Engineering News

Vol. 19 No. 4

DECEMBER, 1974

A QUARTERLY JOURNAL OF

PAKISTAN ENGINEERING CONGRESS

CODE OF ETHICS

WEST PAKISTAN ENGINEERING CONGRESS

بِسْمِ اللّٰهِ الرَّحْمٰنِ الرَّحِيْمِ

In the name of God, the Beneficent, the Merciful

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

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

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

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

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

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

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

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

۲۔ أَوْفُوا الْمِكْيَالَ وَالْمِيزَانَ بِالْقِسْطِ وَلَا تَبْخَسُوا
النَّاسَ أَشْيَاءَهُمْ وَلَا تَعْثَوْا فِي الْأَرْضِ
مُفْسِدِيْنَ ○

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

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

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

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

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

۴۔ أَوْفُوا بِالْعُقُودِ ۭ

"Fulfil the obligations". v : 1

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

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• Price Rs. 5.00 per copy Rs. 20.00 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 about engineering works, news about engineers, photographs and technical data etc. are cordially invited.

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Back Cover, Outer Page	...500
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Ordinary Full Page	...150
Half Page	...100

Price of this Issue : Rs. 5.00

NINTEENTH YEAR OF PUBLICATION

ENGINEERING NEWS

Quarterly Journal of the West Pakistan Engineering Congress

Vol. XIX DECEMBER, 1974 No. 4

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TITLE COVER

A view of Downstream Right Bank Tarbela Dam showing muddy water discharging through Tunnels 1 & 2.

Printed by Sajjad Almakky at the Allied Press, 26-Shahrah-e-Quaid-e-Azam, Lahore

Reflections on Tarbela

Man has built great engineering monuments in the past to satisfy his personal ego, provide places of worship and defend himself against the ravages of nature or fellow-men. Today, both the motivation and scope of his exploits are undergoing a change. Modern technology has widened his scope to an extent where most engineering ideas that he conceives can also be translated into reality. Moving mountains is no longer a rare exploit of faith but a matter of casual engineering routine. As for motivation, it is no longer subservient to any whims for personal glory. The emphasis has shifted a long way from pyramids and Taj Mahals to Aswan and Tarbela Dams.

With all the material advancement however, man's exploitation by man continues with ever increasing vigour. In the highly complex, competitive and cut-throat political, socio-economic set up of human society, the poor nations of the world are faced with a paradox. Having escaped the fires of a long physical subjugation they are roasting in the frying pans of economic slavery. Militarily impotent, politically in-effective, economically dependent, technically backward, over-populated in proportion to their resources and production-specially food production-they are natural targets for perpetual exploitation. On the one hand, their delicate freedom or even existence is at stake and, on the other, they aspire for economic independence and prosperity, entering a desparate race to catch up with the advanced nations of the world. What keeps them in this unequal race ? At worst, surely, a lack of any other alternative but at best the grit, determination and will to survive and excel, compete against all odds and win.

Tarbela Dam, for Pakistan, is a symbolic manifestation of that stubborn will.

How many millions of rupees, cusecs of water, kilowatts of electricity, cubic feet of soil, sand, stone and cement, tons of steel and man-hours of sweat and toil add up to make a dream come true ? In the glitter of staggering statistics one can easily overlook the real significance of this gigantic exercise in civil engineering we call Tarbela Dam.

Viewed in the historical perspective of Partition of India, the conspiracy of the Radcliffe award, Indian hostilities, military escapades, political and economic blackmail and sabotage, stoppage of irrigation waters and the circumstances leading to the Indus Basin Treaty, the real significance of Tarbela is thrown into prominent relief. Suddenly, it ceases to be just a marvel of engineering and becomes one of a series of difficult battles against hostile forces opposing the very creation and continued existence of Pakistan.

The superstitious might find a lot in common between 'Trouble' and 'Tarbela' but dams are known to have run into difficulties in the distant as well as the not too distant past. The Egyptians learnt their first lesson almost five thousand years ago as they tried to tame the Nile The play has been re-enacted over the centuries with different settings around the globe. Some recent examples of failures were South Fork, America in 1889 due to overtopping ; Malpasset, Southern France in 1959 because of failure of left abutment and Viaiant, Northern Italy in 1963 as a result of a massive landslide.

So, the mere fact that there has been a temporary set-back at Tarbela is neither unique nor critically important. The question of financial losses is of lesser importance even allowing for the fact of our poverty. The important thing is have the lessons of Tarbela been well learnt, if at all ? A lot of questions plague the inquisitive and the concerned mind. What actually caused the failure ? Will the repair work be satisfactory and completed in time ? Will the project be as efficient now as originally envisaged ? The whole world is talking about the trouble at Tarbela and its implications (a sample of what they say is being reproduced in this issue for the benefit of our Pakistani readers). The pertinent question is why is there an ominous silence in Pakistan on the subject ? Is it some dubious expediency, lack of knowledge at the required level or a simple lack of courage ?

Pakistan is not passing through the best of times. Our salvation lies in speedy exploration, development and utilization of all our natural resources for the betterment of our poor masses. The ironic fact is that for this vital exercise we are entirely dependent on outsiders. We look on as silent spectators while our projects are planned, designed and implemented by outsiders who do not share our hopes and aspirations, being interested purely in their profits. Are we really aware of the dire necessity of increasing self reliance in all fields of national endeavour ? Are we striving to achieve an economically healthy and politically stable society which will enable us to defend our freedom and seek outside co-operation as equal partners ? Have we got our objectives and priorities right ? In the answers to these questions may well lie the future place and status of Pakistan in the comity of nations.

DR. MUBASHIR HASSAN

the new President of Pakistan Engineering Congress was born in 1922 at Panipat, East Punjab, India. He was educated at Government College, Lahore and graduated in Civil Engineering from the Engineering College, Lahore, in 1942. He joined the Punjab Irrigation Department for a few years after which he proceeded to the United States of America for higher studies. He obtained his Master's Degree in Civil Engineering from the Columbia University, New York. On his return in 1948 he joined the Civil Engineering Department of the Engineering College, Lahore.

Once again he proceeded to America for higher studies and earned his Ph.D in Civil Engineering from ths State University of Iowa in 1956. On return, he resumed his duties at the Engineering College which later became the Engineering University, He rose to be the Professor and Head of the Civil Engineering Department at the University. He was elected President of the University Teaching Staff Association. In 1962, his services at the University were terminated under a Martial Law Order by Mr. Ayub Khan. The termination of his services and of his other colleagues resulted in widespread student strike and protest demonstrations which led to the removal of the Vice Chancellor.

Dr. Mubashir then established his private practice at Lahore as a consulting civil engineer. When Chairman Zulfikar Ali Bhutto announced his intention to form a new political party, Dr. Mubashir joined him and the Pakistan Peoples Party's first Convention was held at his residence in Lahore in 1967. During the mass movement against Ayub Khan, Chairman Bhutto was arrested on November 13, 1968, from the residence of Dr. Mubashir along with Mr. Mumtaz Ali Bhutto and Dr. Mubashir.

Dr. Mubashir was again arrested by the Yahya regime on March 28, 1969 for flying the Pakistan People's Party flag on his house.

He was elected to the National Assembly from north Lahore constituency in the 1970 general elections by an overwhelming majority. When People's Party assumed power in 1971, Dr. Mubashir was sworn in as the Central Minister for Finance, Economic Affairs and Development. This office he held till recently when he was nominated as the Secretary General of the Pakistan People's Party.

Dr. Mubashir's keen interest in the activities and welfare of engineers is well known. Under his able and fearless leadership, the Pakistan Engineering Congress will not only maintain its cherished traditions but reach new heights of excellence.

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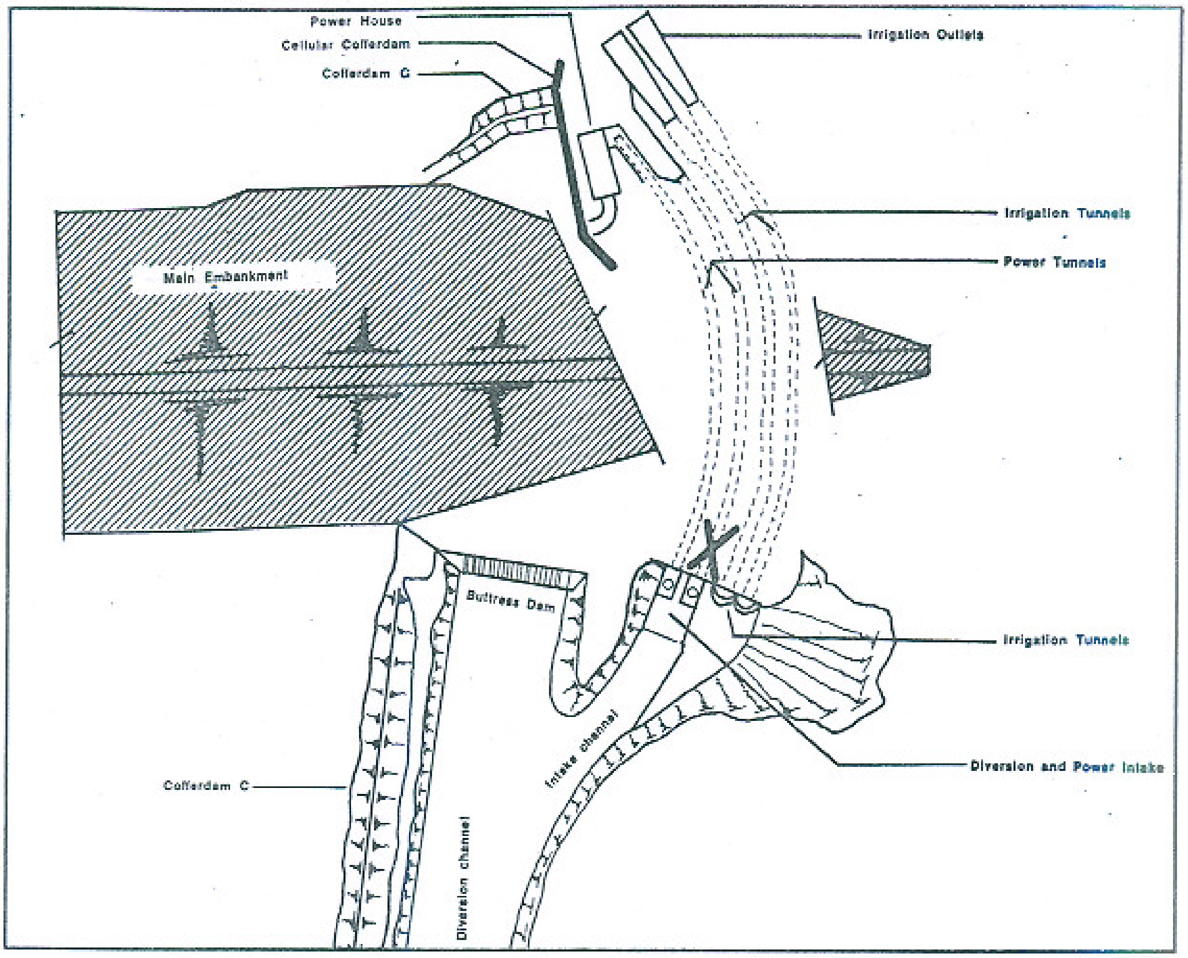


