to date he is working as Chief Engineer and Adviser, Agricultural Development Corporation, West Pakistan. He is a brilliant Civil Engineer and an active member of Engineering Organizations. He has varied type of publications including papers read by him on International forums. Such experienced members are an asset to the Engineering Congress.

(2) Mr. MAZHAR-UL-HAQ

Mr. Mazhar-ul-Haq the Vice-President, is a very active member of the West Pakistan Engineering Congress. Since long he has been in the Executive Council of the Congress and serving in one capacity or the other. A graduate of Civil Engineering from Punjab Engineering College, Mr. Mazhar-ul-Haq joined Punjab P.W.D. (Buildings and Roads) in February 1950 as an Assistant Engineer.

In 1953 he was attached to the UNESCO team for three months to assist in preparing development of Highways in Baluchistan area and then he was in charge of construction of Hill roads in Rawalpindi area. Up to 1957 he was working in B & R doing different jobs, both construction and design, when Mr. Mazhar-ul-Haq joined Public Health Division, Lahore. In 1962-63 he was entrusted with the construction of boundary pillars in riverain area between India and Pakistan.

In 1968, he was appointed as Superintending Engineer, Public Health Engineering Circle, Peshawar; in January, 1970 to date he is working as General Manager-cum-Project Director, Lahore Improvement Trust, Water Wing. This is a gigantic task of supplying good water to ever-increasing population of Greater Lahore and it is hoped he will serve the citizens of the Capital of the Punjab with zeal and gusto.

HONORARY SECRETARY

MALIK MUHAMMAD ASHRAF

Born at Mandi Bahauddin (District Gujrat) in 1928, graduated in Electrical Engineering from the Punjab University in 1951. Joined the Punjab Electricity Department in May, 1952 and has since worked in different capacities both in Construction and Distribution.

In 1967-68 received advanced training in United States in the field of Design and Construction of 220 kv Transmission Lines and Grid Stations.

Mr. Malik is at present working as Director (Grid System Construction) in charge of construction of transmission lines and grid stations throughout West Pakistan.

He is an active Member of the Institution of Electrical Engineers and Associate Member of the Institute of Engineers (Pakistan).

The members of the Engineering Congress earnestly hope that the new Secretary will do his best for the cause of Engineering profession.

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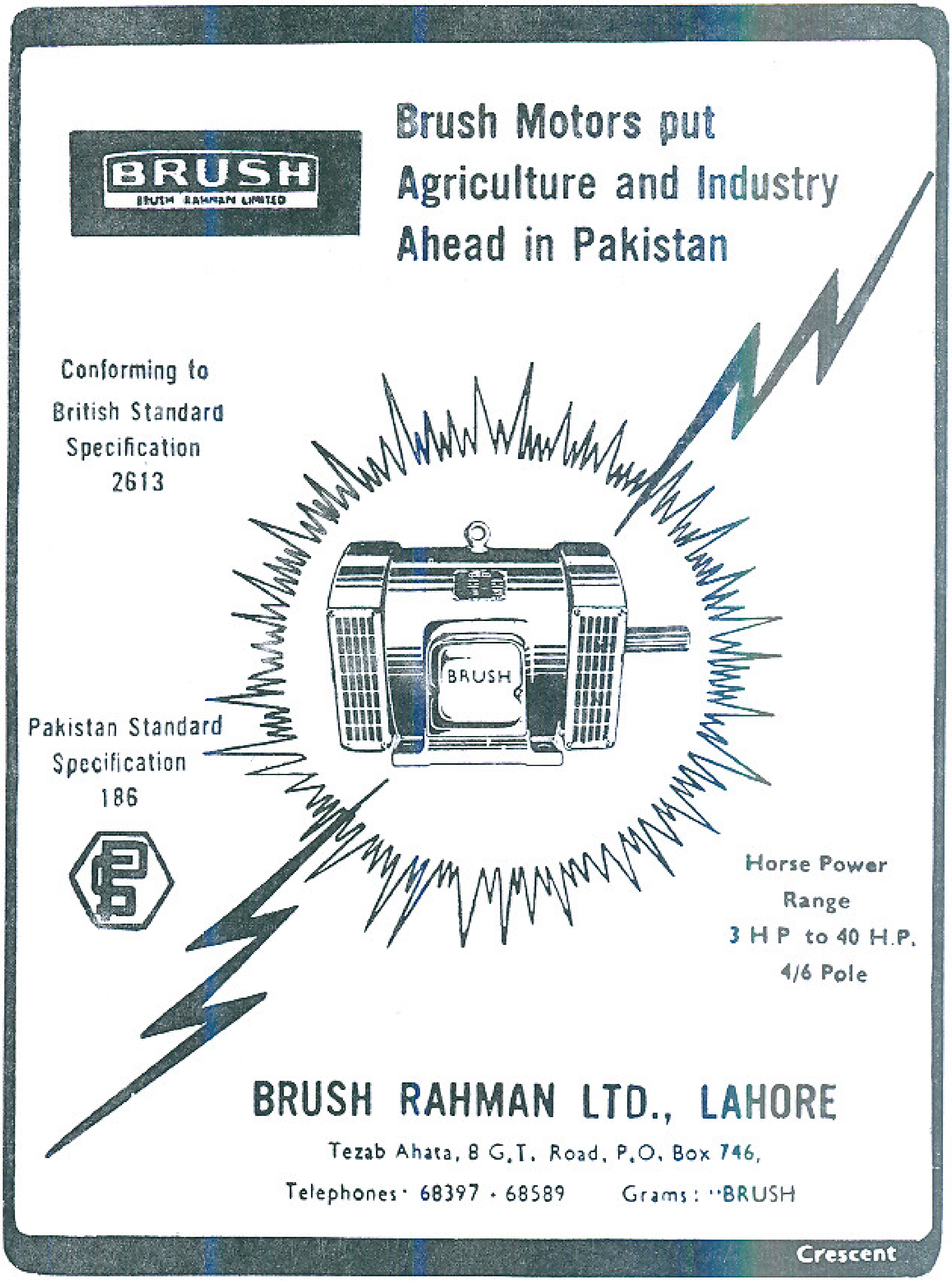
IRRIGATION AND POWER SECTION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Agricultural Development Possibilities in Non-Irrigated Areas of West Pakistan

By **M. ASHRAF ALI** *and* **AURANGZEB***

Non-irrigated cultivation (rain-fed cultivation and dry-farming) is mainly concentrated in the north-eastern plain parts of West Pakistan. Very small patches of torrent-watered cultivation occur in the piedmont plains of the arid western areas.

The Potwar uplands, discussed below, form the largest contiguous block of dry-land cultivation in West Pakistan, covering some three quarters of the total rainfed and dryfarmed acreage. Other significant areas are the Himalayan piedmont in the north-eastern fringe of Jhelum, Gujrat and Sialkot districts, Swabi Maira (the rolling sand plain) in Mardan district, Kohat valley and the piedmonts of the Marwat range in Bannu and D. I. Khan districts. The information presented below for the Potwar uplands has been summarized from Ashraf Ali 1967 *and Aurangzeb et al.* 1970. *A report on the Agricultural development possibilities of the Potwar uplands with a map at a scale of* 1 : 250,000 *is in preparation (Ashraf Ali and Aurangzeb* 1971).

THE POTWAR UPLANDS

General

Slightly over half of the Potwar uplands is cultivable, about a third has a potential for grazing, and the remainder is agriculturally unproductive. Scarcely a cultivable area is left untilled and there is little scope for extension of the cultivated area. Economic increases in production can only be obtained by modern management including selection of drought-resistant crops adapted to the climate and soil conditions, and by development of rangeland, depending on land capability.

The small areas of cultivable land adjacent to main streams are well suited to intensive irrigated agriculture, and a two to threefold increase in production can be obtained by improved agronomic practices. In the remainder of the area, scarcity of ground water, broken terrain and deeply incised stream courses preclude development of irrigation, and so rain-fed cultivation has to be the prevalent land use. However, due

1. Assistant Soil Survey Research Officers, West Pakistan Directorate of Soil Survey, Central Soil Research Institute (Soil Survey Project of Pakistan), Lahore.

to the low and irregular rainfall it is generally not possible to utilize the full potential of this land. The rainfall becomes progressively more limiting to the south-west, so that the variety of crops decreases as well as the proportion of land under crops. Some soils of the drier south-western part could better be removed from cultivation and developed as rangeland. Most of the eroded and rocky areas also have a potential for grazing or for forestry.

A generalized map of development potential in the area is affixed. The development possibilities of each mapping unit are described below:

1. Land with a very high potential under irrigation.

2. Land with a high potential under irrigation.

3. Land with a moderate potential under irrigation.

4. Land with a moderate potential under rain-fed cultivation.

5. A complex of land with a moderate potential for rain-fed cultivation and land with a low potential for grazing.

6. Land with a low potential for dry-farming.

7. A complex of land with a low potential for dry-farming, land with a low potential for grazing and agriculturally unproductive land.

8. Land with a fair potential for grazing or forestry.

9. Land with a low potential for grazing or woodland.

10. Agriculturally unproductive land.

Mapping unit descriptions

1. *Land with a very high potential under irrigation* covers very small, narrow belts of land adjacent to main streams. At present about half of this land is irrigated by persian wheels, whereas the other half is rain-fed. The soils of these areas have no limitations for agriculture and production could be increased very considerably if all the land were irrigated and modern management practices introduced. Sufficient quantities of high-quality ground water are expected to be available at accessible depths for tubewell irrigation. These areas are very well suited for vegetables and other high-value crops.

2. *Land with a high potential under irrigation* occupies small areas in the river plains. This land is rather slowly permeable due to the moderately fine to fine textured soils, so that it is intermittently saturated during the rainy season. With sufficient irrigation supplies, part from the small locally managed canals and part from tubewells, proper water management to avoid over-irrigation and provision of local drains for disposal of excess water, this land could be used for intensive cropping of vegetables, tobacco and orchards.

3. *Land with a moderate potential under irrigation* comprises the plains areas near Abbottabad and Mansehra towns (locally called Rash and Pakhli plains respectively). In the Rash plain, crop production suffers from somewhat poor drainage, the water-table fluctuating between 30 and 75 cm (1 to 2½ feet) depths. With provision of surface drains and, if necessary, tubewells to evacuate excess water, and of interceptor drains to keep off run-off from the surrounding higher areas, this land could produce a wide range of summer crops and a few winter crops. Alternatively, this land could give high yields of leafy vegetables without expenditure on drainage. In the Pakhli plain the central major part is irrigated and the remainder

* Reconnaissance soil survey available

rain-fed to produce one good to very good summer crop per year. Small areas are sown to wheat, but due to very low winter temperature yields are poor. With irrigation relatively little increase in production or in range of crops is obtained because of the hazardous, limited winter cropping. Under both rain-fed and irrigated conditions, this land generally produces one high-yielding crop of maize in almost all years. With irrigation, however, this land could also be used for an early summer crop, e.g. vegetables, or for rice or fruit orchards, for example, plums, apricots and pears.

4. *Land with a moderate potential under rain-fed cultivation* occupies large areas in the subhumid north-eastern Potwar uplands. Under traditional management these areas produce two moderate rain-fed crops per two years on the average (under a system of one year double cropping and one year fallow). Most of the land is loamy or silty and has limitations mainly due to in sufficient natural moisture. The moisture becomes increasingly more limiting from north-east to south-west. With emphasis on summer crops like groundnuts and application of adequate fertilizers this land could produce one good to very good crop per year. Part of the land is sloping or gently sloping and therefore susceptible to erosion by water. Production from such land also could be increased by emphasis on summer crops, especially groundnuts, and by soil conservation measures such as improved levelling and embankment of fields and provision of structures for disposal of run-off. In a small portion of this land the choice of crops is limited by the clayey nature of soils besides insufficient natural moisture. Higher benefits from this land could be obtained by introduction of drought-resistant varieties of crops like sorghum and pulses and by application of fertilizers.

5. *A complex of land with a moderate potential for rain-fed cultivation and land with a low potential for grazing* covers considerable parts of the north-eastern half of the uplands. This land is severely affected by past geologic erosion. (Present erosion mainly attacks areas that are terraced and cultivated after removal of natural vegetation.) About a quarter of this area is gullied land unfit for cultivation, supporting sparse vegetation used for grazing. Three quarters is terraced, markedly sloping, erodible land used for cultivation. As most of the precipitation is lost as flash run-off and very little of it is absorbed into the soil for use of crops, the yields are not as high as on the adjacent high level areas. The yields could be improved by emphasis on summer crops, especially groundnuts, and soil conservation measures such as improved levelling and embankment of fields, provision of grassed or paved structures for disposal of run-off, and controlled grazing. Investigations are suggested into the economies of reseeding and phosphate fertilizing of gullied land.

6. *Land with a low potential for dry-farming* occupies most of the drier south-western half of the Potwar uplands. Some of the soils of the area are loamy or somewhat sandy, some are clayey and some are gravelly or have sandy topsoils. The low and erratic rainfall of a semi-arid climate is the dominant limitation of the loamy or somewhat sandy soils. The use and management of clayey soils is further adversely affected by difficult land preparation and high non-beneficial evaporation, and of sandy and gravelly soils by low water-holding capacity and rapid permeability. This class of land also includes small patches of gravelly soils occurring in

a high subhumid climate. Under traditional management all these soils produce one poor (marginal) crop per year on the average. Response to modern management is expected to be low. The economies of introducing improvements like drought-resistant crops and phosphate fertilizers on this land will have to be carefully investigated before undertaking any large-scale activity. Only soils that are reasonably level and at least moderately deep over impervious strata, and those with favourable textures for dry-farming should be cultivated. The others could better be removed from arable cropping and developed as rangeland.