Plan of Tarbela's right abutment, showing the line of the four tunnels and the layout of the downstream works.

Tarbela tunnel drama

BRIAN APPLETON

Startling evidence emerged from Pakistan this week of the operational tightrope walked by engineers controlling discharges from Tarbela dam's four diversion tunnels, even before failure of tunnel 2 led to an emergency drawdown of the almost full reservoir and before discovery of a 360,000 cu m 'swallow hole' in the dam's right abutment.

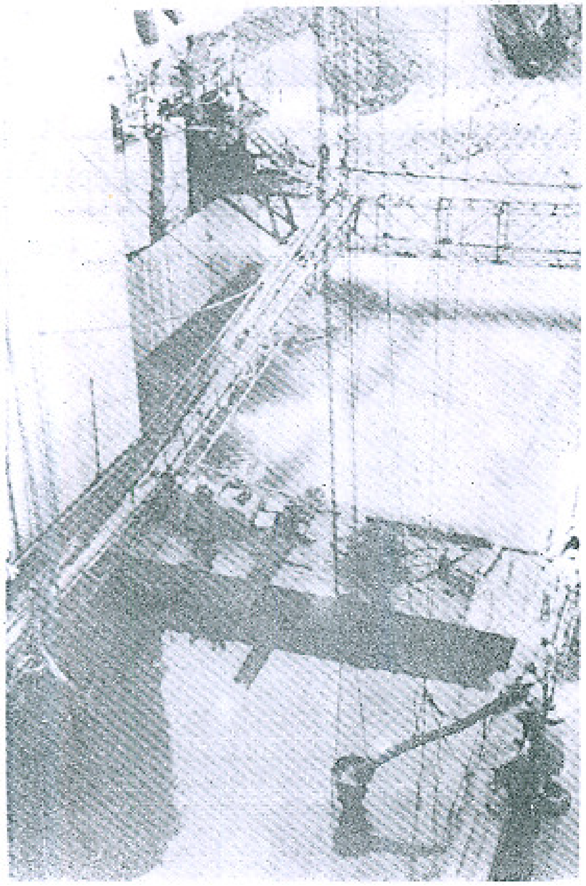
An inquiry into the collapse of the tunnel was begun last Friday by a 66-member panel of engineers and government officials. A special report, prepared by New York consultants Tippetts-Abbett-McCarthy-Stratton obtained at the site by *NCE*'s news editor Martin Routh, gives a detailed timetable of events before and after the draw down was ordered on 22 August.

The report shows the delicate juggling with intake gates on each of the tunnels that was necessary to control discharges and prevent erosion damage to the downstream works from mid-July onwards. As gates on one tunnel were closed to regulate the discharge, new flow patterns threatened cofferdams and structures in one section, but compensating adjustments to the tunnel gates only transferred the danger to other installations.

Filling of the reservoir officially started on 1 July 1974, according to a carefully prepared schedule, which would provide as much water as possible for downstream irrigation and still bring the reservoir to spillway level (455m) by the end of the wet season in September. By periodic operation of the ten intake gates (three each on tunnels 1 and 2 at level 332 m and two each on tunnels 3 and 4 at 354 m, fine regulation of irrigation releases was to be combined with performance testing of each gate.

But the schedule was upset on 15 July, when the first full closure of all three gates on tunnel 2 led to undermining and collapse of one cell of a cellular cofferdam protecting the power house. This sharp reminder of the need to control the pattern as well as the volume of flow downstream introduced a new element in the regulation procedure, and subsequent gate operations were aimed at minimising scour effects. It was found for instance that tunnels 3 and 4 could not both be fully open at the same time as wave action put the drainage works at risk.

On 24 July tunnel 1 was completely closed for the first time and this led to a



Tremie pipes (foreground) feed concrete down to a ten metre deep cavity below the intake foundations, where the rush of water through the damaged tunnel has washed away the natural bedrock.

threat to the second of the cofferdam cells. Attempts were made to close the centre gate of tunnel 2, but it would not move beyond the first 4.7 of a total 14.5 m closing distance. The gate was reopened.

Then on 31 July two earthquakes of strength 6.5 on the Richter scale jolted the site. No signs of damage were visible, but perhaps significantly just two days later, on 2 August, the first rock and concrete fragments were noticed downstream of tunnel 2. At that time it was not certain that the debris was coming from the tunnel, as turbulence could have been causing erosion of the downstream concrete splash slab.

More attempts were made to close tunnel 2's centre gate on 7 August but with no more success, and next day gates on tunnels 3 and 4 were partly opened to stop any further reservoir rise. Again dangers to the downstream drainage meant that the gate openings had to be reduced.

On 13 August the steel liner in one outlet passage of tunnel 3 failed and tunnels 3 and 4 were again closed off. Inspection revealed extensive damage to the liners in both tunnels and cavitation of the concrete.

Tunnel 2 centre gate was fully opened on 15 August and immediately stones appeared downstream. The gate was returned to the partly closed position where again it stuck, and the following day a side gate on tunnel 1 also jammed when partly closed.

So the first six weeks of reservoir filling provided a fair share of excitement for TAMS engineers on site. But these

The Tarbela timetable

29 September 1973
Gate closure in the buttress dam across the diversion channel sent the first flow through tunnels 1 and 2 (intake level 332 m) allowing construction of the closure section in the main dam.
Later the closure gates on both tunnels were satisfactorily operated, raising the reservoir level to El 340 m to prevent erosion of a rock weir protecting the diversion channel's impervious blanket.

15 June 1974
First flow through tunnels 3 and 4 was caused by increasing river flows raising the reservoir level to El 354 m.

1 July
Reservoir filling started, according to a predetermined procedure, aimed at regulating releases for downstream irrigation and filling the reservoir to spillway level by the end of the wet season in September.

8 July
Because of low river flows the filling curve was 23 m behind schedule.

13 July
Side gates of tunnel 2 closed in accordance with filling procedure.

15 July
Centre gate closed, blanking off tunnel 2 completely. Cell 18 of the cellular cofferdam attached to the power house collapses and is washed away. Collapse blamed on high outflow from tunnel 1 undermining cofferdam. Tunnel 2 centre gate reopened; tunnel 1 sidegates closed to protect remaining cells 13-17.

16 July
Downstream cofferdam G breached.

19 July
Resettlement authorities request slower rise in reservoir level to allow evacuation of remaining residents.

20 July
Gates on tunnels 3 and 4 fully opened. Wave actions threatens downstream drainage works. Tunnel 4 gates revised to half open.

24 July
Tunnel 1 centre gate closed. Tunnel 1 now completely blanked off.

26 July
"Machine gun or clapping noise" reported from air vent between outlet gates.

27 July
Outflow from tunnel 2 threatens cofferdam cell 17. Attempt to close centre gate fails after 4.7 out of 14.5 m. Gate reopened.

31 July
Two earthquakes.

2 August
First rock fragments and concrete aggregate observed downstream of tunnel 2 — source uncertain.

7 August
Second attempt to close gate 2 on tunnel 2 fails.

8 August
Tunnels 3 and 4 gates partly opened. Downstream drainage threatened — openings reduced.

13 August
Steel liner fails in tunnel 3

14 August
Tunnels 3 and 4 closed. Inspection reveals severe cavitation of chute concrete and damage to steel liners.

15 August
Tunnel 2 centre gate raised. Stones observed in tunnel discharge. Gate returned to partly closed position.

16 August
Tunnel 1 side gate sticks part open.

19 August
Stem breaks on Tunnel 2 centre gate.

21 August (23.30 hours)
Loud explosive noise from tunnel inlet area, discharge from tunnel 2 jumps from 1,000 to 4,000 cu m/sec. Discharge contains large quantities of rock debris. Whole abutment shakes with a "thumping vibration" of about 2 cycles/second.