7. *A complex of land with a low potential for dry-farming land with a low potential for grazing and agriculturally unproductive land* is fairly extensive in the semiarid southwestern zone. About two thirds of this land is dry-farmed. Inadequate and erratic supply of moisture is the overriding limitation for sustained agricultural production. About one-third is severely eroded (gullied) land unfit for cultivation. Some of the gullied areas support a sparse grass vegetation which provides light grazing; the remainder is so steep that it does not bear any vegetation and is unproductive. Traditionally, the terraced cultivated areas produce one poor (marginal) crop per year, but crop failures are common. Irrigation is not feasible and response to modern management including soil conservation measures is expected to be low. Although it would be technically possible to level a part of the gullied land, this would be uneconomic in view of the low potential of both the resulting and adjacent land, and the increased erosion due to the levelling would add to the silt load in the rivers. The severely eroded land is best left in its natural state.

8. *Land with a fair potential for grazing or forestry* occurs in several small patches. Some of them comprise sandy soils occurring along active stream courses and thereby receiving extra moisture, and some are steep eroded areas occurring in a high subhumid climate. These areas are capable of yielding high benefits from planted forest or by reseeding with palatable grass species and controlled grazing or cutting.

9. *Land with a low potential for grazing or woodland* is quite extensive. This land, most of which is severely dissected by erosion, is silty, loamy, stony or rocky. It supports a sparse cover of scrub providing rough grazing and some fuel. Although production from these areas could be increased by fertilizers, this would probably be uneconomic.

10. *Agriculturally unproductive land* occupies small patches throughout the Potwar uplands. These are either so steeply dissected or so rocky that there is almost no vegetation.

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Causes of Earth Dam Failures—Some Case Histories

By **SHAFAAT AHMAD QURESHI**
Assistant Director Review Cell, Lahore.

بِسْمِ اللّٰهِ الرَّحْمٰنِ الرَّحِيْمِ

لَقَدْ كَانَ لِسَبَاٍ فِيْ مَسْكَنِهِمْ اٰيَةٌ ۚ جَنَّتٰنِ عَنْ يَّمِيْنٍ وَّ شِمَالٍ ۚ كُلُوْا مِنْ رِّزْقِ رَبِّكُمْ وَ
اشْكُرُوْا لَهٗ ۚ بَلْدَةٌ طَيِّبَةٌ وَّ رَبٌّ غَفُوْرٌ ۝ فَاَعْرَضُوْا فَاَرْسَلْنَا عَلَيْهِمْ سَيْلَ الْعَرِمِ
وَ بَدَّلْنٰهُمْ بِجَنَّتَيْهِمْ جَنَّتَيْنِ ذَوَاتَيْ اُكُلٍ خَمْطٍ وَّ اَثْلٍ وَّ شَيْءٍ مِّنْ سِدْرٍ قَلِيْلٍ ۝
ذٰلِكَ جَزَيْنٰهُمْ بِمَا كَفَرُوْا ۚ وَ هَلْ نُجٰزِيْٓ اِلَّا الْكَفُوْرَ ۝ (34 : 15-17)

The above Ayats from the Holy Quran tell us of the flourishing community of Saba and the God's wrath that descended on them in the form of great flood caused by the dam bursts. The following is the English translation of the above:—

(15) There was indeed a sign for Sheba in their dwelling-place: Two gardens on the right hand and the left (as who should say): Eat of the provision of your Lord and render thanks to Him. A fair land and an indulgent Lord:

(16) But they were froward, so We sent on them the flood of 'Iram, and in exchange for their two gardens gave them two gardens bearing bitter fruit, the tamarisk and here and there a lote-tree.

(17) This We awarded them because of their ingratitude. Punish We ever any save the ingrates?

According to Arabic legend, information supplied by Roman history and details gathered from 3,000 plaques & tablets uncovered recently as a result of archeological diggings, rise of the great nation Saba or Sheba (as someone may like to call it) was mainly due to trade and irrigated agriculture. Their system of Irrigation was of such a high order that none equalled it except that of Mesopotamia at that time. In the absence of perennial rivers, as we have here, canals would offtake from artificial tanks/ponds created by construction of earthen embankments or small masonry dams across seasonal nullahs and rivulets which would rise during rainy season. One, but the greatest, source of such a system was the tank behind the dam called Sad-i-Mairab, constructed at Mairab the then capital of Saba—60 miles from the present city of Sana. The dam is believed to have been started and constructed during the reign of Queen Sheba. Other emperors who followed, strengthened and enlarged it. But when God's wrath came the dam failed in the 5th century A.D. (around 450-451 A.D.) due to undermining of the foundation by rat-holes and other animal burrows. The destruction of the dam and the ensuing wave of flood is supposed to have been responsible for overtopping a series of such dams resulting in complete chaos.

In fact dam construction is not a new affair and dams have been constructed since earliest times for flood protection and Irrigation purposes e.g., Egypt claims to have had the world's oldest dam (355 ft. long and 40 ft. high) built about 5,000 years ago. In China too a dam constructed in about 2,000 years B.C. still stands and is in use even today. Construction of earthen embankments for conservation of water for drinking purposes, as flood protection measure and as diversion and storage work for Irrigation purposes is thus regarded as man's instinct for survival. The design and construction of these dams in old days and even till quite recently has however been very much more of an art than a science and many dams thus constructed, though have stood the test of time, yet there are many more which failed one way or the other either during construction or gave trouble later during operation. The knowledge gained from the failures or unsatisfactory performance of these dams has been very rewarding and has been responsible for evolving most of the procedures which constitute now the modern practice for design and construction of earth dams. The development of soil mechanics in the last 40 years or so has placed a powerful tool in the hands of engineer and many rules of thumb, earlier developed for the design of earth dams have now been thrown away. Because of the confidence gained due to this advancement in soil mechanics and hydrology some of the earth dams have recently been constructed at sites and with heights which would have been thought dangerous or impossible only a few years earlier. For example, concept of the famous Aswan Dam is thought to have been first given by Al-Hazan (965-1038), a great Muslim Physicist, Mathematician and an Astronomer who had suggested to the Caliph to control the Nile at Aswan. He could not, however, implement the scheme and ran to Syria.

In spite of all the strides and advancements made in the design and construction of these dams, though the number of failures have considerably reduced yet there have been recently one or two such failures in France and Italy which remind us of God's superiority and confirms our belief as Muslims that these and such other disasters are the signs of God's displeasure. However out of the causes listed for failure of earth dam the most common cause of complete catastrophic failure has been that of water overtopping the earth dams due to insufficient spillway capacity. The two other principal causes of catastrophic failure are piping i.e. progressive errosion of leaks which develop under or through the earth dam and earth slides in the downstream portion of embankment or foundation.

The above factors together with those which have caused great maintenance problem are discussed below:

I. Overtopping

Many failures of earth dams have been caused by improperly designed spillways or by spillways of insufficient capacity. However, failures due to overtopping are not considered deficiencies in the design of earth dam itself but rather the result of inadequate hydraulic design. After assessing the maximum discharge anticipated flood routing is carefully done such that the net-free board—the vertical distance from the reservoir surface to the top of dam at the time that the spillway is discharging the greatest flood to be expected—should be the sum of the heights of tides, seiches, wind set up and the height to which waves will ride on the upstream face plus a margin of safety based on judgement.

II. Piping

Piping is a problem related to seepage—and in fact to uncontrolled seepage. As we know, depending upon the character of the material comprising the foundation and embankment, which has a very important influence, the seepage takes place through and under all dams, both earth and concrete. The problem is therefore to minimize and control it, so that it will have no harmful effects (control will include control on the position of seepage line as well as the pressure at exit). As water seeps through a compacted soil of an embankment or natural soil of foundation, the pressure head is dissipated in overcoming the viscous drag forces which resist the flow through the small soil pores and thus the seeping water generates erosive forces which tend to pull the soil particles with it. If the resisting forces which depend upon the cohesion, the inter-locking effect and the weight of the soil particles, as well as on the action of downstream filter if any, are less than those which tend to cause it, the soil particles are washed away and piping commences. Piping is thus a progressive erosion of concentrated leaks and commences when seepage water issues from an embankment or ground surface under sufficient pressure and with sufficient velocity so that the particles comprising the material are carried away.

In some cases when first observed, the leaks which have led to piping first appeared as a small seep, which to the naked eye, ran clear for years and then increased gradually until rapid failure occurred. Piping is of two kinds:

(A) Vertical Piping,

(B) Horizontal Piping.

(A) Vertical Piping

When the natural surface layer of the foundation is sandy and the upward flow of seepage water is strong enough to carry sand particles with it, it is common for the sand to be deposited around the springs emerging on the ground surface in the form of a ring. Sand boils, if unobserved and unattended can lead to complete failure by piping. Measurement of foundation pore pressure along Mississipi river dykes where large underseepage and boils developed indicate that while the conditions which require the formation of boils differ considerably from site to site, boils began to show in fine cohesionless soils when the escape gradient exceeded 0.5 to 0.8. In some of the Philip's experiment, however, though boils began to form with this seepage gradient of 0.6 to 0.8 but actual piping with a final blow out failure did not occur until the gradient exceeded 1.5 (there was, however, one exception to this where the material was fine sand, complete failure occurred at gradient of 1.04). Vertical piping taking place, to save the dam it is necessary to quickly ring the boils with the sand bags, putting a low auxiliary dam or adopt other means so that some back pressure on the boils is exerted.

In case where an earth dam is proposed on the foundation of alluvial deposits of pervious sand and gravel with an impervious strata (rock, clay etc.) not very deep, it would be advisable to provide a cut-off extending into the impervious stratum in the foundation. The cut-off can be an extension of the impervious core or a concrete well/trench etc. jetting into the impervious strata. Sometimes instead of using a cut-off an impervious upstream blanket is considered preferable if the depths/cost of cut-off is found excessive.

(B) Horizontal Piping

With horizontal piping the water sometimes gushes from the downstream face of an earth dam and erodes it below the point of egress. The horizontal piping soon results in the formation of a small tunnel running back into the dam with the roof continually falling in and being carried away by the piping water.

CAUSES OF HORIZONTAL PIPING

(a) Damaged outlet pipes

Sometimes cause of embankment leaks leading to piping has been cracking in the outlet pipes which is caused by foundation settlement, spreading of the base of the dam, or deterioration of pipe itself.

(b) Absence of filters in zoned dam

Piping can also commence in a zoned dam where the seeping water discharges from the finer material of the core into coarser adjoining pervious zones. Failure of 62 ft. high Sheffield dam, provides a good illustration of the trouble that can arise in such constructions. This dam, completed in 1926, when filled for the first time in 1927 was observed to have cracks with a maximum of $1\frac{1}{2}''$ opening on the upstream slope. In the spring of 1928 when the reservoir reached a new high level, large quantities of the upstream earth section were washed into the voids of the downstream rock zone. Complete failure was however averted by an emergency effort round the clock. In this dam no filters were placed between the earth and rock fill section. It is therefore recommended that relative gradation of adjacent soil zones must meet the established filter criteria; and where the difference in coarseness between the fine and coarse embankment zones is too great to meet filter criteria zones of intermediate gradation must be provided. The following rules, which, though conservative, are widely used concerning quantative criteria for satisfactory filters:

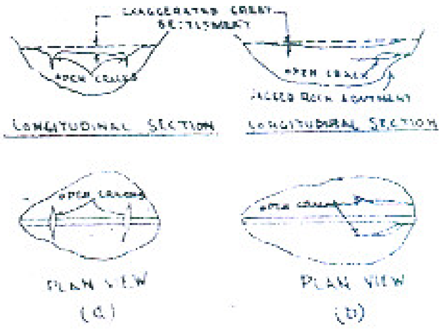
1. The 15% size of the filter (i.e. the particle size which is coarser than the finest 15% of the soil, D_{15}) should be at least five times as large as the D_{15} size of the soil being protected by the filter.

2. The D_{15} size of the filter should not be larger than five times the D_{85} size of the protected soil.
3. The gradation curve of the filter should have roughly the same shape as the gradation curve of the protected soil.
4. Where the protected soil contains a large percentage of gravels, the filter should be designed on the basis of the gradation curve of the portion of the material which is finer than the 1 inch sieve.
5. Filters should not contain more than about 5% of fines passing the No. 200 sieve, and the fines should be cohesionless.

(c) Differential Settlement Cracks.

While the danger of embankment cracking has not been widely publicised, it is understood that a large group of failures which occurred when the reservoirs were filled for the first time were wrongly attributed to piping leaks along the outlet conduits or at the abutment or foundation contact. They were in fact due to piping developed through embankment cracks.

Cracking develops because portions of embankment are subjected to tensile strain where the dam is deformed by differential settlement. There may be different cracking patterns depending upon the geometry and relative compressibility of the foundations, abutments and embankments etc. Evidently, transverse cracks creating a path for concentrated seepage through the core are most dangerous. In narrow valleys with rock abutments, arching of the upper portion of the embankment can prevent the crest from settling as much as the foundation and roughly horizontal cracks may open at the bottom of the arch. Cracking of this type was responsible for the failure of the 112 ft. high Apishapa Dam in 1923.



TYPICAL TRANSVERSE DIFFERENTIAL SETTLEMENT CRACKS

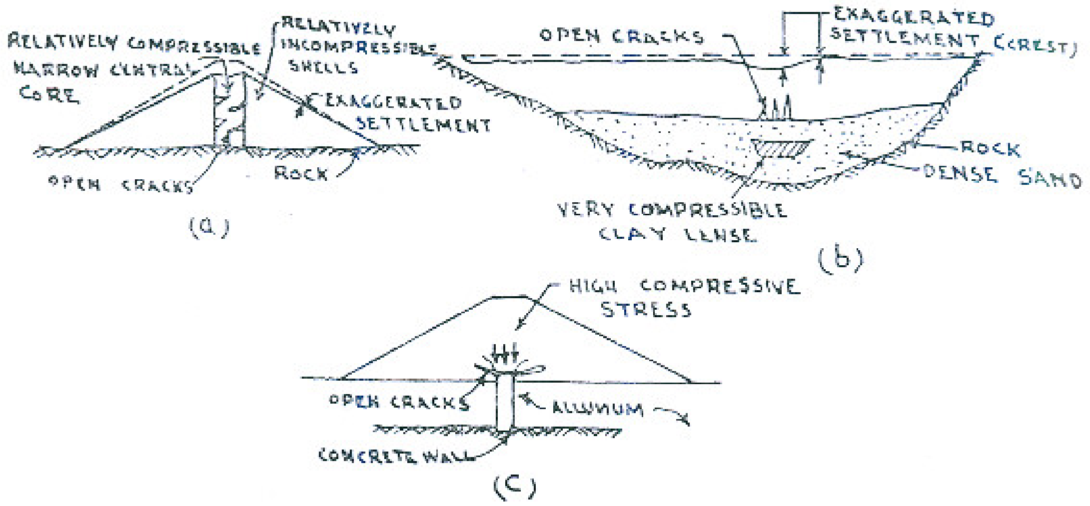
Fig. 1

Another kind of transverse cracking has developed due to differential settlement at a Fensmall dam where section of rolled earth embankment have been placed in trenches through compressible natural soil foundation of the purpose of supporting the outlet conduits.

Longitudinal cracks are not normally dangerous but can occur frequently by several types of differential movement. The main danger associated with longitudinal cracks is that they may occur in conjunction with other unseen cracks running transversely through the core. It is thus always advisable to put down test pits to make sure that the crack is actually vertical and longitudinal and not surface manifestation of an inclined crack cutting through the dam core.

In addition to cracks discussed above there is another category which is never observed on the surface of dam and whose existence can only be inferred. One such type may occur in dams with narrow vertical central cores of compressible impervious material. During construction the core tends to compress more under the weight of the overlying fill than the shells do, so that a part of the weight of the core is transferred to the shells by shear stresses and arching. This arching effect combined with variable shear strength developed on the vertical plan separating the core and shell can conceivably result in horizontal cracks in the core. Cracks may also occur where a relatively short length of embankment is underlain by a more compressible foundation material than existed under the rest of the dam.

more susceptible to cracking when compacted dry than either finer or coarser material. This also shows that clays of higher plasticity which are finer than the gradation range will withstand much high deformation without cracking. The study also provided some evidence for susceptibility to cracking is high in embankments of residual soils containing coarse particles of soft rocks which break down and become appreciably



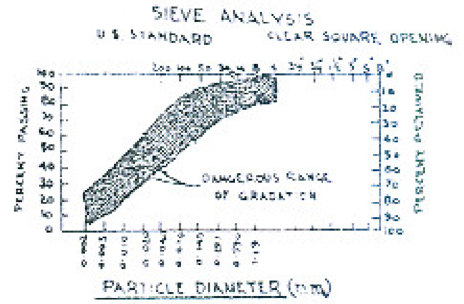
.INTERNAL EMBANKMENT CRACKING.

Fig. 2

Tension may develop in the lower part of the dam and the cracks may open through the core. Localised interval cracks also open in impervious zones adjacent to interior concrete structure because concrete is much less compressible. At Neversuk Dam which had a rigid concrete foundation cut-off wall cracking of this type produced a pervious zone 50 ft. above the foundation in the otherwise very impervious core.

A study of the performance of some dams which cracked indicate that embankments of inorganic clays of low to medium plasticity i.e., P.I. less than 15 with gradation curves falling within the range shown are probably

finer when they are being placed and compacted. Most of the embankment studied which developed sever cracking were com-



RANGE OF GRADATION OF SOILS SUSPECTED TO BE MOST CRITICAL EMBANKMENT MATERIALS FROM STAND POINT OF CRACKING.

Fig. 3

pacted at low construction water contents (usually 5 or more drier than standard or proctor optimum) but dangerous cracks had also opened in embankment which were not compacted excessively drier.

Once horizontal piping has started, the only remedy is to dump rock, grading from fine to coarse, right into the downstream face where the horizontal piping is occurring, so that an improvised drain and filter is formed and piping is stopped. In the original design the possibility of serious piping may be prevented by having the path of percolation sufficiently long in relation to the head, to reduce the hydraulic gradient; by providing properly designed and constructed filters and drains and by using correct type of construction materials and compacting the same at moisture content slightly higher than optimum.

III. Sloughing

The position of the line of seepage in the cross-section of an earth dam is also important. If this line is allowed to intercept the outside downstream face much above the toe, serious SLOUGHING may take place and ultimate failure may result. In fact progressive sloughing is a type of damage closely related to piping. The process begins when a small amount of material at the downstream toe erodes and produces a small slump leaving a relatively steep face which when saturated by seepage slumps again. The failure of Sinker Creek Dam provides a good illustration of the mechanics of progressive backward sloughing. This homogeneous dam, composed primarily of silty sandy gravel, was raised to the height of 70 ft. by 1918. During 25 years of its operation large sections of its downstream slope used to be saturated whenever reservoir was full for any length of time. Although to overcome this condition the period of maximum storage was kept as short as possible but the state of dam was not considered critical. In 1943, however, the reservoir was filled several months earlier than usual resulting in large areas on downstream slope saturated more than in previous years. Thus on the evening of June 19 a small amount of downstream toe softened in the action of some small concentrated leaks. This progressive sloughing continued until in about 8 hours time the whole reservoir broke through to form one large flood wave downstream.

IV. Embankment and Foundation Slides

Slides occur when average stress along any potential sliding surface becomes greater than the average strength. Slides can be grouped into three categories :

1. Slides during construction involving the upstream or downstream slope, or both.
2. Slides on the downstream slope during reservoir operation.
3. Slides on the upstream slopes after reservoir drawdown.

Slides during construction

Relatively fewer slides have occurred on rolled earth dam during construction; however, in the past a number of failures took place in hydraulic fill dams—the best known is that of Fort Peck Dam in Montana where in 1938 about 5×10^6 cu. yards of earth slid rapidly killing 8 men. The failure of hydraulic filled dams is supposed to be due to their method of construction which generally results in embankments of very loosely compacted saturated sands that liquefy. Even otherwise in every case of construction slide the dam is found to be underlain by foundation of either soft, brittle or sensitive clay and a large portion of the sliding surface passed through the foundation.

Depending upon the properties of foundation clay two types of slides viz. slow or rapid slides can take place. Slow slides occur when the foundation is more or less homogeneous deposits of soft clay which is not sensitive. Rapid slides result when the foundation clay contains horizontal bedding planes of coarse silt or fine sand.

A typical slow slide occurred at North Ridge Dam in Canada in September 1953 when the dam was nearly complete. The first indication of the slide were the appearance of cracks (which opened to a maximum of about 6 in. at the top, were 11 ft. deep along the upstream slope and 22 ft. deep along the downstream berm) and a slight bulging of the fill. The ground surface also heaved for a distance of 60 ft. beyond the downstream toe and moved outward by about 3 ft. So fill construction was immediately stopped and berms were added at both toes. The dam was thus successfully completed in 1956 to its full height.

The failure of Marshal Creek Dam in Kansas which occurred in 1937 when the embankment was within 10 ft. of the crest, is a typical example of a rapid slide. In rapid type of slides the dams can be reconstructed to the originally planned height without a long wait for foundation consolidation.

Downstream slope slides during reservoir operation

There are two distinct types of slides which have occurred on downstream slopes during reservoir operation.

(*a*) Deep slides, which generally pass through the clay foundation and which nearly always take place during full or almost full reservoir.

(*b*) Shallow slides.

A typical deep downstream slide occurred at the Fruit Growers Dam—a 32′ high dam in W. Colorado, in 1937, when the reservoir was at maximum elevation. The sloping soil buried the downstream end of the outlet conduit making it impossible to lower the reservoir. Complete failure was averted by a cut made through the embankment and the natural abutment where the dam was only 5 ft. high.

A similar experience occurred in 1958 at the 50 ft. high great Western Dam near Denver, Colorado.

Shallow slides, most of which follow heavy rain-storms, do not as a rule extend into the embankment in a direction normal to the slope more than 4 or 5 ft. These rain caused slides are generally due to accumulation of surface puddles in the poorly drained downstream berms or roads which saturate the embankment directly below the berm. Shallow surface slips involving only the upper few inches have sometimes occurred when the embankment slopes have been poorly compacted.

Upstream slope slides following reservoir drawdown

Except downstream slides during construction generally reservoir draw down conditions are dangerous. Since the slide comes to equilibrium at a stage of low reservoir, there is small likelihood of catastrophic failure even though a large earth movement may have taken place. A study of upstream slides in 12 dams indicates that the majority was caused by a draw down approximately between maximum water surface and mid-height of the dam at average rates varying between 0.3 and 0.5 ft./day. Most slides developed when the reservoir was lowered for the first time though a few occurred after many years of their successful operation. In every case, however, the slides were caused by a draw down which was either faster or over a greater range than had occurred previously.

Upstream slide at Belle Fourche Dam in Montana is a typical example. The Dam built as a homogeneous embankment of highly plastic clay has a 2 : 1 upstream slope and a maximum height of 115 ft. In 1931, more than 20 years after construction the reservoir was drawn down at an unprecedented rate and a shallow layer of soil slid down the upstream slope. The slide had a depth of about 10 ft. and difference between the top and bottom edges of the slide was approximately 45 ft.

Influence of Soil type on Slides

Almost all slides during construction and of upstream and downstream slides after construction have occurred in dams underlain by foundation of clay, relatively high in plasticity and natural water content. In a study of 65 old homogeneous dams in the western United States it has been found that embankments constructed of clay having an average grain size (D_{50}) finer than 0.006 mm have more chances of sliding failure. For embankments constructed with grain sizes ranging between 0.02 mm to 0.006 mm. chances were reduced to a half. None of the dams constructed of soils with an average grain size coarser than 0.06 mm. failed by sliding even though many had steep slopes and were poorly compacted.

V. Reservoir Wave Action and Upstream Slope Protection

The upstream slope of earth dam must be protected against destructive wave action. The erosive wave action, which causes most of the trouble at earth dam occurs during unusually severe storms. Damage from wave action has not caused a serious threat of complete failure except in rare cases; although it has usually necessitated costly repairs.

The near failure of Point-of-Rocks Dam in N.E. Colorado illustrate the worst trouble that has occurred on a few poorly constructed dams with completely inadequate free board or excessive steep upstream slopes. The dam, constructed in 1911 with a maximum height of 90, is composed of homogeneous clay. Original protection consisted of a 4″ thick concrete slab reinforced by wire fencing laid over upstream bank slope of 1.5 : 1. Since the dam is located in a flat part of the great plain of Colorado where winds produce high waves, the steep upstream slope and concrete facing proved unsatisfactory from the very beginning. In 1915 the waves cracked the slab and eroded the embankment to a depth of 2-3 ft. over long distances. A layer of riprap with a thickness of 2 ft. at crest and 5 ft. at toe was finally placed over the concrete slab. However, this too failed and the slope proved too steep for the riprap repair to stay permanently.

On May 8, 1927 when the reservoir was 17 ft. below the top, a 2-day wind-storm of hurricane proportion began. The crest was nearly eroded through and a disastrous failure was averted only by the constant effort of about 1000 men working day and night and bagging at the high water line.

The upstream slope of most earth dam have been protected with one of the following materials (in decreasing order of frequency):—

(*i*) Dumped rock riprap.

(*ii*) Hand placed rock riprap.

(*iii*) Articulated concrete pavements consisting of individual slabs.

(*iv*) Monolithic R.C.C. pavements.

Slope protection of articulated concerete paving has been least successful of all because wave water could wash large quantities of filter or embankment material through cracks between individual slabs. The experience at Belle Fourche dam, Miniature dam, Kingsley dam & Harold dam illustrate this type of damage. Belle Fourche dam, for example, which was completed in 1908 with a maximum height of 115 ft. and a length of 1 mile is known to have a homogeneous section consisting completely of highly plastic clay, well constructed in thin layers. The upstream slope is 2 to 1 and the slope protection is composed of 8-in. thick precast concrete slabs, 6×5 ft. in plan, underlain by 1 to 2 ft. of gravel filter. The individual slabs are not connected in any way, and a space of ½ in. was allowed between them for drainage. Wave action between 1912 and 1915 caused minor damage to the upstream face, displacing some of the slabs each year. In 1916 a serious break brought about the dislocation of more than 350 of the slabs and made extensive repairs necessary. In 1922 a similar large displacement occurred and was repaired. This problem still continues, with damage to the upstream slab following every major storm. Jets of water have been seen sprouting from the joints between the slabs as waves pass and in one case an individual concrete slab was observed to shoot out about 10 ft. from dam face.