22 August
Emergency lowering ordered. All tunnel gates opened progressively over next three days.

23 August
Large cavity noticed in free draining fill above tunnel 2.

24 August
Cofferdam G breached by downstream discharge of 7,500 cu m/sec.

25 August
Discharge peaks at 11,000 cu m/sec.

26 August
Cofferdam cell 17 collapses.

27 August
Cofferdam cell 16 collapses.

30 August
Cofferdam cell 15 collapses.

31 August
Cofferdam cell 14 collapses.

14 September
Damage to tunnel 2 visible for first time.

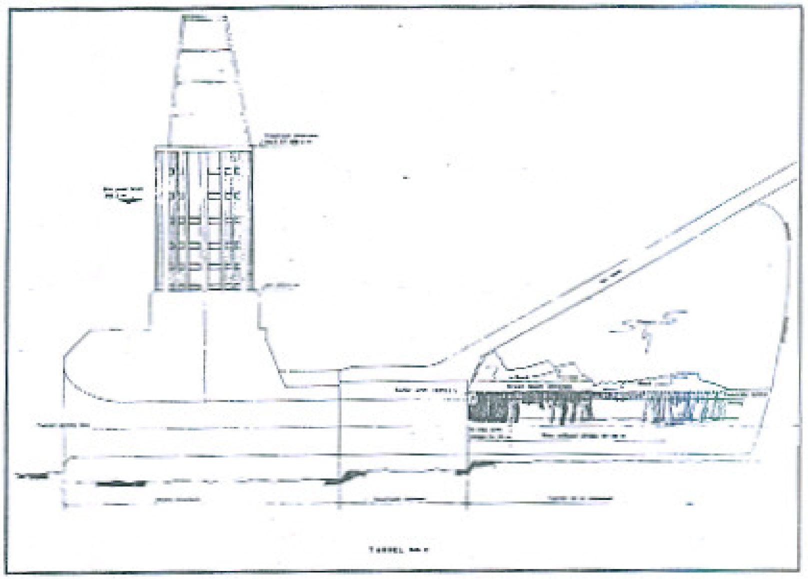
19 September
Soundings indicate deep erosion of rock between intakes 1 and 2.

problems were dwarfed when, half an hour before midnight on 21 August, a loud 'explosion' was heard in the area of the intakes and the discharge from tunnel 2 shot up from 1,000 to 4,000 cu m a second. A strong thumping vibration came from the portal area of tunnel 2, and could be felt over the whole abutment. Clearly this was the time when the 60 metre length of tunnel gave way and the rush of reservoir water began to wash away the hillside.

But at the time, with 100 m of water covering the tunnel, the cause of the explosion and its dramatic effects were not so obvious. First theories naturally associated events with the jammed gate, and reports suggested that the complex flow through the restricted opening had caused failure of an adjacent gate and the supporting pier (See *NCE* 29 August).

An immediate emergency lowering of the water level was ordered, and with the River Indus still feeding 4,000 cubic metres a second into the reservoir, this clearly meant a substantial jump in the downstream releases. Flow patterns no longer had any significance, and damage to the downstream cofferdams became a daily event as the discharge rose to 11,000 cu m/s.

It was not until three weeks later, on 14 September, that the full extent of the damage was seen for the first time. As this week's first pictures from the site clearly show, a vast chunk of the dam's abutment has been washed through a



This drawing produced by TAMS site staff for the consultant's panel shows the extent of the damage to the tunnel and the chasm in the hillside.

60 m long hole in tunnel 2. In addition the rush of water has scoured a 10 m deep hole below the foundation level of intakes 1 and 2. A fleet of lorries is presently plying concrete to the spot and tremie pipes feed down through the now steady water level.

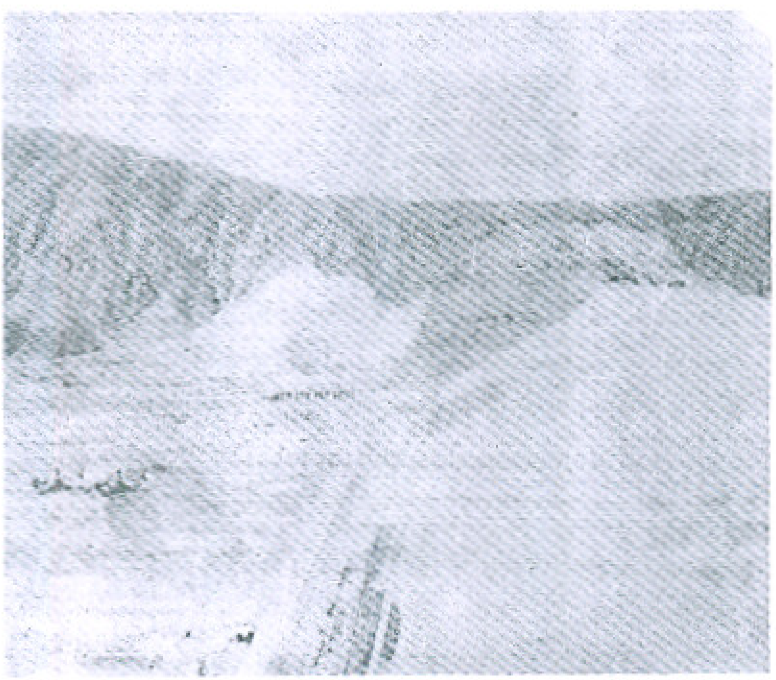
Large concrete air conduits, once supported on free draining fill placed over the natural rock abutment now span dangerously across the 75 m gap, preventing close inspection of the damaged tunnel. The Indus flow is still passing through one gate of tunnel 2 as well as the fully open tunnel 1. It is expected that another ten days or so will see the flow sufficiently low to allow closure of the final gate of tunnel 2 the explosion has apparently freed the blockage of the centre gate, which had closed when the water level fell. Experts will then scrutinise the remains of tunnel 2 for the cause of failure.

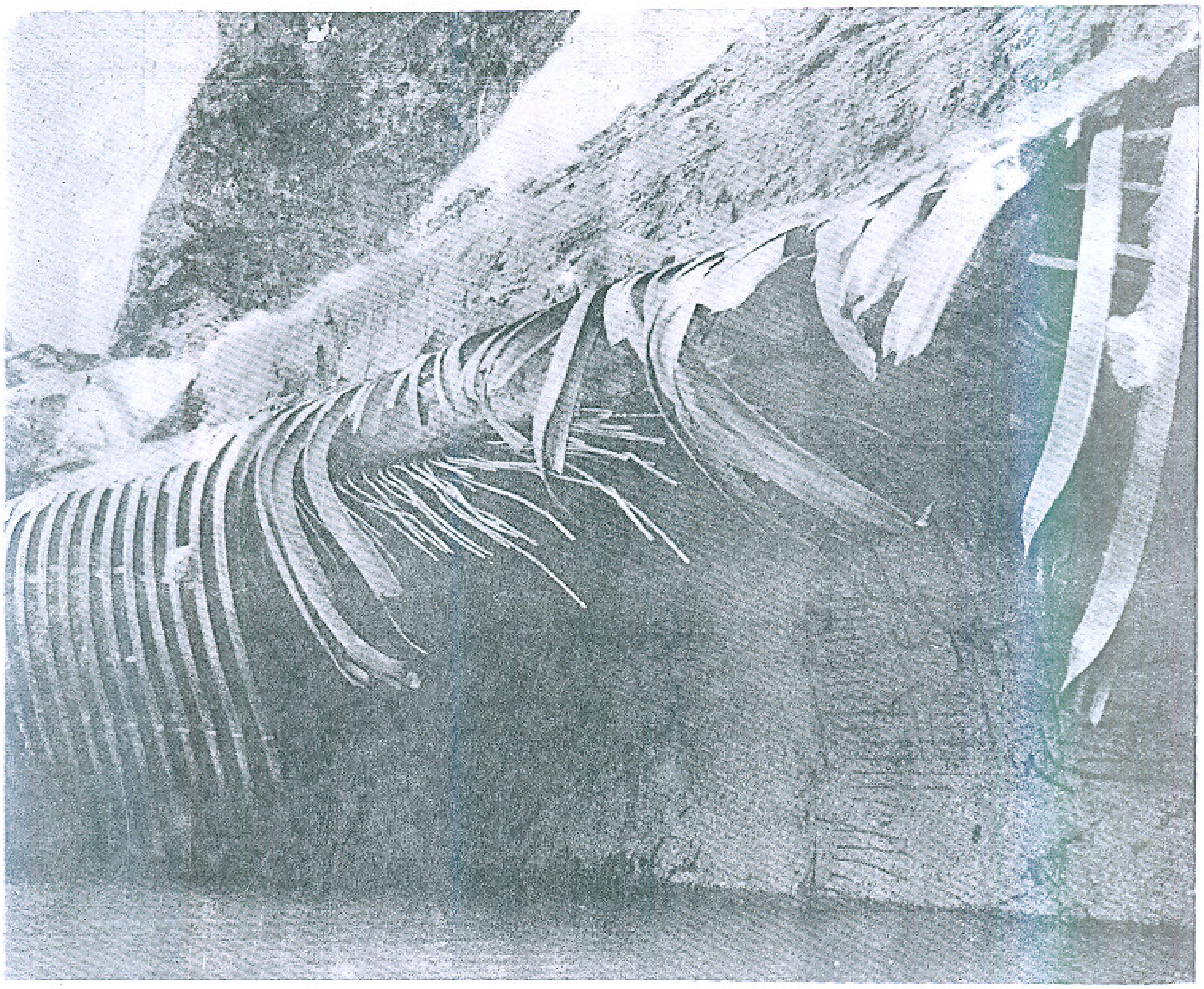
Meanwhile the consultants panel is to meet every day this week to try to determine where to go from here. Clearly the delay to commissioning of Tarbela's 700 MW power station is going to be considerable – probably well in excess of earlier estimates of one year. This would require impounding to start again in July 1975. The scale of damage revealed must make such hopes impossible.

One very important plus note to set against the chapter of disaster that has dogged Tarbela's last three months – the rapid drawdown by 100 m in 23 days is said to have caused no damage to the embankments themselves. Clearly pore pressures within the embankments have been dissipated sufficiently quickly to prevent any instability occurring as the water level fell.

This must at least provide some solace for the band of engineers who have had to react instantly to a continuous series of emergencies for three months in the face of alarmist reports about impending disaster.