Monolithic R.C.C. slabs have so far been very successful. The only disadvantage with this type is that wave ran up the smooth slope further than they do on rough rock surface. Steps used as wave breakers have been spectacularly unsuccessful. Instead curved wave walls at the tops of the slopes have been very effectve in turning back the wave water and keeping the crest dry. Other types of protection that have been used are steel facing, soil cement and bituminous pavements, and (on small and relatively unimportant structure) willow mattress and saked concrete.

VI. Downstream slope protection

If the downstream slope of an embankment consists of rock or cobble fill no special surface treatment is necessary. Downstream slope of homogeneous dam or dam with outer sand and gravel zones should be protected against erosion by wind and rainfall by a layer of rock or cobbles. If grasses are planted, those suitable for a given locality should be selected.

VII. Damage due to burrowing animals

Burrowing animals have been responsible for piping failure in a number of small earth dams and dikes as also for Mairab Dam described before in the beginning of this paper, but have not caused trouble in major dams because animal holes do not penetrate to great depth. Muskrats, ground squirrels and porcupins are the worst pests.

Muskrats burrow into embankment to make holes to dig passages from one pond to another. The entrance to muskrat hole is usually under the water surface and it digs only far enough to get under the upstream slope. When many muskrats are involved, dangerous honey combing of a small earth dam may result.

Ground squirrels and porcupines normally dig in dry soil to get close to cold wet soil near the upstream face and this may result in serious piping sometimes. All earth dams in regions with many muskrats should be carefully inspected at frequent intervals. Trapping and poisoning may keep the burrowing under control.

VIII. Damage due to water soluble materials

The leaching of natural deposits of water-soluble materials from abutments and foundations has caused difficulty at some dams. Gypsum which is gradually dissolved by seepage water from the reservoir has been particularly troublesome in this respect. At the Dry Canyon Dam in Southern California, which was constructed to 55 ft. in 1912, leakage through the abutments and foundations gradually increased. Laboratory tests performed in 1933 to analyse the condition, showed that approximately 5 cu. ft. of solid material was being removed per day, and that most of it was gypsum. By 1935 the rate of seepage had approximately doubled, and it was estimated that the annual loss of solids from the abutments varied between 40 and 100 $yd.^3$ per year. The leakage continued to increase until extensive cement grouting was carried out in the abutments during 1936 & 1937.

IX. Flow slides due to spontaneous liquefaction

One of the most difficult problems facing the earth dam designer is the analysis of the stability of loose sand foundation against the possibility of liquefaction or flow slides. At the present state of our knowledge, however, neither theory nor experience offers much reliable assistance for the evaluation of the susceptibility to possible liquefaction.

It is well known that "flow" or liquefaction slides have occurred in slopes of natural soil consisting of deposits of fine sand and silt and in some types of clays. The slides are caused when shear strains imposed on the saturated material result in a tendency for volume decrease and consequent transfer of stresses within the soil mass from the grain structure to the pore water. Then large part of the weight of overlying soil is carried momentarily by the pore water, the shear strength of the mass is literally reduced to that of liquid, hence the term flow or liquefaction slide.

Flow slides in natural soil deposits have been triggered by Earth-quake shocks and by under cutting of the toe of the slope. The failure of Sheffield dam in Santa Barbara in 1925 following an earthquake shock probably was caused by liquifaction of the portion of the dam or foundation. In this case the records are vague about the manner of construction and the construction material, but it is known that the embankment was poorly compacted and that the material was basically granular and fully saturated.

In several ways this failure was unique. First, because the whole central portion of the dam moved in a more or less intact condition about 100 ft. downstream. Second, because the concrete slab facing remained intact in the lower portion of the embankment across the full length of the dam.

X. Damage caused by downstream deflection in rockfill dam with central cores

Rockfill dams with steep slopes, resting on rock foundation and having thin central cores of earth or concrete do not develop slope slides as may occur in earth fill dam. When the reservoir is filled for the first time the dam may however start deflecting downsteam if the slope of bank downstream is too steep. This movement may continue gradually with a creep like action. No theoretical basis is available for the analysis of this phenomenon.

The record of Crane Valley Dam constructed in 1910 in Madera County, California provides an example of excessive and long-time movement.

In 1914 measuring points were established on the top of core wall. These indicated a movement of about 5 ft. between 1914 & 1952. It is estimated that before measurements were begun already about 9 ft. movements had taken place. To retard this movement large qualities of rock materials were dumped in the downstream slope by dumping from crest, such that downstream slope is 1.3 to 1.4 the crest width is presently about 100 ft.

In 1928, a vertical exploratory shaft was sunk at the downstream edge of the concrete core wall in the rockfill section near the centre of the valley. It was found that the bottom of the wall, which had been keyed into the granite bedrock of the foundation, had not moved. At a height of about 6 ft. above the bedrock, however, there was a horizontal crack and a change in the slope of the wall; though the wall was essentially intact above the crack (at least at the location of the test pit) and that most of the crest deflection resulted from rotation at the crack. Seepage was, however, prevented by fine soil from upstream hydraulic fill section which sealed the crack.

Similar excessive downstream movement occurred in 1929 during the first filling of the reservoir behind 115 ft. Oued Kebir Dam (Tunis), where the crest settled 2 ft. and moved 2′—8″ downstream. The bottom of the concrete core wall sheared off and moved 5 inches downstream.

XI. Damage due to surface drying

Surface drying cracks which usually develop near the top of dam parallel to crest have caused a constant maintenance problem on a few dams constructed with homogeneous section of clayey soil. Excessive drying cracks frequently develop on low clay dams constructed for flood control purposes; where sometimes a designer may in view of short periods of water retention and that too at the frequent intervals, specifies less embankment compaction than normally required. Surface erosion concentrates in the embankment cracks which develop leading to an awkward situation requiring in many cases refinishing of slopes and crests completely and in worst cases almost reconstruction of the whole dam.

If the construction surface of an embankment of fine-grained soil is allowed to dry in the sun, drying cracks can greatly increase the overall permeability of the material. This has happened even on dams constructed in accordance with good modern practice. The Bureau of Reclamations' Lovewell Dam in Nebraska illustrates the kind of trouble which can arise. After the dam was finished, an exploratory hole was drilled through the embankment near the downstream edge of the crest for the purpose of testing the strength of the soft foundation clay. The engineers were surprised to find that a considerable amount of drilling water was lost through the hole into the impervious embankment at a level about 5 ft. above the foundation. Water was poured in at the rate of 100 gal./min. and completely disappeared.

Because of this episode, specifications for the USBR's Twin Buttes Dam, which was to be constructed of similar material, required that the contractor protect completed portions of the embankment against drying out by sprinkling or covering with loose earth.

Earthquake Damage

The information available concerning the performance of earth dams during earthquake is very meager, however, observation on similar structures such as highway and railroad embankments etc. indicate clearly that loose uncompacted earth embankments subjected to severe earthquake shocks are literally shaken to pieces. They settle as much as 50% of their height, spread at the base and develop large cracks in all directions. In fact earth dams are not absolutely rigid and when excited into oscillations by strong earthquakes they respond in a manner, dictated by the nature of the ground movement and geometry of the structures (the period and amplitude of the ground movement are also important). Ground acceleration, may induce acceleration in the body of the dam with associated fluctuating stress. These stresses may alter the strength properties of the fill or become momentarily large enough to cause failure. The ground movements which are usually recorded in three orthogonal directions in terms of acceleration, may be considered to be composed of a sequence of individual pulses. Each of these pulses will induce a distortion which travels up the dam, whose combined effect is to produce a particular response in the dam. To find this response we need to know not only the properties of the structure but also the complete history of the ground accelerations. It is obvious therefore that unless there is sufficient information how the ground moves during a strong earthquake, it is not possible to formulate and to solve the problem of finding the distribution and time history of the accelerations that arise in a dam. However, there is some evidence that dams may be worse affected by ground movements with longer periods than these which cause the most harm to buildings. (Note that the ground movements which cause the most damage to building structures have dominant periods ranging between 0.1 and 1.0 seconds. In severe shocks, the ground has usually been shaken between 30 and 90 seconds with near maximum intensity). Similarly it has been observed, with few exceptions of course, that structures founded on solid hard rock are less damaged than similar structures at a given distance from epicentre founded on soil. Experience also indicates that upper portion of the embankment is subjected to greater forces than the lower and the most common damage due to foundation shaking is the development of nearly vertical longitudinal cracks on or near the crest.

Given below are some case histories which are more instructive and better documental as regards earthquake intensity and damage caused by them :—

San Francisco Earthquake 1906

This earthquake was one of the most severe on record in U.S.A. The intensity of the shock in the area of San Andreas dam, Pilarcitos dam and Upper crystal dam was at least 10 on modified Mercalli scale.

The San Andreas and Upper Crystal Springs Dams were constructed in the main valley of the San Andreas Fault. During the earthquake this Fault had a horizontal displacement as large as 21 feet at some points. The 90 ft. high dam originally built in the 1860's and subsequently raised in 1875, was constructed of well-graded gravelly, clayey sand compacted in thin layers with a 3-ton roller. It has conservative slopes of 3½ to 1 upstream and 3 to 1 downstream and a relatively narrow central core of wet puddled clay.

The dam was evidently subjected to an extremely severe shock. The main fault, though it did not cross the foundation, passed through the east abutment directly adjacent to the embankment. The concrete-lined outlet tunnel in the abutment was twisted 10 ft. out of line. The natural ground around the dam was everywhere crisscrossed with cracks. A single longitudinal crack 2 to 3 in. wide opened parallel to the axis of the dam near the center of the crest. Even though the reservoir was almost full at the time of the shock, no leakage or oter alarming symptoms developed; as for example at the Hebgen dam where waves of such a magnitude had developed that they overtopped the crest of the dam.

Upper Crystal Springs Dam.—The Upper Crystal Springs Dam was built in 1887 with approximately the same construction method and design section as San Andreas Dam. It is probably the only dam of any size that has actually been displaced by an earthquake fault which passed through the reservoir, intersecting the dam at a point where the crest was about 20 ft. above the bedrock foundation. The crest was offset approximately 8 ft. during the earthquake, and both longitudinal and transverse cracks developed on it. Some longitudinal cracks with a maximum width of 6 in. had an observed depth of only 4 ft. The transverse cracks were closed by the general compression which appeared to develop in the direction normal to the fault plane in the immediate vicinity of the fault.

Since there was no water head and the dam was being used merely as a highway embankment with outlet conduit through the dam open, the experience provided no information on leakage. The tightness of the reservoir was not influenced noticeably even though the fault ran through the middle of it. Engineers who inspected the dam after the earthquake dô not agree whether it would have failed if it had been retaining a reservoir. Some decided that the core was sufficiently plastic so that no leakage would have occurred at all; while others believed that considerable leakage wôuld have developed through cracks in the clay core and would possibly have led to piping failure.

Pilarcitos Dam.—Built in 1874, the Pilarcitos Dam was 95 ft. high at the time of the earthquake. The upstream slope was 2½ to 1 and the downstream 2 to 1. Like the San Andreas Dam, it had been constructed in thin compacted layers with a puddle core of clay. When it was inspected soon after the earthquake, it showed no cracking or other visible evidences of damage.

The records of the San Andreas and Pilarcitos Dams, which are now approximately 100 years old, constitute two of the few reliable guides at the action of severe earthquake shocks on earth dams. Constructed of clay with methods similar to those used for modern dams, both have performed very well, and no damage has developed in either from earthquakes except for the longitudinal cracking in the center of the crest in San Andreas Dam.

Kwanto Earthquake, 1923—Ono and Lower Murayama Dams

The 120 feet Ono and 80-ft. Lower Murayama Dams near Tokyo, Japan were subjected, on September 1, 1923, to very severe shocks, probably of about the same magnitude as the 1906 shocks near San Francisco. Both the Japanese dams had been compacted in layers and provided with central clay cores.

As a result of the earthquake when the water in the Ono Reservoir was 19 feet below the top of the dam; the crest settled approximately one foot and the embankment developed serious cracks varying in width from 2 to 10 in. on the surface and extending to depths of 35 to 70 ft. Two local slides about 60 ft. long also occurred. No leakage is mentioned in the reports. One Dam was evidently very badly shaken even though it was located 60 miles from the epicenter. The cracks were filled with a clay-sand grout, and the dam continued in use.

At the Lower Murayama Dam, where the water level was 30 ft. below the crest at the time of the earthquake, the reports stated that the crest settled about 8 in., longitudinal cracks opened in the upper portion of the embankment, and the downstream slope bulged about 6 ft. No leakage was mentioned in these reports either, and the dam continued in use.

Santa Barbara Earthquake, 1925: Sheffield Dam

Sheffield Dam, which was built in 1918, is the only dam known to have failed completely as the result of an earthquake. The shock, which moved wooden houses near-by as much as 4 in. on their foundations, was estimated at about 9 on the Rossi-Forel scale. The dam failed early in the morning of June 29, 1925. It is believed that the construction was in relatively thick layers with a minimum of compactive effort, and probably consisting only of the travel of the hauling equipment, and that the material contained cobbles and boulders up to 2 ft. in diameter embedded in fine-grained soil. The upstream slope was faced with a 5-in. thick slab of reinforced concrete, the lower edge of which was extended with a vertical wall down to bedrock as a foundation cutoff.

Before the failure, a considerable quantity of leakage had occurred at the downstream toe for several years, and the lower portion of the downstream slope had been saturated continuously when the reservoir was full. The water level was about 15 ft. below the crest of the 35 ft. dam at the time of earthquake. The dam is supposed to have failed due to sudden liquefaction of the lower portion of the embankment or upper part of foundation.

West Yellowstone, 1959—Hebgen Dam

Hebgen dam on the Madison River near West Yellowstone, Montana had always been a satisfactory structure before the earthquake of 1959. It was constructed in 1914 to a height of 90 ft. as a rolled fill from a gravelly clay of medium plasticity.

The dam has a thick central core of concrete, which is founded on bed-rock over most of its length. The reservoir has a length of about 17 miles.

A relatively severer shock intensity of which was probably at least 10 on modified Mercalli scale occurred at 11 : 35 P.M. on August 17, 1959. One of the main faults passed along the north shore of the reservoir within about 700 ft. of the dam, such that at a point near the dam due to earthquake there was a maximum vertical displacement of the ground surface of 15 to 18 ft. though no appreciable horizontal displacement took place. Due to the earthquake although the dam did not fail, but it was badly shaken.

The earthquake caused :—

(*i*) Large waves on the reservoir which overtopped the dam; at least four times, according to an eye witness. The crest and the downstream slope were surprisingly resistant to erosion.

Much of the vegetation on the downstream slope was swept away, but there were no concentrations of erosion which in any way threatened the dam. The deepest erosion gullies were about 3 ft.

(*ii*) Subsidence of the ground and settlement of slopes such that both upstream and downstream portions of embankment settled over the whole length of the dam with respect to the concrete core wall. The amount decreased almost linearly with the distance from the right abutment, where the maximum settlement downstream from the core wall was about 4 ft. at a point adjacent to the concrete spillway. At this point the dam was lowest but the thickness of alluvium under it was greatest. The settlement at the left abutment was only about 1 ft. The upstream slope slumped more than the downstream, the maximum settlement upstream with respect to the core wall being approximately 6 ft. near the middle of the dam.

(*iii*) Large open cracks in the core wall and the embankment. The core wall which was exposed when the upstream portions slumped, was in perfect condition over most of its length, although four vertical cracks about equally spaced over some 85 ft. developed at the extreme right end where the wall joined the spillway structure. The maximum displacement of these cracks was about 3 in.

Longitudinal cracks, the widest being 12 in. and the average 2 to 3 in. opened in the embankment on the crest downstream from the concrete core wall. These were most severe at the right end of the dam in the immediate vicinity of the spillway, but they extended over the whole crest. The downstream slope was free from cracking except at the right end, where a number of cracks developed. No bulging was visible at the toe. A number of severe longitudinal cracks appeared on the portion of the upstream slope exposed above the high water line. The maximum width of these was about 6 in., and some were several hundred feet long.

Gaps also opened on both sides of the core wall between the concrete and the earth embankment. Near the central portion of the dam the spaces between the embankment and the wall were as much as 6 in. wide.

(*iv*) A large number of leaks developed near the abutment contact at the right end of the dam in the general vicinity of the core wall cracks. Seepage water was clear from the beginning and not plentiful, and it finally stopped when leaks in the badly cracked spillway structure were sealed.

(*v*) A number of land slides around reservoir edge and downstream of it created some problem. From the incidents described above and from other records, the following observations are warranted :—

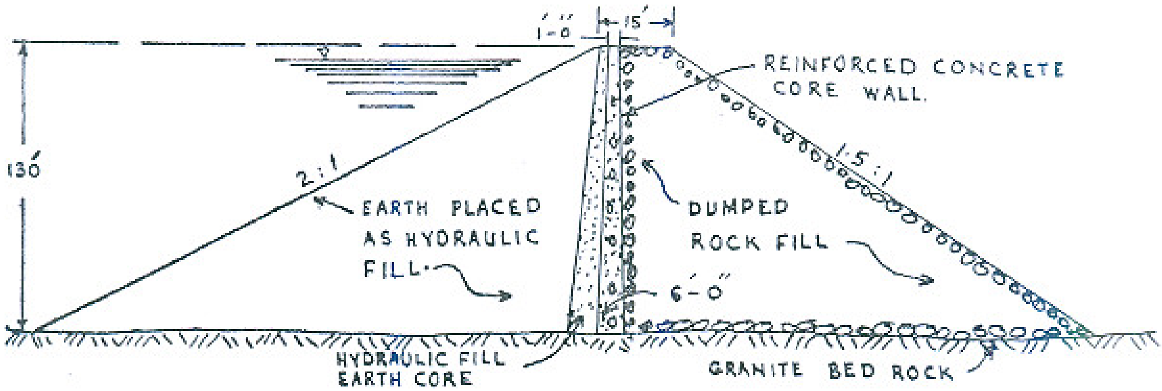
1. In the majority of dams shaken by severe earthquakes, two primary types of damage have occurred : Longitudinal cracks at the top of the embankment, and crest settlement. The crest settlement was not usually great enough to threaten failure by overtopping.

2. Only in the case of the Sheffield Dam did a complete failure occur in a

structure which could properly be called a rolled earth dam. The Sheffield failure was probably due to liquefaction of the very loose and saturated lower portion of the embankment.

3. The damage to the typical dam appears to have been caused primarily by the horizontal component of the earthquake movement in the upstream-downstream direction; that is, the direction transverse to the longitudinal axis of the dam. Probably the amplitude and the acceleration of the horizontal component of the movement of the crest in this direction is much larger than the movement of the foundation, and this results in a "whipping" action of the thinner top of the dam and causes longitudinal cracks.

CROSS SECTION OF CRANE VALLEY DAM AS CONSTRUCTED (1909)



4. Earthquake shocks have caused remarkably few slope slides in earth dams even though some of the embankments must have been very poorly compacted.

5. There is some evidence for the speculation that earth dams are damaged more severely by ground movements with longer periods (lower frequencies) than those which cause greatest damage to building structures. For this reason dams located very near the earthquakes centres may not be as badly cracked as dams many miles distant. For example, the Ono Dam in Japan, which was very badly shaken and damaged in the Kwanto Earthquake, was located approximately 60 miles from the epicenter.

6. There is strong evidence for the conclusion that earth dams with central concrete core walls are more severely cracked by earthquakes than embankments without core walls. The records of both the St. Mary and the Hebgen Dams indicate that the embankment and the concrete wall did not vibrate together when the dam foundation was shaken. From the damage to the Ono and Murayama Dams there is even some indication that cracking may be accentuated by the existence of central clay cores.

So much of the damages: now to sum up I give below a check list which may be consulted to ensure safe design and performance of earth dams :—

1. There is no danger of overtopping.
2. The seepage line is well within the downstream face.
3. The upstream face slope is safe against sudden drawdown.
4. The upstream and downstream slope is flat enough that with the materials utilized in the embankment they will be stable and show satisfactory factor of safety by a recognized method of analysis.
5. The upstream and downstream slopes of the earth dam are flat enough that the shear stress induced in the foundation is much less than the shear strength of the material in the foundation to ensure a suitable factor of safety.
6. There is no opportunity for the free passage of water from the upstream to the downstream face.
7. Water, which passes through and under the dam when it reaches the discharge surface has a pressure and velocity so small that it is incapable of moving the material of which the dam or foundation is composed.
8. The upstream face is properly protected against wave action and downstream face is protected against the action of rain.

For Earthquakes

In addition to above it must be remembered that design of earth dam to increase its safety during earthquakes is one of the most difficult special problems facing the engineer. However, in spite of our present limitations as briefly described before and in view of the experience gained from the behaviour of earth dams during earthquakes there are some very practical things which can be done e.g. :

1. As a first activity, the designer should study the special characteristics of his site, specially from the standpoint of faults, settlement and liquefaction potential.
2. Provide extra free board; although no rules can be given for the extra height that is desirable, it will depend upon the nature of the foundation the expected settlement and size of reservoir.
3. Use graded filters, with more attention to gradation and dimension, in the zone located just downstream from the dam core; and the filter should extend nearly to the top of the dam. In addition downstream zones of large quarried rock should be considered whenever rock is available.
4. Interior concerete core walls should be avoided instead impervious core of the soil most resistant to piping should be selected and a thick core should be used.
5. Increase top width of the dam and use flatter slopes near the top of the dam.

In the end I must once again remind that the above check list or any other endeavours that we make to represent our efforts, as human beings, towards perfection; yet we should not forget that real Creator is God Who is perfect and all-Powerful and all knowledge and strength comes from Him. We should, therefore, pray and seek His benevolence even after doing all in our power.

ACKNOWLEDGEMENT

For the completion of this note I am highly indebted to the authors on whose work I have freely drawn upon, and to Professor Mr. Wahid Ali Khan who kindly helped me by giving useful suggestions and also references for study. I am also grateful to Professor Dr. S. Nazeer Ahmad and M. Yar Sadiq who sustained my interest in the assignment.

Thanks are also due to M/s Sh. Mohammad Ayyub & Mohammad Saleem Akhtar for various diagrams and for typing my poorly scribbled notes.

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Failure of a Syphon and its Reconstruction

By **R. K. ANVER*, P.S.E.I., D.H.E. (Delft) M.I.E. (Pak) M. Am. Soc. C.E.**

A little time past midnight, on the 30th of November 1970, while everybody was sound asleep, the Gauge Reader at R. D. 121 B. S. Link and the people of nearby villages were shaken awake by a thundering sound like that of a subdued bomb explosion. In sheer panic the Gauge Reader came out, looked around and saw a cloud of dust over a nearby Hydraulic structure. By this time he could also hear the noise of rushing water. As the dust settled down, he to his amazement saw a wide gap on right bank of the canal and the huge R. C. C. monolithic structure of the syphon completely collapsed. Downstream portion of the barrels along with the roadway had sunk deep into the gushing waters. Almost the entire supply of B. S. Link was now jetting into the Sukh Bias Nallah through the breach. He immediately informed the overseer about this mishap. B. S. Link at head was closed to save further damage but by that time the breach had developed to about 150 ft. in width causing deep scour and the gigantic structure had disappeared apparently to unfathomable depths.

This was how a serious accident on B. S. Link I took place when the supply in the canal at head was hardly 6725 Cs. against the designed discharge of 12000 Cs. Obviously under such apparently safe conditions such sort of happening is least expected. The causes of failure however have been identified and would be discussed at length in a subsequent issue.

B. S. Link connects river Ravi at Balloki Headworks to river Sutlej at the Sulemanki Headworks and feeds en route the truncated channels of Dipalpur Canal system as well. Its present designed discharge at head is 18500 Cs, while originally when it was commissioned in April 1954, it had an authorized full supply capacity of only 15182 Cs. The subsequent increase in discharge was envisaged as a part of Indus Basin Treaty.

Consequent thereupon, B. S. Link II was constructed in the years 1965-67 by WAPDA, offtaking at R. D. 73250 B. S. Link I with a full supply discharge of 6500 Cs. Both the links run parallel to each other with a common bank and outfall into the same creak of

*Superintending Engineer Link Circle, Lahore.

river Sutlej. B. S. Link I is lined from the point of offtake of B. S. Link II to tail (R. D. 266000) whereas the latter is an earthen channel all through up to tail (R. D. 194000). B. S. Link II runs in deep cutting up to R. D. 106 while its remaining reach traverses across the loose and sandy terrain of old river bed and thus is in heavy filling.

As usual with Irrigation channels, B. S. Links intercept the line of natural drainage and to provide disposal of storm and seepage water, apart from a few inlets three Nos. syphons have also been constructed. The biggest syphon for the disposal of flood waters being on Sukh Bias Nallah crossing B. S. Link I at R. D. 121560. It was constructed in 1953 with a designed capacity of 3000 Cs., comprising 8 R. C. C. barrels of $6' \times 6'$ each, spanning about 230′ from end to end.

Besides feeding Depalpur Canals, in fact B. S. Link serves as a life line for delivering vital Irrigation supplies to SVP canals. With the life line thus severed, the prospects of sowing Rabi crop in Bahawalpur, Sahiwal and Multan area presented a gloomy picture. The adverse blow to the economy of the country could be well imagined when we consider the vast tracts of fertile lands in Bahawalpur, part of Punjab area which instead of yielding bumper crops would have faced veritable famine conditions.

Breach occurred approximately in a width of 150′ and had slumped the barrels in the right half of channel into deep scour while in the other half, they were tilted upwards. The structure had completely failed and was found beyond repairs.

Reconstruction of the syphon in a short period appeared to be an impossible task, more so, as the situation around was such that alternative site for its construction was not available. The only way out was to construct the structure at its original site but that meant complete removal of the huge monolithic structure sunk deep into the scour pit and also screening of the entire substratum so as to ensure that no debris was left behind which could present a potential threat to safety of the new structure. This had to be done within the minimum possible time. Obviously it was humanly impracticable to accomplish this assignment within a few days but at the same time restoration of supplies to SVP canals system was a matter of extreme urgency and could not wait that long. The only alternative was to start with the construction of a diversion channel at the breach site simultaneously along with the preliminaries concerning its reconstruction without losing any time.