A general view of the downstream face. At peak discharge in August the flow in the Indus reached 11,000 cu m/sec and washed away large sections of the downstream cofferdams around the power station at the foot of the embankment.





Collapsed area intake west face outside view of damaged ribs of Tunnel No. 2 of Tarbela Dam

April 17, 1974

September

Effect of Residual Sodium Carbonate in Irrigation Water on Soils

by

MUHAMMAD ALTAF HUSSAIN
MUHAMMAD ASGHAR and
GULZAR HUSSAIN

Effects of different levels of residual sodium carbonate in irrigation waters on soil properties has been studied under cropping conditions and the results are reported here. The waters used were fit for irrigation when considered on the basis of water quality parameters except residual sodium carbonate.

The quality of irrigation water is mostly evaluated on the basis of the soluble salts, relative proportion of sodium to calcium and magnesium known as the Sodium Adsorption Ratio (SAR) and other toxic elements especially Boron present therein. It is further assessed according to the extent of residual sodium carbonate contained in it. Certain standards of classification of irrigation waters on the basis of the soluble salts and SAR values have been established. Boron is seldom found in ground waters of the Punjab. Residual sodium carbonate, however exists in such proportion that it becomes a limiting factor to use these waters successfully for crop production. Its harmful effects are due to the alkali conditions produced by it in the soil. The limits of RSC of these waters used on agricultural lands without harm, needs careful investigation. The present work gives the results of recent experiment conducted to fix suitable limit of this factor.

Physical Chemist, Assistant Research Officer and Research Assistant respectively, Land Reclamation Punjab, Lahore.

Review of literature :

Eaton defined residual sodium carbonate (RSC) as :

$$RSC=(CO_3 + HCO_3) - (Ca + Mg)$$

He considered that the precipitation of calcium and magnesium to be quantitative with respect to the residual sodium carbonate present in irrigation waters. Wilcox etal showed that waters with RSC more than 2.5 meq/1 are not suitable for irrigation purpose. He reported the safe limit to be upto 2.5 meq/1. He also found that with RSC in excess of 2.5 meq/1, the accumilation of exchangeable sodium was significantly greater under low leaching regime than under a high one.

Kelly found that the use of irrigation water containing bicarbonates in excess of Ca and Mg may lead to relatively high concentration of sodium carbonate in the soil solution. In agricultural operation, the concentration of sodium earbonate may not be significant. Thus he stated the expression RSC of an irrigation water is a misnomer.

Bower et.al. found that the RSC concept of Eaton seems to have some merits. Massland etal stated that the concept of RSC was more valid than it was some times believed. Hameed etal has been of the opinion that waters with RSC equal to 5 meq/l could be safely used in SCAPP-1 even without mixing. Hussain and Nur-ud-Din in their laboratory studies, while using four types of waters of low and high RSC values for leaching with varying delta of irrigation, observed increase in the exchangeable sodium contents of the soil in all cases. It was, however, very significant when water having RSC 7.4 meq/l was applied.

Material and Methods :

The experiment was conducted in pots, 10″ deep and 8″ in diamter. Fine sandy loam soil was filled in them and properly packed. An outlet was provided at the bottom to allow free drainage of water. Three and a half inch space at the top was left to apply irrigation water. Each container had 10 lbs. of soil which was analysed before starting the experiment and after completing it. The waters having RSC 0.0, 2.0, 4.0, 6.0 and 8.0 meq/l were used for irrigation. Canal water (free from RSC) was kept as control. The complete analysis of the waters applied is given as under :-

Table : 1 Showing the analysis results of waters used for irrigation.

$ECx10^6$	Ca (meq/l)	Mg (meq/l)	Na (meq/l)	CO_3 (meq/l)	HCO_3 (meq/l)	Cl (meq/l)	SO_4 (meq/l)	SAR	RSC (meq/l)
300 (canal water)	1.56	0 94	0.85	—	1.49	.26	1.60	.76	—
650	1.56	0.94	3.86	—	4.50	.26	1.60	3.5	2.0
850	1.56	0.94	5.86	—	6.50	.26	1.60	5.3	4.0
1050	1.56	0.94	7.86	—	8.50	.26	1.60	7.1	6.0
1250	1.56	0.94	9.86	—	10.50	.26	1.60	8 9	8.0

In no case the $EC \times 10^6$ was higher than the permissible limit fixed for the use of water for irrigation. The maximum upper limit of SAR was less than 9.0. The crops grown were wheat and rice. Each time a measured quantity of irrigation water was applied. The irrigation to the wheat crop was given whenever it needed, but in case of the rice crop the losses of water in each container were made up almost daily. The water drained out in each case was collected and tested for its content of residual sodium carbonate.

Results and Discussion :

The analysis of the soil saturation extract before starting the experiment gave the following results :-

Table : 2

$EC\times10^3$	PH	Ca+Mg (meq/l)	Na (meq/l)	SAR
3.6	7.9	34.5	2	.46

The soil thus contained very high quantity of soluble calcium and magnesium but was low in sodium. Its sodium adsorption ratio was thus very low.

Similar analysis was conducted for soil after the completion of the experiment. The results are given in Table No. 3.

Table : 3 Showing the Chemical analysis of soil saturation extract after completing the experiment.

Treatment	Replication	EC×10³	pH	Ca+Mg meq/l	Na meq/l	SAR
I	1	2.0	8.3	16.5	2.5	.87
	2	1.9	8.3	16.0	2.5	.89
II	1	2.5	8.4	12.0	12.5	5.10
	2	2.5	8.4	13.5	12.0	4.40
III	1	2.0	8 4	10.5	18 0	7.8
	2	2.6	8.3	9.0	17.0	8.06
IV	1	2.4	8.3	5.5	17.0	10.63
	2	2.4	8.3	6.0	18.0	10.4
V	1	2.4	8.4	5.0	17.0	10.62
	2	2.5	8.4	4.5	21.0	14.00

The total delta applied to the wheat crop for each treatment was 22 inches. The rainfall received during this period was 1.59 inches, out of this crop no drainage surplus was ever received.

As far rice crop, the measured delta applied in each treatment was as under :-

Table : 4

Showing the water applied in inches in each treatment

Treatment	X R 1	X R 2	Average
I	159.82	163.8	161.81
II	152.00	151.0	151.50
III	154.00	135.0	144 50
IV	95 00	100.0	97.50
V	100.00	93.0	96,50

X Replication

In addition to this 6.76 inches rainfall was also received during this period.

The analysis given in Table No. 3 shows that in treatment No. 1 where canal water was applied, much of calcium and magnesium was washed out of the soil profile but it could not increase the SAR of the soil to disturb its properties In treatment No. II where the applied water contained RSC equal to 2 meq/l, the SAR of the soil was increased, but it did not have any significant adverse effect on the infiltration rate in the soil. However, the adverse effect has been much pronounced in all the other treatments.

The results given in Table No. 4 indicate that the total delta applied to the crop for various treatments decreased progressively with the increase in RSC of the water used for irrigation. It was due to the fall in the infiltration rate of the soil which is the logical result of alkali conditions produced with the use of residual sodium carbonate waters. These results confirm the findings of the soil analysis given in Table No. 3.

The leachate was tested for the RSC. It was found that, it was always free from residual carbonate which indicated that it was retained in the soil which would have resulted in precipitation of calcium and magnesium from the soil solution. The whole soil in the containers was thoroughly mixed before the final analysis was done after completing the experiment. If the analysis of the top 2-3 inches of soil would have been made separately, the deteriorating effect would have been more pronounced when compared with the present results. However, the results are comparable to the findings of almost all other workers except Hameed et.al. who suggested that waters having RSC equal to 5 meq/l could safely be used in SCARP-1 for irrigation purposes.

Conclusion :

The residual sodium carbonate in irrigation waters has the hazardous effect on the soil conditions. The infiltration rate of the soil is decreased as a result of alkali conditions produced with the use of such waters. In case of waters with RSC close to 2 meq/l, the effect is not significant. Such waters can safely be used provided they are fit for use when judged on the basis of other parameters evaluating their quality for irrigation.

Acknowledgement

The authors are most highly indebted to Dr. Nazir Ahmad Ex-Phycist, Directorate of Irrigation Research, Lahore, for his valuable guidance in the compilation of this article.

Thanks are also due to Mr. Zafrullah Research Assistant who worked hard to carry out the analysis work.

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Buildings
Highways
and
Bridges Section

Pile Foundations for Bridges

by

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INTRODUCTION

Throughout the ages both builders and laymen have recognised the necessity of good foundations, as did the wise man who saw that even the super-structure would resist the forces of nature better if it were founded upon the rock.

Piles, for the support of prehistoric dwellings, have been found in Lake Lucerne, and similar structures are now built in such primitive places as new Guinea.

Caesar built a pile bridge across the Rhine. When the Campanile fell in Venice in 1902, the submerged piles driven in A.D. 900 were found in good condition and reused.

A structure is founded on piles if the soil immediately below its base does not have adequate bearing capacity or if the estimate of costs indicate that a pile foundation may be cheaper than any other.

Piles may be classified on the basis of their use or the material from which they are made. On the basis of use there are two major classifications, sheet and load-bearing piles.

Load-bearing piles may be classified as follows :

- (i) Timber - Untreated or Treated with preservative.
- (ii) Concrete-Precast, Prestressed and Cast-in-situ.
- (iii) Steel - H-Section and Steel-Pipe.
- (iv) Composite.

This article deals with only load-bearing precast concrete piles as used for the construction of bridge-foundations. To our knowledge bridges in this country have always been provided with raft, well or rigid-bent types of foundations and the pile-foundations have been adopted for the first time during the construction of bridges built across the Link Canals of the Indus Basin Project. Speedy construction of hundreds of bridges on the eight Link Canals in a period of less than eight years was possible only by adoption of precast or prestressed concrete piles for their foundations. The classical well-type of foundations previously adopted ; apart from the usual construction

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difficulties, encountered during the sinking of wells ; would unduly prolong the construction period if a well was tilted or drifted away from its desired location. Similarly the shallow well foundations always needed a costly bed protection requiring recurring expenditures for the maintenance of the loose-stone aprons provided to guard against normal scour. Such bridges are never safe against abnormal scours resulting from concentrated flows.

Design of various types of load-bearing piles and the techniques for their driving have been enormously advanced in the developed countries of the world and voluminous literature is now available on this subject. The writers had the opportunity to witness the manufacture, driving and load-testing of precast concrete piles on the various Link Canals in general and the Taunsa-Panjnad Link Canal in particular and hence deemed it imperative to record this extremely useful experience for the use of future bridge builders of the country. An attempt has been made to increase the practical utility of this article by inserting working drawings and photos showing the design, manufacture, driving and load-testing of the piles. Tables have also been given for indicating the methods for recording the driving of piles, pile-load-test results ; the equipment needed for pile-driving and pile-load-testing ; and the different types of pile hammers.

The subsequent paragraphs of this paper deal with the manufacture and driving of piles ; pile-load tests and determination of ultimate loads and lengths of piles from the test results ; pile-driving formulae ; pile driving and pile-load testing equipments ; and different types of pile driving hammers.

Space in this paper does not permit inclusion of the detailed design and specifications of piles which may be taken up in a separate and comprehensive article at a latter stage.

MANUFACTURE OF PILES

In order to simplify the casting operations, all precast or prestressed concrete piles required for the execution of a project should be of the same cross-section. For exercising better quality control, the piles are preferred to be constructed in a central casting yard.

Precast concrete piles are cast on level and tight platforms constructed to prevent any settlement during casting and curing operations. The casting beds are made of brick mosonry with steel float finish in cement plaster on tops to preclude likely sticking of the piles to the casting beds. To economise in the construction of casting beds, the piles are constructed in tiers, varying from two to four piles in one tier, by casting piles one upon the other. The subsequent sequence may be followed for the construction of precast concrete piles.

1. Accurately position the reinforcement by using annealed iron wire ties at intersections, and support it by concrete or metal supporters, spacers or metal hangers.
2. Set, and bolt or clamp reusable steel forms to the casting beds.
3. Place and vibrate concrete and start water curing after initial setting of concrete.

4. Remove steel forms after 24 hours. Cover the green concrete and continue water curing for at least 14 days.

5. Place the piles in their final location after at least 28 days of casting.

For the construction of prestressed concrete piles, the following sequence of operations may be adopted :

1. String the prestressing strands and tension them to the required amount by hydraulic jacks.

2. Wrap spiral wire ties and place steel tip reinforcement for the pile.

3. Set, and bolt or clamp reusable steel forms, and place inflated rubber tubes for forming hollow pile cores.

4. Place and vibrate concrete. Initiate water curing.

5. Deflate and remove rubber core form after 12 hours, and remove metal forms after 24 hours.

6. Transfer prestressing force to cast concrete members by cutting prestressing strands between members when concrete attains a strength of 3, 750 pounds per square inch, as verified by concrete cylinder tests.

7. Transfer Cast Concrete members to storage areas and continue water curing for at least 14 days.

8. Transport completed piles to point of use by the trailer especially built for this purpose,

Prestressed or precast concrete piles of the cross-sections shown on Figure 1 were used for the Construction of various types of bridges on the link canals constructed under Indus Basin Project. Prestressed concrete piles were used only on Rasul-Qadirabad Link Canal. In view of the difficulty that was experienced in driving prestressed concrete piles, precast concrete piles were adopted on all the other Link Canals.

DRIVING OF PILES :

Piles may be driven with a hammer or with allied use of hammering and water or air jetting. Piles shall be driven as accurately as practicable in the correct location, true to line both laterally and longitudinally. Under difficult conditions, the driving of piles should be started in a hole or a guiding templet or other necessary means provided to ensure driving in the proper location.

Water jetting is frequently required for driving heavy weight and long precast or prestressed concrete piles to relieve driving stresses, secure desired penetration and save time. When water-jetting is employed the volume and pressure of water should be sufficient to freely erode the material adjacent to the pile and, then, allow the discharge to come up around the pile to the surface. The plant should have adequate capacity to deliver at lea-t 200 psi pressure at 3/4 in nozzle. Before the desired penetration is reached, the water jetting should be dissociated and at least the last five feet of the pile be driven exclusively with driving energy of the hammer.

When there are good reasons to substantiate that piles cannot be driven in the manner related in the preceding para, holes may be drilled to facilitate the driving. Where drilling is permitted, the holes drilled shall have a diameter not more than an inch larger than the tip diameter of the pile and the drilling will continue only through the strata of hard material offering inordinate resistance. Where the hard material extended below the desired penetration, the drilling shall be stopped above that penetration level and the pile finally set under normal driving by at least 50 blows from a gravity or single-acting steam hammer or 200 blows from a double-acting steam hammer.

It is, invariably, advisable to drive the piles to the full penetration as determined from pile load tests, regardless of the fact that sufficient bearing capacity, as determined from the formulae, is achieved at lesser depths.

The foundation piles of the roadway bridges, constructed in connection with the construction of the link canals under Indus Basin Project, were driven with combined operations of hammering and water-jetting except those of Rasul Qadirabad Link where the pre-drilling of the holes of almost pile diameter had to be adopted to obviate the breakage of the hollow prestressed concrete piles.

On Taunsa-Panjnad Link Canal, all the piles of the roadway bridges were placed successfully by the combined operations of water-jetting and hammering, as illustrated in connection with the driving of test and anchor piles in the section of pile load test. The contractor deployed two sets of 4850 pound ram (piston) Delmag D-22 hammers attached to leads, having tracks in which hammers were engaged for the full length of travel, fixed at the face of the booms of P & H cranes. The handling and lifting of the piles were carried out by means of ropes passing over the pulleys mounted at the top of the lead and controlled from the electric-operated winchinstalled on the crane. Besides, two stage centrifugal pumps driven by diesel engines, jet pipes, and hose pipes were used for the execution of water-jetting. The placement of 308 piles, which virtually means the construction of piers for 14 bridges, was carried out in a period of about one year.

The subsequent succession of operations were generally adopted for the driving of 18-inch square precast concrete piles on the Indus-Basin Link Canals.

1. Steel-girder template, for fixing location of the pile in the pile-bent and guidance in side-jetting, was installed in position by long steel pegs.
2. The pile was lifted and held vertically along the face of the lead attached to a crane standing near the final location of the pile, and hammer was centered over the pile so that the blow would be entirely axial and pile would not be driving out of the specified lines and grades.
3. Prior to driving of piles, a defined hole, having depth about 5 feet less than the requisite penetration

of pile, on the final location of pile was formed by means of 3 inches dia, jet-pipe having 3/4" dia, nozzle and discharging water at about 200 psi.

4. Soon after the completion of satisfactory formation of hole, the pile was dropped vertically and freely into the hole.
5. Immediately after the dropping of pile, hammering was done with full energy of driving till the penetration of pile encountered considerable driving resistance.
6. To ease further driving and obviate the breakage of pile, water-jetting on either side of the pile was done, and hammering was resumed for further driving of pile.
7. To seat the pile firmly in its final position, the driving of its last five feet was carried out exclusively by means of hammering To ensure adequate load carrying capacity of the pile, it was observed that under the last 30 to 40 blows the penetration of pile was less than one inch.
8. After the cessation of driving, the hole formed by jetting was filled in layers with reasonably compacted good sandy soil. The arrangements for driving of piles and performance of pile-load tests is shown on the photographs appearing on the subsequent pages.

Pragmatical Difficulties and Remedial Measures.

On Taunsa-Panjnad Link it was experienced, during the driving of piles at A.R. Bridge RD. 142+00 that after 30 to 40 feet depth of jetting the discharge of the jetting water ceased to come up to the ground surface, and accordingly, the soil displaced by water stopped escaping from its position. Consequently the formation of hole beyond such depths became impossible. To secure the upward flow of jetting discharge along with the soil, a jet of compressed air in conjuction with the water jet was introduced. After the introduction of air jet, the jetting-water started emerging back upto the ground surface. In this way the jetting of the hole upto the required depths could be materialized.