All this was a big challenge to the Engineers in charge. The construction of diversion alone which was about one mile long and about 310′ in width involved an earthwork of 75 lac cft. Similarly in reconstructing the syphon at the old site, the trickiest and the most arduous task was the extraction of sunk in monolithic structure after complete dismantling under 20′ water column and combing of entire substratum so as to be sure that all debris had been weeded out. Shattering of the monolithic barrels was a delicate affair as the contiguous syphon under B. S. Link II had to be safeguarded against any possible damage caused by shock produced during blasting or Hammering operation and called for utmost vigilance.

The reconstruction of the syphon envisaged complete boxing of the area by driving 15′ long sheet piles aggregating 13000 sft., Sand filling 3.20 lac cft., Earthwork 8 lac cft., R. C. C. and Cement concrete 1,21,000 cft., Cement masonry 80000 cft., and relaying tile lining 20000 sft.

Both the works which cost about Rs. 45.6 lacs were started simultaneously. The huge diversion was completed in a short period of, 30 days and supplies restored : The old syphon was shattered and extracted out to clear the site and the new structure was constructed in a brief period of about 120 days from the date of start. Most of the work of reconstruction was done under 20' depth of water. About 300 skilled, and semi-skilled labour along with heavy machines and other appliances toiled around the clock for four long months to complete the job.

Behind the successful operation lay the masterly planning of the Engineers rich in construction experience and the dedicated efforts of the staff at site who had to work relentlessly with a missionary zeal coordinating their efforts with various agencies to achieve a high quality and the optimum progress.

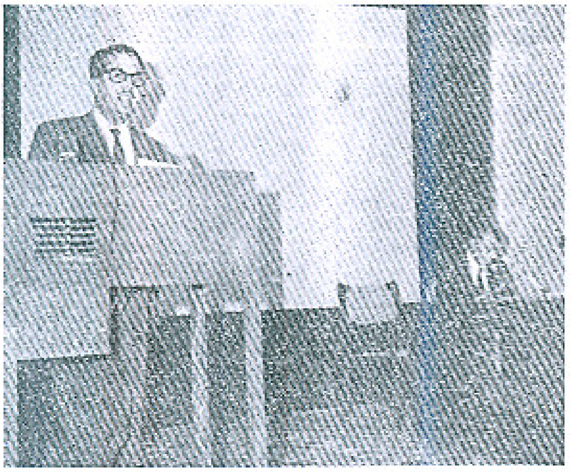
Credit for this seemingly insurmountable job specially in view of the tight schedule available goes to the staff and officers in-charge of this assignment. The syphon which was completed in June, 1971 would however be commissioned on restoring the banks at entry and the exit ends which is subject to the availability of a fortnight closure at the head.

BUILDINGS AND ROADS SECTION

Gypsum as a Building Material

A LECTURE BY AN AUSTRALIAN SCIENTIST

Dr. M. J. Ridge, D.Sc., Senior Principal Scientific Officer, Division of Building Research, CSIRO, Australia was in Lahore from 30th July to 4th August 1971. This trip was sponsored by CSIRO Australia and formed part of a usual feature in the research administration of Australia to enable its scientists to establish contact with research workers abroad and get abreast with the developments taking place in the rest of the world. Dr. Ridge visited the Building Research Station, Lahore on his own initiative and was persuaded to give a lecture on "Use of Gypsum in Buildings in Australia". He delivered his lecture in the Irrigation Research Hall on April 2, 1971 at 9 a.m. The lecture was presided over by Mr. Muhammad Khalil, Chief Engineer Buildings, Punjab, and was well attended by Engineers and Scientists.



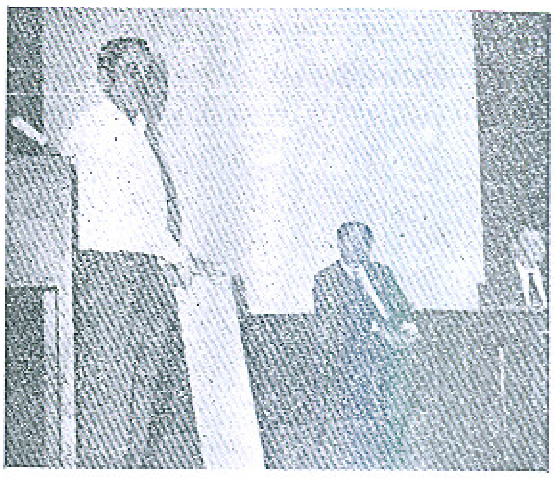
Mr. Mohammad Khalil, Chief Engineer, Punjab Buildings Department, introducing the guest speaker, Dr. M. J. Ridge.

Dr. M. J. Ridge delivering his lecture on Gypsum as a building material.

Dr. Ridge informed the audience that Gypsum was being used quite successfully in Australian Buildings for well over half a century. The principal form of gypsum in Australian buildings was Gypsum boards or Fibrous Plaster Sheets. These boards were nailed on the timber frame for walls or ceilings. The gypsum boards consisted of calendered panels covered with vynyl or cardboard or paper sheet while the fibrous plaster consisted of gypsum reinforced with about 4% Sissal fibre. The Australian industry had also produced high strength gypsum having a crushing strength of about 3000 p.s.i. as against the normal strength of 1800 p.s.i. Dr. Ridge informed that load bearing and structural gypsum was also being used in Australia. The only snag in the use of gypsum in buildings was that it would not stand against direct water contact.

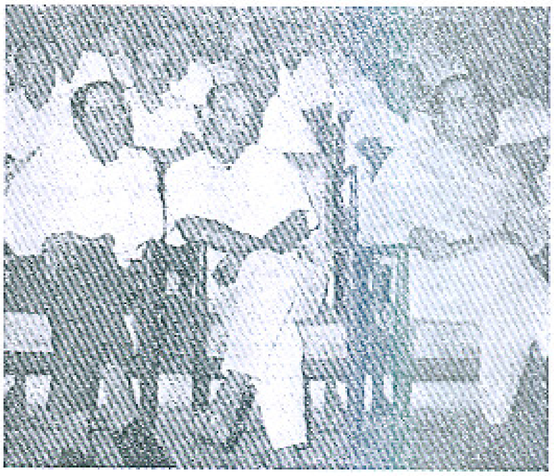
The popular form of the use of gypsum in Australia was, of course, fibrous plaster sheets of 3/8" to 1/2" thickness of varying lengths. At some places sheets over 35 feet length had been cast in factories. The casting method initially was labour intensive while it has now been mechanized due to increasing labour costs. The casting method was quite simple; gypsum slurry with about 70% water/solid ratio was poured on highly glazed and greased surface of a table to the required thickness and teased Sissal was spread uniformly and then pressed down by special rollers while the slurry was still in a liquid form. The cast sheet was de-pelletized after about 20 minutes and allowed to dry in a special hangar so that air could circulate both the surfaces of the table. This board was nailed to timber framework of the building at 18" c/c. The basic qualities of the fibre suitable for this purpose were its tensile strength and its property of remaining stiff when wet. The Sissal fibre was being imported by Australia from East Africa.

The lecture was very interesting and was well illustrated by slides. It evoked interesting discussion afterwards in which a number of Engineers took part. Mr. Ashfaq



Mr. Ashfaq Hasan, Director, Building Research Station, elaborating some points. Here a sample of fibre plaster sheet made at the building research station is being shown.

Hasan, Director Building Research Station, Lahore intimated that it was a pity that in spite of abundance of gypsum in Pakistan, we were not using it in the Building Industry



A view of the audience.

although the climate of part of Australia was quite similar to that of West Pakistan. He informed that the Building Research Station had cast a board 1″ thick using 'Munj' fibre as reinforcement but the tensile strength of fibre had not yet been determined owing to lack of equipment. He was of the opinion that there was a large scope of exploitation of this material for building and even bloated blocks could be produced to reduce load, increase productivity and lower construction costs. The bloating was necessary on economical grounds because the current price of calcined gypsum (Plaster of Paris) was about Rs. 250 per ton in Lahore.

It was surprising that the gypsum stone from Daudkhel should be available at about Rs. 7/- per ton while its cost should jump to Rs. 250 per ton after calcining. Mr. Ashfaq Hasan informed that the calcination of gypsum was done at only 180°C in a much simpler and cheaper plant than cement while its cost was much higher than the latter. The industry needed a thorough probe and revitabilization. He thought that calcined gypsum (hemihydrate) popularly known as Plaster of Paris should not cost more than Rs. 110/- per ton inclusive of 20% profits, as per following analysis based on 10 tons/day production:

Raw Material	Rs.	7 per ton	
Freight up to Lahore	Rs.	35 per ton	
Total	Rs.	42 per ton	(A)
Capital cost of Machinery	Rs.	3,00,000	
Building and Installation	Rs.	1,50,000	
Total	Rs.	4,50,000	
Depreciation of machine/ton $\frac{300,000}{20 \times 300 \times 10}$ ..	Rs.	5	(B)
Depreciation of building/ton $\frac{150,000}{50 \times 300 \times 10}$ ..	Re.	1	(C)
Staff Pay per day	Rs.	150	
Fuel etc. per day	Rs.	30	
Sundries	Rs.	20	
Maintenance and repairs	Rs.	50	
	Rs.	250	
Running cost	Rs.	25 per ton	(D)
Bagging	Rs.	15 per ton	(E)
Total cost: A+B+C+D+E=Rs. 42+5+1+25+15	Rs.	88	
Profits	Rs.	18	
Taxes	Rs.	4	
Sale price ..	Rs.	110 per ton.	

The work on gypsum as building material was being organized in the Building Research Station, Lahore on the following lines:

(*i*) Evaluation of quality of gypsum rock;

(*ii*) Evaluation of quality of calcined gypsum;

(*iii*) Casting of boards and blocks;

(*iv*) Production of light weight, high strength and structural gypsum.

Dr. Ridge informed that the Australian Building Research Organization would do whatever was possible to help the popularising this material in Pakistan. He offered to help in analysing a few samples of gypsum rock and calcined gypsum besides providing with relevant papers and literature.

Strength Aspects of Cement with the proposed Pakistan Standard Sand

By **MOHAMMAD TAHIR[1], G. F. ZAFAR[2]
I. H. HAMDANI[3]**

This report supplements special report No. 351-Ph/BM 14/67 already published by the Irrigation Research Institute and compares the Tensile as well as Compressive strengths of cement obtained with Karachi sand (Thano Bulla Khan site) and the imported Ottawa sand. It has been observed that the values of tensile and compressive strengths obtained with Karachi sand are slightly lower than those obtained with Ottawa sand but still they are much above the requirements of International Standards.

Mathematical relationships have also been established between tensile and compressive strengths of cement obtained with Karachi and Ottawa sands respectively.

INTRODUCTION

Some work in connection with selection of Standard Sand for Pakistan was carried out in the Irrigation Research Institute, Lahore so that a substantial amount of foreign exchange required to import this material from abroad could be saved and dependence on other countries for procurement could also be avoided. Various sites were explored and samples were analysed both physically and chemically. All the results are discussed in the special report No. 351-Ph/BM-14/67 issued by the Institute. Considering all factors regarding specifications of standard sand and economics involved in procurement of sand from various deposits, the best suited site of Thano Bulla Khan (near Karachi) was recommended in the above-mentioned report.

The one aspect which remained to be studied was whether the cement mortars with this sand would yield the same tensile and compressive strengths as obtained with the imported one. Therefore these experiments were carried out and the results obtained are discussed in this special report

EXPERIMENTS

I. Tensile strength

Separate sets, for Karachi sand (grade 20 mesh to 30 mesh) and imported Ottawa sand, of six briquettes each for 3, 7, 14,

1. Principal Research Officer. 2. Research Officer. 3. Assistant Research Officer.

TABLE No. 1

Comparative compressive strengths of cement with Pakistan Standard sand and Ottawa standard sand.

Cement-sand ratio = 1 : 3 (ASTM) Water for gauging mortar=8.0%

Curing period in days	Type of sand used	Compressive strength lbs/sq. inch						Average compressive strength lbs/sq. inch
		1	2	3	4	5	6	
3	Pakistan standard sand	1980.0	2117.0	2365.0	2255.0	1787.0	2447.0	2158.0
	Ottawa standard sand	2475.0	3190.0	2310.0	2200.0	3080.0	2860.0	2685.8
7	Pakistan standard sand	3132.5	2842.0	2375.0	2640.0	3456.0	2560.5	2834.3
	Ottawa standard sand	2997.5	3025.0	3410.0	3135.0	3520.0	3795.0	3313.7
14	Pakistan standard sand	4207.5	3740.0	4251.5	3212.5	3190.0	4246.0	3807.7
	Ottawa standard sand	4565.0	4317.6	4455.0	4152.5	4620.0	4097.5	4367.9
28	Pakistan standard sand	4482.3	4235.0	4270.0	4194.0	3576.0	4541.0	4216.4
	Ottawa standard sand	4973.0	4835.0	5211.0	4742.0	4684.0	4888.0	4888.8
90	Pakistan standard sand	3947.0	4579.0	4564.0	4837.0	4598.0	4533.0	4509.6
	Ottawa standard sand	5403.0	5228.0	4679.0	4924.0	5097.0	5190.0	5086.8

TABLE No. 2

Comparative tensile strength of cement with Pakistan standard sand and Ottawa standard sand.