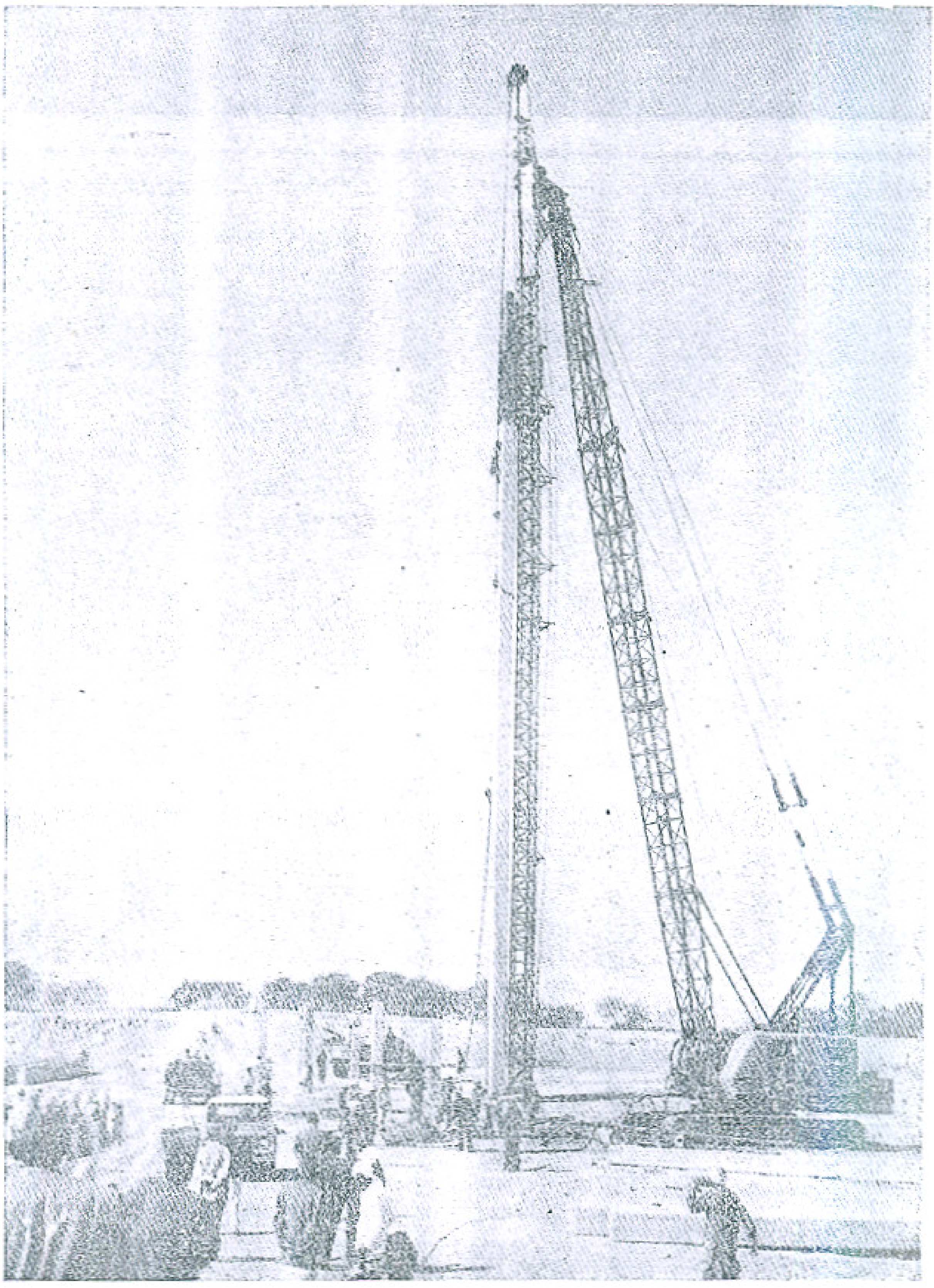
The driving resistance of the pile was significantly minimised by the air-jetting and the progress of placing the piles was appreciably increased. But, paradoxically enough, the air jetting displaced the soil around the piles very disastrously, and resulted in the formation of bigger holes. The piles, accordingly, had easy way to deviate from their correct location during driving. In view of this difficulty the air-jetting was discontinued and only water jetting, with increased pressure and amount of the jetting discharge was used. The formation of bigger hole with the air-jetting could be controlled, by adjusting the optimum intensity of the air pressure, but it could not be arranged by the contractor.

On Rasul Qadirabad Link, 22 - inch Octagonal prestressed concrete piles, with 13 inch diameter hollow core and ten one-half inch dia., prestressing strands, were used for founding the roadway bridges. For the execution of the pile load-test, the anchor and test pile were driven with the normal means of hammering and water-jetting, but all the three piles developed cracks during driving. Following completion of the final load test, the test pile was extracted and was found to have two 15-foot vertical 1/32 of an inch wide cracks starting just above the pile tip. Numerous hair-line cracks and spalling were also found near the top of the test pile. In view of the foregoing damage of the piles, the contractor was directed to improve his pile jetting and driving equipment and procedure in order to preclude pile breakage.

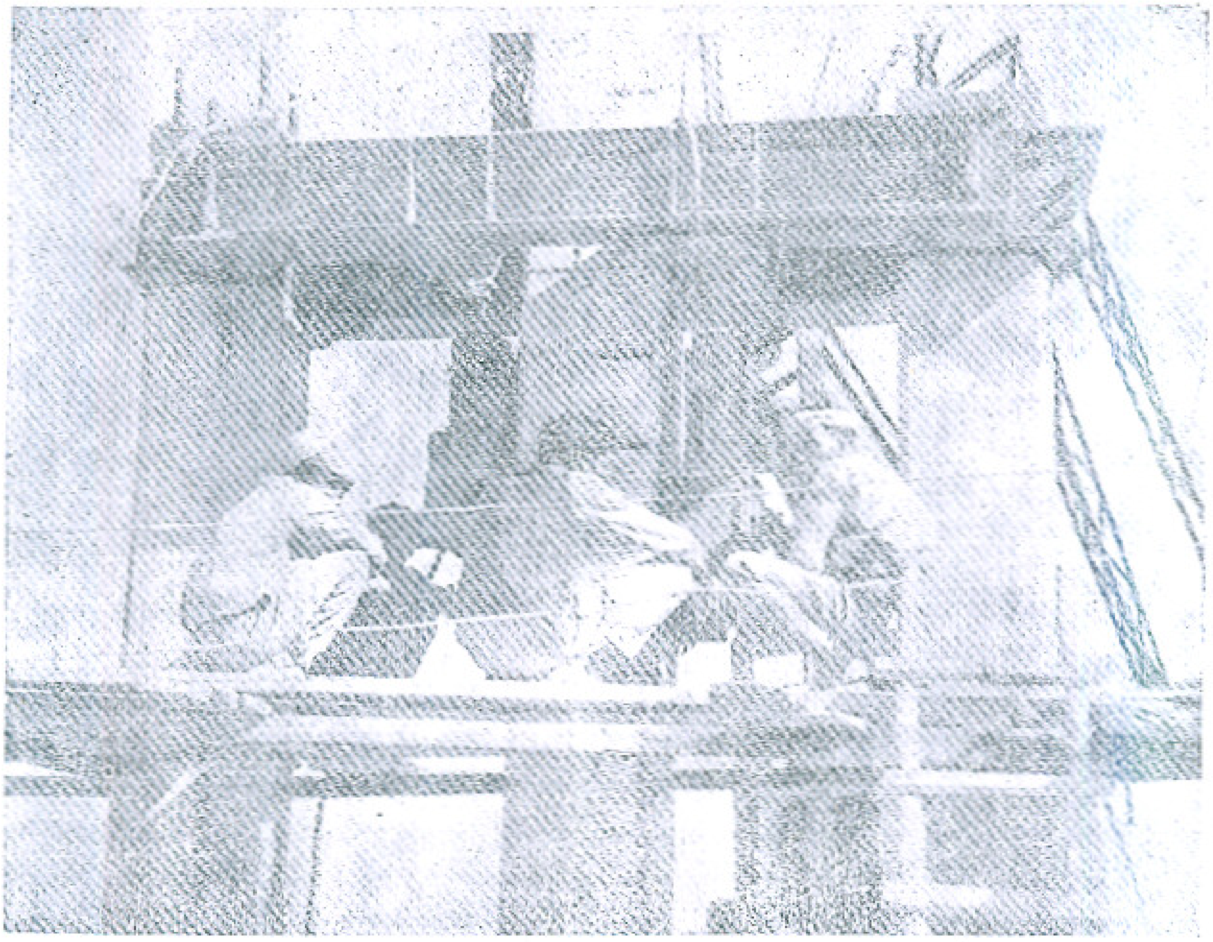
The contractor proposed to drastically alter his pile driving procedures in an effort to overcome the problem of pile breakage. The contractor proposed to predrill a hole of approximately the pile diameter to within a few feet of the final pile tip elevation using reverse rotary drilling equipment. They proposed to place the pile in the predrilled hole and then seat the pile by driving it to its final position. In order to demonstrate the effectiveness of the revised pile placement method, the contractor offered to conduct a series of pile tests about 1,500 feet down the link from the location where the previous tests were made. Test piles were placed in a 20 inch hole to 41 feet of penetration, and in a 24 inch hole to 54 feet of penetration, respectively. It was found that the required driving effort was not appreciably reduced using the predrilled 20 inch hole and the test pile was cracked during driving. High load bearing values were, however, obtained from the test pile placed in the 20 inch predrilled hole. The required test pile driving effort was substantially less in the predrilled 22 inch hole, and load bearing value obtained approximately equalled those obtained by placing pile by the previously used jetting and driving method. Driving effort was greatly reduced using the 24 inch predrilled hole, but test results indicated that the pile skin friction resistance was greatly lowered. It was concluded that the use of the 22 inch predrilled hole was acceptable, and the contractor was notified that pile placement could be continued using the proposed predrilling method.

The use of the predrilling method of pile placement substantially reduced, but did not entirely eliminate, the problem of pile breakage. Also, difficulty was experienced using this method where clay layers were encountered in the upper strata of the soil. Following the drilling of the hole for the placement of the pile, the clay tended to squeeze in to the predrilled hole causing high resistance to pile penetration. This difficulty was overcome by predrilling a 28 inch hole down to the canal bed elevation and predrilling a 22 inch hole from that point down. A few piles cracked during driving, despite the predrilling of the hole to facilitate pile placement. Cracking was evidenced by the appearance of ground water in the hollow centre of the pile. Where this occurred, the water was pumped out and the pile was filled with concrete to a point above which cracking was known to have occurred.

Because of the difficulties encountered in placing the prestressed concrete pile on the Rasul-Qadirabod Link, the pile supported structures were carefully observed after they were placed in service. These obser-



Driving of precast reinforced concrete piles for constructing a village road bridge on Taunsa-Panjnad Link Canal



Pile-load test being performed on a link canal bridge vations confirmed that the piles performed their function satisfactorily.

On Chasma-Jhelum Link Canal, the contractor proceeded to drive two anchor piles and a test pile and made three tests at RD. 127+222. He was then furnished pile lengths for all bridges from RD. 93+222 to RD. 191+682 on the link. The contractor encountered hard driving and difficulty in meeting the required pile position tolerances during his initial pile driving. He, therefore, proposed changing his method of pile driving. In stead of the original jet-assisted pile driving, he proposed a predrilling method of pile driving much like that used on the Rasul-Qadirabad Link. He proposed to drill a 22 inch diameter hole to within 3 feet of the pile tip elevation, to drive the pile with jetting in the predrilled hole, to drive it to its final position, and to then post-jet around the sides of the pile to ensure that the skin friction resistance of the pile was fully developed. The contractor proposed to immediately make the second field pile test to demonstrate that the revised placement method would give pile lengths equivalent to those obtained using the initial method of pile driving. The contractor made the second pile test at RD. 42+222 using the predrilling, driving, and post-jetting method. The satisfactory results obtained from this test enabled the Engineer to give the contractor pile lengths for bridges along the upper third of the link, from RD. 18+947 to RD. 87+222.

The contractor experienced further difficulty using the 22 inch predrilled hole and requested permission to enlarge the predrilled hole to 24 inches. He offered to make two additional field pile tests at RD. 42+222 using a 24 inch predrilled hole and post-jetting. These tests, showed that use of a 24 inch hole did not adversely affect the load-bearing capacity of the pile, and use of the larger hole was approved. Pile driving then continued without further difficulty.

Pile Load Test

Before performing a pile load test for adopting pile foundation one must bear in mind that a satisfactory pile-foundation must have a sufficient ultimate bearing capacity (i.e , that soil shear strength must be adequate) and that settlement under working load must be tolerable. With this elemental consideration in view, the pile load tests are carried out in the field at representative sites of the structures to be founded on piles for the determination of the following :-

(a) The ultimate-load capacity of test-pile.

(b) What part of the ultimate-load capacity of the test pile is developed by skin-friction and what part is developed by that of point-bearing.

(c) To obtain data for use in making up a schedule of pile length for the entire structure or all the structures, as the case may be of the project under consideration.

Methods of Making Pile Load Test :

The major difficulty in carrying out the pile load test is the application of the loads to the test pile. The test pile, however, may be subjected to loads by means of a wide variety of arrangements of direct and indirect application of loads. The direct application of the loads may be carried out by loading a platform, especially constructed on the top of the pile, with heavy loads in the form of earth or sand bags, pig irons, water tanks, having arrangements for filling and releasing of water, or any other form of load which may be manoeuvred with the greatest economy of the readily available materials and shortest consumption of time. The indirect application of the loads to the test pile may be manipulated by jacking against an existing structure or piles especially driven for this purpose.

The methods of the direct application of the loads to the test pile are quite conservative and cumbersome as they involve handling of quantities of the materials which, consequently, results in considerable wastage of time. Jacking against fixed jacking-platforms is, therefore, preferable. Furthermore, another advantage of jacking is that the loads can be applied and released quickly, as and when required, permitting immediate determination of the permanent settlement or penetration of the test pile in the substrata, after the rebound of the pile has occurred.

The application of the loads in all the pile-load tests, carried out on the link canals constructed under Indus Basin Project, was executed by jacking against a jack-beam connected with the exposed reinforcing bars of the adjoining anchor piles. A comprehensive description of the arrangements and procedures of the pile-load test is related below, with particular reference to the tests carried out on Taunsa-Panjnad Link Canal for better perspicuity of all the operations and difficulties which may arise during actual performance of such like tests.

Arrangements for Pile Load Tests.

For the economical and efficient performance of the pile-load test and the subsequent considerations the following arrangements were adopted :-

1. The preliminary investigations revealed that geological conditions of the sub-strata along the entire link were almost homogeneous. Accordingly, the pile load tests were carried out on the following two sites, for penetrations mentioned against each, to determine the actual length of piles required to withstand the loads for which bridges had been designed.

(i) Village Road Bridges at RD. 39+380

(ii) Village Road Bridge at RD. 72+425

Two tests for 45 & 55 ft. penetrations.

Three tests for 45, 55 & 65 feet penetrations.

2. Two anchor piles each 90 feet in length, and a test pile of 75 feet length were constructed at each of the test site. The length of the test pile may vary, as required, to provide the specified penetrations. The test pile was provided with three sets of holes, at right angles to the centre line of the pile, required for fixing the jack-blocks to the test pile in different positions. The spacing between these holes depends upon the height of the jacks to be used for applying measured incremental loads to the test pile. The number, size, location and spacing, as used on Taunsa-Panjnad Link Canal of these holes is shown on Figure 2.

3. The initial load test was made at the site of village road bridge at RD. 39+380. Two anchor piles, each 90 feet in length were driven 8 feet apart at the permanent locations of piles of one of the intermediate piers to make their use in the final structure. The 75-foot test pile, which was second in the process of driving, was placed in the centre of the two anchor piles. The placement of these piles was carried out by the combined continual operations of water-jetting and hammering.

To create favourable conditions for maximum free-fall of the pile, initial jetting was done upto a depth of 80 feet. After having a free fall of only 14 feet, a great resistanse was encountered in its driving. Although heavy jetting and full driving energy were used, the head of the pile was sheared off only at the 30th foot of its driving. The pile head was cut smoothly and further driving resumed with the assistance of efficient hammering, and water jetting on sides. On 76-77 feet of its driving the pile head again got crushed and, consequently, further driving of this pile was discontinued. With combined operations of jetting and hammering the test pile was, then, driven up to the depth of 45 feet which was the proposed penetration for the first test. The second anchor pile, was also driven upto the refusal depth of 75 5 feet. The driving of piles for the second and the subsequent tests was carried out with similar operations of jetting and hammering.

A complete record of the driving operations of each pile was kept. This record indicates make and model of hammer, location of pile, free-fall, that is depth penetrated without driving, elevations and times at which jetting or hammering was commenced or stopped, and the number of blows for every foot of penetration during driving, and number of blows for the last inch of penetration before stopping further driving, elevation of top of pile when driving was completed and other pertinent data of interest to the interpretation of the results of these tests. The pile-driving record of test pile is shown, as a specimen, on Table-1.

4. A steel jack-beam was attached to the anchor piles by welding the bolts passing through the jack-beam with the exposed reinforcing bars of these piles, in a manner which permitted application in increment of measured loads to the test pile either in a downward direction for determining load bearing capacity or in an upward direction to measure skin-friction resistance. For downward application of loads, the hydraulic jacks were placed underneath the jack-beam, to permit jacking against the jack-beam, on the jack-blocks bolted with the test pile through the holes already provided in the test pile for this purpose.

The arrangements for jacking in order to apply load to the test-pile in the upward direction are mentioned in the section of "Pulling Test". Figure 2 shows assembly of the arrangements for pile-load test. Very sensitive deflection gauges, capable of measuring upto one thousandth part of an inch, were firmly fixed to the test and anchor piles to measure their settlement or uplift during the various operations of the test.

Test Procedure.

In order to permit the piles to restore adequately their-point bearing capacity and skin-friction resistance, which were considerably reduced due to the disturbance of the soil particles during the operations of pile driving, all the pile-load tests were carried out at least four days after the completion of pile driving. The test loads were twice the design load on the individual pile, arrived at in accordance with the classification of the bridges, the locations of which were elected for the performance of the tests.

The loads adopted for various pile-load tests are tabuiated below :

LOCATION	DESIGN LOAD	TEST LOAD
(i) Village Road Bridge-cum-Aqueduct at R.D. 39+380	130 Kips	260 Kips.
(ii) Village Road Bridge cum-Aqueduct at R D. 72+425	130 Kips	260 Kips.

The sequence of operations adopted for the performance of test was as follows :

The first increment of load applied to the test pile was the pile designed load. The load on the pile was increased to the test load by supplying additional loads in four equal increments. A minimum duration of two hours was observed between the application of each increment. The full test load was, however, maintained on the pile for forty-eight hours. For each increment of load, the total settlement of the top of the pile, while under load, was recorded ; then the load was released for 15 minutes and thereafter the total permanent settlement of the top of the pile, with no load, was recorded. The deflection gauges clearly revealed the rebound of the pile after the release of the heavier loads. After the release of the test load and the ultimate load, the readings of the deflection gauges were recorded till they showed no movement in the upward direction. The difference of the readings of the deflection gauges, fixed with the test pile, noted after the pile had fully rebounded and the reading noted initially prior to the application of loads is identified as permanent settlement. In case when full test load caused more than one quater inch of permanent settlement in a period of 48 hours of continuous application of the test load, the test pile was considered to have failed the tests for that particular penetration, indicating that longer piles were required.

If the pile did not fall under the test load, the pile was further loaded in 10 Kips increments to determine the test pile maximum-load capacity which is the maximum-load on the test pile that does not cause a total settlement (with load) of more than an inch or a permanent settlement (after the releasing of load) in excess of one-quarter of an inch when the load is maintained for forty-eight hours. It is observed that maximum-load capacity, that indicate failure load of test pile, is, invariably, in excess of the test-load which is twice the pile design load. The settlements of test pile, as observed during the execution of the pile-load test for 55 feet penetration, are abridged in Table 3. A typical form for recording observations of pile-load test is shown in Table 2. The load-settlement curves, for penetrations of 45&55 feet, are shown on Figure 3.

Pulling Test

To evaluate what part of the total ultimate capacity of the test-pile was developed by skin-friction and what part was developed by point bearing, pulling test was carried out. Jack-blocks were fixed with test-piles, by means of bolts passing through the holes already provided in the pile, above the jack beam and the jacks were placed directly on the top of jacking beam in order to apply the load in the upward direction. With tops of the outer two anchor-piles still connected to the jacking beam, the test pile was jacked, upwardly. The load indicated on the jacks when the test pile yielded perceptibly in the upward-direction, reudced by the weight of the pile, after taking the effect of bouyancy into consideration, determined the ultimate amount of skin-friction resistanced eveloped by the test pile. The pull uplift curve, for 55 feet penetration, as shown on Figure 4, is also similar to the load settlement curve.