Cement-sand ratio=1 : 3 (ASTM) Water for gauging mortars=8.0%

Curing period in days	Type of sand used	Tensile strength lbs/sq. inch						Average tensile strength lbs/sq. inch
		1	2	3	4	5	6	
3	Pakistan standard sand	387.5	342.0	368.0	371.0	358.0	366.5	365.5
	Ottawa standard sand	424.0	400.0	369.0	372.0	385.0	383.0	388.8
7	Pakistan standard sand	434.0	488.0	440.0	444.0	466.0	442.0	452.3
	Ottawa standard sand	485.4	450.2	474.2	450.2	478.0	493.0	472.5
14	Pakistan standard sand	570.9	560.4	482.0	499.4	568.0	523.0	550.6
	Ottawa standard sand	593.2	586.4	543.6	576.5	568.0	543.0	568.4
28	Pakistan standard sand	597.4	572.8	590.0	595.5	599.0	587.8	590.4
	Ottawa standard sand	610.5	633.7	592.3	609.0	612.5	611.6	612.0
90	Pakistan standard sand	598.3	639.4	640.2	609.0	623.0	620.0	630.0
	Ottawa standard sand	660.2	663.8	670.5	662.1	662.6	665.3	664.5

TENSILE STRENGTH VERSUS CURING PERIOD (DAYS)
CURVES OF 1 : 3 CEMENT - SAND MORTARS WITH PAKISTAN STANDARD SAND AND OTTAWA STANDARD SAND

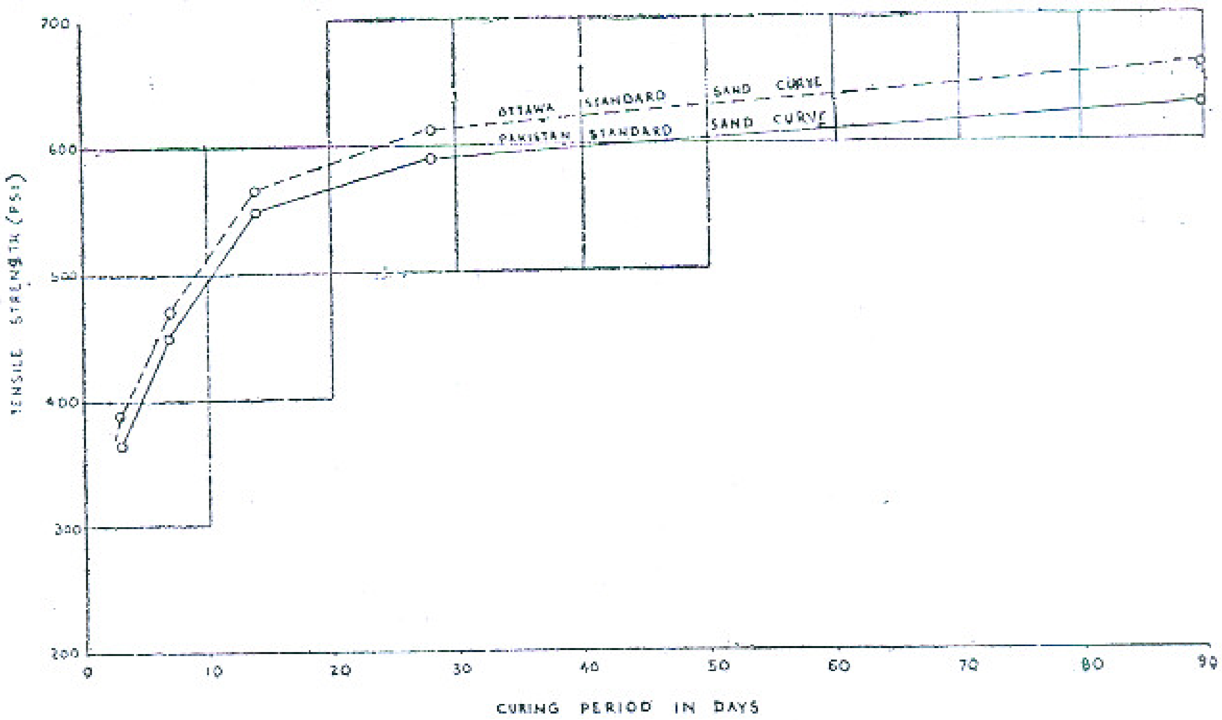


Fig. 1.

COMPRESSIVE STRENGTH VERSUS CURING PERIOD (DAYS)
CURVE OF 1 : 2.75 CEMENT SAND MORTARS WITH PAKISTAN STANDARD SAND AND OTTAWA STANDARD SAND

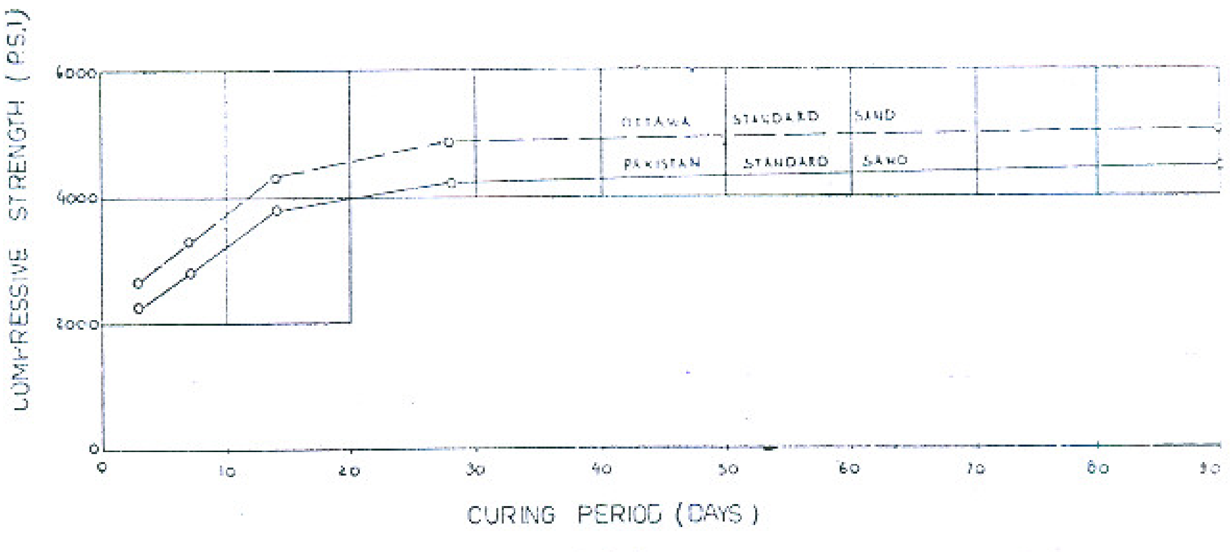


Fig. 2.

COMPARATIVE TENSILE STRENGTHS OF 1 : 3 CEMENT-SAND MORTARS WITH OTTAWA SAND AND PAKISTAN SAND FOR EQUIVALENT CURING PERIODS

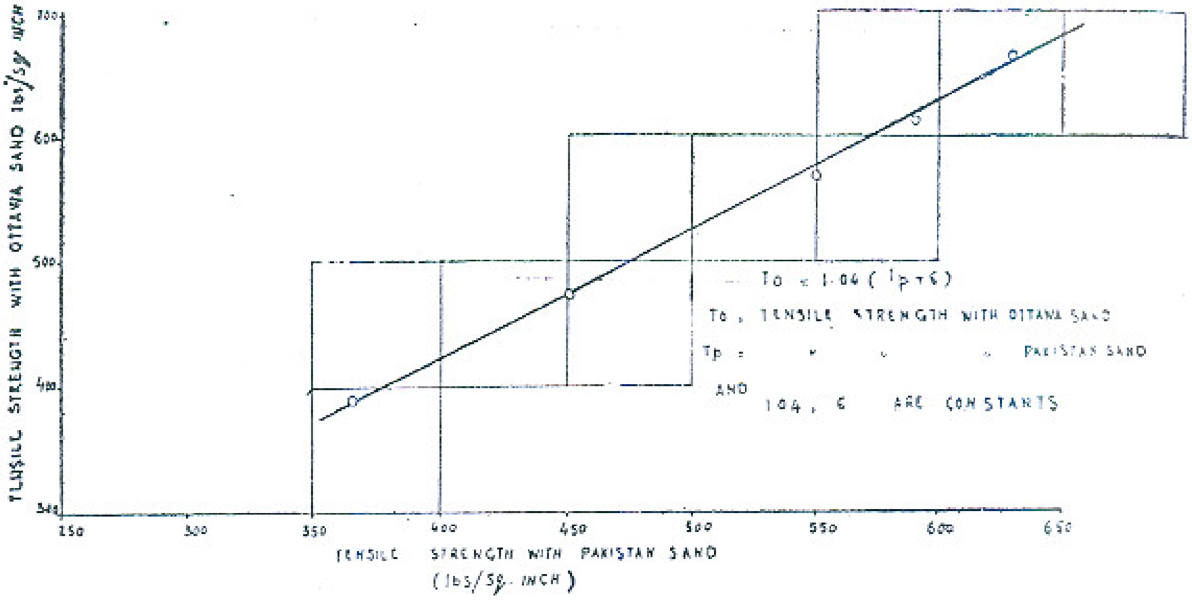


Fig. 3.

COMPARATIVE COMPRESSIVE STRENGTHS OF 1 : 2.75 CEMENT-SAND MORTARS WITH OTTAWA SAND AND PAKISTAN SAND FOR EQUIVALENT PERIOD OF CURING

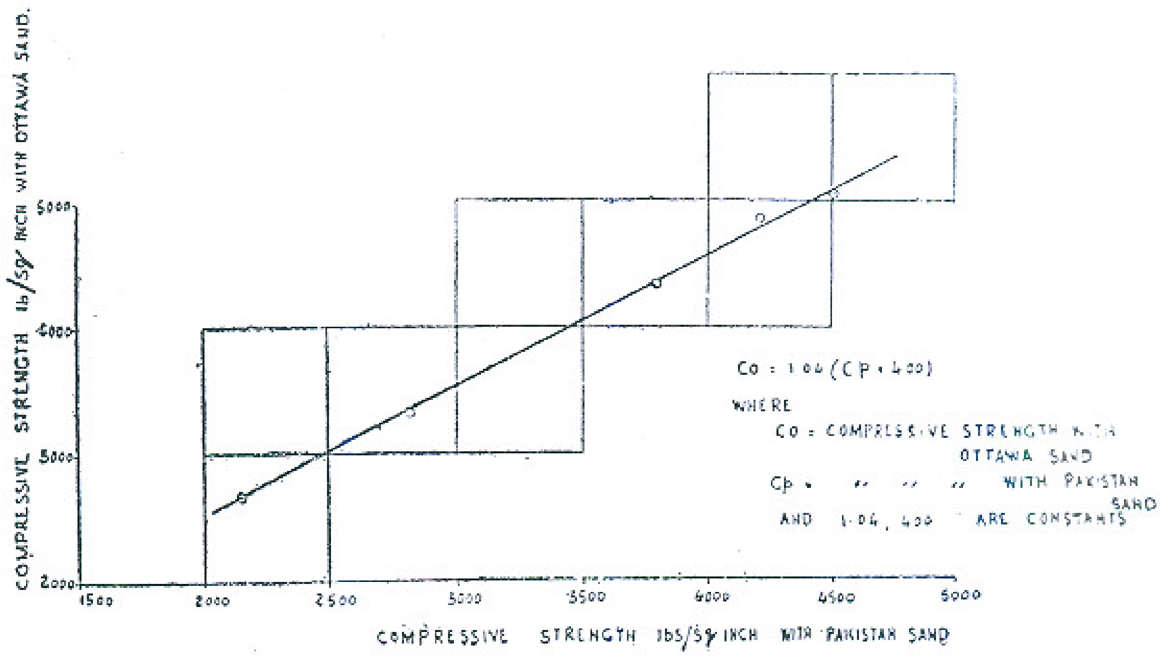


Fig. 4.

28 and 90 days strength were prepared as per ASTM keeping the cement-sand ratio to be 1 : 3. The testing was carried out simultaneously under identical conditions and results are given in Table 2.

II. Compressive strength

Similarly sets of six cubes for each observation of compressive strength (as for tensile tests) were prepared with cement-sand ratio of 1 : 2.75 as per ASTM and tests were carried out on due dates for 3, 7, 14, 28 and 90 days curing periods. These results are included in Table 1.

DISCUSSION OF RESULTS

It has been already mentioned that both tensile and compressive strength obtained with Karachi sand (although within standard specifications) are slightly lower than those obtained with Ottawa sand. The fluctuations range from 3.0 to 6.0% in case of tensile strength and from 11.0 to 19.0% in case of compressive strength for different curing periods, as shown in Tables 1 and 2 and graph sheets 1 to 2.

Graph sheet No. 3 shows a plot of tensile strengths obtained with Karachi sand against the corresponding results of imported Ottawa sand. Similarly graph sheet No. 4 shows a plot of compressive strength for the two sands. It is seen that in both the cases, a very good straight line relationship exists. The equations of these two straight lines have been derived as given below:

I. $T_o = 1.04\,(T_p + 6)$ . . . . . (1)

where T_o = tensile strength of cement obtained by using Ottawa sand in the rato 1 : 3.

and T_p = tensile strength of cement obtained by using Pakistan (Karachi) sand in the same ratio 1 : 3.

II. $C_o = 1.04\,(C_p + 400)$ . . . . (2)

where C_o = compressive strength of cement obtained by using Ottawa sand in the ratio 1 : 2.75

and C_p = compressive strength of cement obtained by using Pakistan sand in the ratio 1 : 2.75 as per ASTM.

Incidentally it is found that slope of both the straight lines is the same, *i.e.*, 1.04. It confirms that development of both tensile and compressive strengths with time is uniform and proportionate.

INFERENCE

1. Pakistan sand available in huge quantities at Thano Bulla Khan near Karachi can replace the imported sand for all testing purposes connected with cement. The grade of this sand, *i.e.*, between 20 to 30 mesh (ASTM) is economically exploitable at the said deposit.

2. Tensile and compressive strengths of cement obtained by using Pakistan sand fall within the International specifications but when compared with the imported sand, they are slightly lower in values but a definite relationship exists between the two *viz.* $T_o' = 1.04\,(T_p + 6)$ for tensile strength and $C_o = 1.04\,(C_p + 400)$ for compressive strength.

Thus Pakistan sand from Thano Bulla Khan is recommended to be declared fit as Pakistan Standard Sand. The strength specifications may not be altered because the results obtained are much above the International Standards.

The following table gives a comparison between ASTM standards and the results obtained by this Institute as discussed above.

Strengths/curing period		Strength specifications as per ASTM C—150 lbs/sq. inch	Strengths as determined with imported standard sand lbs/sq. inch	Strengths as determined with Pakistan standard sand lbs/sq. inch
(*a*) **Compressive strength (psi)**				
1 day in moist air 2 days in water	..	1200	2685.8	2158.5
1 day in moist air 6 days in water	..	2100	3313.7	2834.3
1 day in moist air 27 days in water	..	3500	48888.8	4216.4
(*b*) **Tensile strength (psi)**				
1 day in moist air 2 days in water	..	150	388.8	365.5
1 day in moist air 6 days in water	..	275	472.5	452.3
1 day in moist air 27 days in water	..	350	612.0	590.4

ACKNOWLEDGEMENT

Personal company of Mr. Afaq Ahmad (Deputy Director, Pakistan Standards Institution) to Thano Bulla Khan site to help and guide Mr. G. F. Zafar, Research Officer of this Institute, is gratefully acknowledged. He has also been keenly interested in the progress of this study.

GENERAL SECTION

U.N. Symposium on Planning for Comprehensive Regional Development (Warsaw) 1971

By S. M. H. BOKHARI, PSE-I
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The Inter-Regional Symposium on Planning for comprehensive Regional Development was convened in Warsaw from 14 to 28 June, 1971 by the United Nations in cooperation with the Government of Poland in pursuance of ECOSOC Resolution No. 1086-C/XXXIX. It was designed to promote exchange of information on research and training in regional planning as an important instrument for national development.

The Symposium was attended by official delegates from twenty developing countries basides a large number of observers from developed countries like USSR, Netherlands and Poland, the United Nations Organizzation, U. N. Consultants and non-governmental organizations affiliated with the United Nations. It was indeed a great honour for me to represent Pakistan on this extremely useful gathering of International Experts on Regional Planning.

The venue of this Symposium was "THE PALACE CULTURE" which is the tallest and the most beautiful building constructed by Russians in the heart of Warsaw as a gift to the Polish nation, during the reconstruction phase of Warsaw after the second World War.

The Symposium was inaugurated by Professor Dr. K. Seconski, the Deputy Chairman of Planning Commission, Poland who welcomed the participants on behalf of the Polish Government and expressed appreciation of the United Nations decision to hold this Symposium in Poland. The United Nations' Director of Symposium Mr. D. Madawela replying to the welcome address, read out a message from the United Nations Secretary-General containing the appreciation for the cooperation extended by the Government of Poland in hosting the Symposium. Mr. Madawela paid tributes to the Polish Organization Committee and particularly the National Director of the Symposium for providing most befitting environments for conducting the Symposium.

During the two weeks of deliberations, a large number of papers written by the United Nations Consultants and Polish Experts were discussed in the Symposium. A country paper containing the National policy on regional development was also submitted by each country representative. Due to the limited space only a brief account of the conclusions drawn is given hereunder :

(1) Regional development planning should be promoted as an important instrument for achieving a more effective integration of economic, social and environmental aspects of development. It can play a major role in adapting national policies to regionally differentiated conditions; providing criteria for better allocation of resources among regions; coordinating sector activities within regions; harmonizing national priorities with local resources, potential and locally articulated aspirations; making economic, social, cultural and political opportunities accessible to population in all parts of a country; and in improving quality of life through improved patterns of urban and rural human settlements and spatial planning.

(2) Comprehensive planning involves participation and action by many population groups, professional disciplines and specialists, administrators and political decision-makers. There is thus a need for training of all these sectors in a manner that besides specialization of various fields, there is a common media for better communication and pooling up of the knowledge for integrated planning.

(3) Flexibility should be maintained in delineating regions to assure their compatibility with the national requirements of socio-economic development and with rapidly changing economic, social and technological developments.

(4) There is an urgent need for continuing exchange of experiences and information amongst different countries, both developed and developing, promote better understanding of the conceptual, methodological and administrative frameworks of the regional development planning and to systematize the knowledge gained in different countries.

(5) Regional statistical and information systems should be re-designed and developed to meet the needs of different users including the regional planners, administrators, political leaders and public at large.

(6) In many developing countries there is a problem to select the leader of the team engaged in the regional planning. Such persons may come from a variety of disciplines and should possess capabilities to coordinate and integrate the specific sectoral plans into a well-knit regional plan. Such personnels must be made the recipients of special training programme to develop their analytical and coordination skills. As they move into administrative positions, they should be given training in managerial and administrative functions.

(7) United Nations should assist the developing countries in establishing national centres or otherwise training regional development planners through multi-national and global network of training centres. The major effort is also needed in developing training materials and methodologies for regional development planning. For this reason, research in different aspects of regional development planning should be increased at national and international levels. Consideration should be given by the United Nations to the issuance of a Regional Development Planning News-Letter to disseminate information on the subject and to promote exchange of experience in different countries.

(8) Future regional and inter-regional symposia should be organized by the United Nations on specific aspects of regional development planning to promote understanding of the concepts and practices of the subject.

Besides Conference-Room deliberations summarised above, I made the best use of the week-ends meant otherwise for rest and sight-seeing in visiting Zwar and Elta Factories of Messrs. Elektrim who are engaged in supplying electrical transformers, circuit breakers, etc., to WAPDA under a barter deal. I also visited the Polish Planning Commission twice to understand their day-to-day functioning and the practical methodology adopted by them for planning comprehensive regional development. One day trip was arranged by the Planning Office for me to see their irrigation, drainage and agricultural practices within fifty miles radius of Warsaw. The International Industrial Fair at Posnan was also witnessed. It is amazing to note that the city of Warsaw, which was reduced to ruins by Nazis, has been reconstructed to the same shape, size and dimension as it stood on the first day of Second World War.

During the second stage development the city of Warsaw is being cleared of slumps and multistoreyed housing apartments are springing up. Each of the newly developed housing schemes is self-sufficient in the essential services and recreational facilities. To relieve the traffic pressures and jams due to level road crossing, under passages and super passages have been planned for the entire metropolitan area of Warsaw. This work is likely to be completed during the next ten years.

In September, 1966, I attended a United Nations ECAFE Conference on Water Resources Development, in Australia, as a

Mr. Bokhari reading his paper in the U. N. ECAFE Conference.

Pakistani delegate and toured throughout irrigated areas of Australia after the Conference. It can be concluded that participation in the current Symposium was a continued link of this series and it added substantially to the know-how on planning in general and regional planning in particular for comprehensive development. Both the United Nations Seminars were extremely educative and afforded opportunities to see the overall development efforts in the different socio-economic systems—the Australia being an open market economy and the Poland a socialist economy. Both these nations are striving hard with obvious successes in the overall development of their countries through schematic planning at local, regional and national levels. Whereas Poland offers a unique example of city planning with minimum financial resources, Australia provides a good demonstration of planned urban as well as agricultural development and technological advancement. The people of both the countries are very hospitable, cooperative and kind-hearted.

While coming back from Warsaw I had the privilege of performing "Umra" and pilgrimage to the holy places in the Middle East.

NEWS AND NOTES

Air Pollution

Air pollution is generally recognized as one of the most significant and challenging problems of modern society. It is a paradox that with an advancing and expanding technology, the quality of air continues to deteriorate as more and more pollutants are poured into the atmosphere. Air quality control limits should be based on an analysis of the medical effects of air pollution, including a calculation of the minimum time of exposure to various pollutants. If situation is considered to be hazardous to health, then it is important to predict short term measures.

On the International level, the United Nations Conference on the Human Environment will be convened in 1972 to consider the global effects of all types of pollution. Among the objectives will be an attempt to organize the pollution control efforts through multilateral action by many nations and formulate world wide air quality standards.

A little electricity cleans waste water

A little electricity can go a long way toward cleaning up industrial waste water by removing particulate matter, according to Swift & Company, U.S.A., which claims its system is more effective and costs less than most facilities now in general use. The system uses coagulating chemicals to join the waste particles, followed by a direct current to drive the particles to the surface of a holding tank, where they can be removed. The process eliminates the needs for settling basins which can take up a large area. Only .1 Kw of D. C. power is required for the largest tank built (100 ft. in dia). This seems to be the limit for a single anode; but they are hoping to experiment with several anodes in larger tanks. The process reportedly has been successfully tested and is being installed at an allied chemical corp fabric-finishing plant at Moncure, N. C. U. S. A.

Intermittent water supply a health hazard

Mr. J. V. Dibble, World Health Organisation Sanitary Engineer in his paper on "Planning and Maintenance of Community Water Supply" observed that water supply must be maintained for 24 hours a day, as intermittent supply was a health hazard.

In an intermittent water supply, he said, there was the risk of waste water entering the supply mains during periods of low or no pressures and this risk was far over-weighed the small additional cost of supply water for full 24 hours with mains under pressure. The cost of one course of antibiotics might cost up to five times the monthly rate of piped water supply round the clock.

Engineering Marvels 1971

APOLLO 15, MOON CAR

United States sent a moon buggy along with Apollo 15 mission on July 16, 1971. This lunar Rover vehicle was driven by the astronauts for 27.8 Killo-meters in three excursions. The lunar Rover vehicle was parked on the edge of Hadley Rille. The vehicle was a complete unit with High gain antenna, 16 mm, 70 mm Cameras and T. V. Camera. The man has used his auto-invention on the surface of moon with equal success, as was using on the earth.

LUNOKHOD FROZEN TO SILENCE IN LUNAR NIGHT

Russia's Lunokhod moon probe has been chilled into eternal silence by the icy lunar night.

It is said the moon buggy ceased functioning when its 12th lunar day was just beginning, after its isotope heat source lost power and its temperature dropped.

Without this artificial heating, the vehicle's systems could not survive the bitter cold of the lunar night, which last the equivalent of about 14 earth days and brings temperatures down to minus 100 degrees centrigrade (minus 159 fahrenheit).

Lunokhod, an eight-wheeled vehicle, was landed on November 17 last year and continued to operate for about twice as long as its designers originally planned.

Lunokhod during its nearly 11 months of operations covered 6.5 miles criss-crossing the sea of Rains took more than 20,000 television pictures and probed the lunar soil more than 500 times.

The information Lunokhod yielded increased man's knowledge of the moon, the sun and space.

Now it was stationary, a French-made laser reflector mounted aboard the vehicle could still be used for experiments for a long time, Tass said.

The reflector is part of an earth-moon radio interference meter and the highly concentrated light beams if reflected back to earth are monitored by scientists.

AND NOW H-POWERED CARS

A Miami (U.S.A.) inventor says he has succeeded in running standard, Detroit-built automobiles on hydrogen, more cheaply than on gasoline, and with no air pollution exhaust emissions.

Morris Klein has been running three different cars on hydrogen for eight months.

Enough hydrogen to drive either car 100 miles costs only about 40 U. S. cents. Klein says he intends to enter a hydrogen-powered car in the 1972, Clean Air Race sponsored by Massachusetts Institute of Technology. He expects ultimately to generate hydrogen on the road from a fuel tank in the car filled with water.

The implications are important. If Klein's system should prove successful, it might solve the automobile exhaust emission problems at no great cost to the automakers or the public.

It also could help solve the world energy problems. Hydrogen is the mo vehicles run source of energey in nature,making up about two-thirds of all fresh and salt water. It is extracted from water by electrolysis and other process or obtained as a by-product of petroleum refining or of the production of nuclear generated electric power.

If automotive vehicles could be switched over to run on hydrogen, vast amounts of petroleum could be diverted to petrochemicals, plastics, man-made fibres and other uses.

Converting a standard auto-engine to hydrogen fuel merely requires the introduction of the gas into the cyclinders by means of tubes, by passing the fuel pump and carburettor. On his station wagon, Klein can switch back and forth between gasoline and hydrogen fuel by turning a knob.

The carburettor and fuel pump have been removed and the vehicles run on Hydrogen only.

Attention : Members Please

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All papers submitted for publication must be forwarded to the Editor Engineering News two months ahead of the date of publication *i.e.* in January, April, July and October. Papers must be typewritten with double spacing on foolscap paper. Two copies of the paper are to be submitted.

All the relevant diagrams must be properly traced on transparent paper with Chinese Black Ink. Photographs should be on glossy paper. The diagrams and photographs must be properly labelled and numbered.

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