The ultimate test-load on the test-pile reduced by the ultimate amount of skin-friction developed by the test-pile, as determined above, gave the ultimate amount of point-bearing developed by the test-pile.

Determination of Ultimate Load from Tests

Ultimate bearing capacity of a pile may be defined as a load beyond which the pile indicates perceptible penetration into its bearing strata. Many arbitrary or empirical rules have been used or are contained in different codes to serve as criteria for ascertaing the ultimate load and finding the gross/permanent settlements from the load test observations. Conclusions drawn from some of the rules recommending final allowances of settlements for satisafactory pile-load test are summarized as below :

S. No.	Name of Authority	Description of Criteria
(1)	(2)	(3)
1.	Department of Public Works, State of California.	Gross settlement, including elastic deformation of pile, is not more than 0.01 inch per ton of test load.
2.	American Association of State Highway Officials (A.A.S.H.O.)	Permanent settlement under test load, after a continuous application for 48 hours, must not be more than 0.25 inches.
3.	Building Laws of the City of New York.	The total permanent settlement under test load maintained for 24 hours should be less than 0.01 inch per ton of total test load.
4.	Dr. R. L. Nordlund, Raymond Concrete Pile Company.	Gross settlement not to exceed 0.05 inch per ton of additional load.
5.	Louisiana Department of Highways.	Permanent settlement not to exceed 0.25 inch after 48 houts under test load.
6.	Bureau of Bridges, State of Ohio.	Gross settlement not to exceed 0.03 inch per ton of additional load.

The terms of total or gross settlement and permanent settlement, as envisaged in these rules may be defined as follows :-

Permanent Settlement :

The settlement of the pile observed when the rebound of the pile has completely taken place, after releasing the test load or ultimate load after 24 or 48 hours as suggested by the respective code, with reference to the original top elevation of pile is identified as permanent settlement.

Gross or Total Settlement :

The settlement of the pile, including the elastic deformation of pile, observed after continuous application of test load or ultimate load for 24 or 48 hours, as suggested by the respective code, is termed as gross or total settlement of the pile. This settlement will also be referred to the original top elevation of the pile.

Rule No. 2 that is the criterion of the American Association of State Highway Officials (A.A.S H.O), which recommends that permanent settlement should not exceed one quarter of an inch, under test load or ultimate load, has proved very rational and justifiable, and has been followed for determining the ultimate load capacity in all pile-load tests executed on the various link canals constructed under the Indus Basin Project. The perusal of summary of observations for test for 55 feet penetration, as given in Table 3, indicates the pile approaching failure at permanent settlement of 0.26 inches, which is very near to 0.25 inches, under load of 120 tons.

Determination of Pile - Length :

The embedded length of precast concrete pile or prestressed piles required to adequately support roadway bridges and aqueducts can be determined from the design curves derived on the basis of pile-load tests made at the site. The design curves are plotted using point bearing capacity and skin-friction resistance of the piles as determined in the field from pile-load tests. Pile lengths for each structure are then derived from the design curves which include a minimum of safety factor of two and a sufficient scour depth allowance. Pile lengths for the roadway bridges and aqueducts of Taunsa-Panjnad Link were derived from the design curves, prepared from the data of the pile-load test executed in the field, which included a minimum factor of safety of two and a scour depth allowance of 12-feet, to resist the theoretical load which the structure would be subjected to in accordance with the loading criteria summarized in Column (6) Figure 3. The effective load-carrying length of piles was considered to be the length of pile below the design scour level at each structure. Abutment piles extended to the same level as the other bridge piles, 18 inches extra length, as an allowance for embedment into the pile cap, was provided with each pile. A design summary correlating ultimate load bearing capacity of the piles, as determined from the load test curves, with the applicable design loads computed for each structure is given on Figure 5.

Recommendations and Conclusions

1. Test piles shall be of the same size and materials as the permanent piles and shall be driven with the same equipment and in the same manner as specified for permanent

piles. Test piles shall be driven and test carried out in advance of the construction of the permanent piles so that lengths for casting may be determined.

2. A comprehensive record of pile-driving equipment and operations, like that of hammer, water-jetting and penetrations, must be vigilantly maintained ; for such records are of paramount significance in deriving conclusions from pile-load tests.

3. During the operations of jetting and driving, the natural soil conditions of stratum surrounding the piles are sufficiently disturbed; and as a consequence, point bearing capacity and skin-friction are substantially reduced. Occasionally, it has been observed that bearing capacity of piles in sand decreases conspicuously during the first 2 to 3 days after driving. To avoid the adverse effects of this phenomenon, pile load test should be commenced after at least 72 hours of the driving of the piles.

4. In this method of pile-load test, the pull if any, on the anchor piles will reduce the settlement of the test pile. It is, therefore, quite significant that during the performance of the pile load test the movement of the two anchor piles should be checked very frequently under each application of load to verify that the anchor piles are not yielding upwardly. Conversely, the downward settlement of the anchor piles would effect the uplift of the test pile during the pulling test. In view of the foregoing facts, the anchor piles should be driven to the maximum possible depth to preclude their movement under the application of various loads during pile-load tests

5. A minimum period of two hours should intervene between the application of each increment of load except that no increment should be added until a settlement of less than 0 0005 of inch is observed for a 15 minutes interval under the previously applied increment.

6. During each application of load, the load must be kept constant.

7. The test load or ultimate load must be maintained on the pile for at least forty-eight hours continuously.

8. If there is a question whether the test pile will support the test load, the load increment should be reduced by 50% in order that a more closely controlled failure curve may be plotted.

9. To avoid anomaly of stresses in the cross-jack-beam, the hydraulic jacks should be equally operated while increasing or decreasing the loads.

10. Rigid type of steel frame arms, to be connected with the deflection gauges, must, be tightly fixed with the piles and the gauges kept pretty clean as the deposition of dust particles etc , may affect their efficiency.

11. While taking final-penetration reading the number of strokes of hammer per minute should always be noted on pile-driving reports, particularly if for an unavoidable reason the speed is less than the maximum specified, for if the speed has fallen off and is not noted, the energy is reduced and the smaller penetration obtained will indicate falsely high bearing values.

12. The general geological history and careful examination of the boring results

of the entire site deserve special attention before preparing the pile length schedule on the basis of pile-load tests. It is difficult to be sure of safe results if the character of the strata along the pile and below the point are not known.

Pile-Driving Formulae :

Where possible, test piles shall be driven and loading tests carried out before construction is taken in hand. If the execution of these tests is not possible or economical, the approximate bearing capacity of piles may be determined from the following formulae :

Engineering News Formula :

The Engineering News Formula was initially derived to be used with drop hammers. This formula, later on, became very popular in United States of America. According to this formula :

$$R = \frac{2 \; Wr \; H}{S + C}$$

where R = Ultimate load capacity of pile in pounds.

Wr = Weight of striking parts of hammers in pounds or in the case of double-acting hammers, the weight of the striking parts of the hammers plus the area of the piston in square inches times the pressure on the piston in pounds per sq. inch.

H = Drop of hammer or piston stroke, in feet.

S = Average penetration, per blow for the last 5 to 10 blows of gravity hammers, or for the last 10 to 20 blows of steam or air hammers in inches.

C = A constant to allow for energy losses depending upon the type of hammer, taken as 1.00 for drop hammers and 0.10 for steam or air hammers.

On account of the shorter interval between blows which presumably allowed less chance for set up between the soil and the pile, the author of the formula reduced the value of C from 1 to 0.1 in case of single/double acting steam hammers.

The foregoing formula rewritten for allowable load after incorporating safety factor (Fs) of 6 will be as follows :-

Drop Hammers :

$$R = \frac{12 \; . \; Wr \; . \; H}{S + 1}$$

Single acting Steam or Air Hammer

$$R = \frac{12 \; . \; Wr \; . \; H}{S + 0.1}$$

Double and differential-acting Steam Hammers.

$$R = \frac{12 \; H \; (Wr + Ap)}{S + 0.1}$$

Assuming the allowable load on the test pile is 50% of the above determined ultimate capacity, the proper value of Fs is computed. This formula with proper value of Fs, substituted therein, is used in determining what the value of S (as defined above) has to be the result in an allowable or design pile load (Ra). Accordingly it can be determined at what depth of penetration the test pile developed the required design from the pile driving resistance diagram.

In this formula the losses due to impact are neglected and the elastic losses in cap. pile and soil are not given full consideration.

Furthermore, ultimate bearing capacity given by this formula is also independent of depth. Experience has shown that ultimate bearing capacity of friction piles is directly proportional to the length of the piles. This fact strictly excludes the application of this formula to friction piles in soft silt or clay. Besides these limitations, this formula contains no term to express properties, length and weight of pile, although it is certain that some of these properties have a prominent influence on the effect of the hammer blow.

"D" Type Delmag Diesel Piling Formula :

This formula, as given below, contains all necessary terms lacking in the Engineering News Formula and may be used for Delmag Diesel hammers. Resistance of pile to penetration is given by :-

$$W = \frac{ER}{167.5\,(t + C.L)\,(R + Q)} = \text{short tons}$$

where W = Resistance of pile to penetration in short tons (2000 lbs)

E = Potential energy of impact in ft. lbs.

R = Weight of hammer in lbs.

Q = Weight of pile in lbs.

t = Penetration of pile under the last blow in inches.

C = Springness of pile and earth in inches/1 - ft length of pile — 0.0036 for concrete and steel pile and -- 0.0072 for timber piles.

L = Length of pile in ft.

t = is determined from the penetration T under the last 10 blows, so that t = T/10. Due to higher frequency blows of diesel hammers they remain economical in operation even if the penetration under last blow drops to 0.02" . . . . 0.04".

The foregoing formula is developed by the manufactures of "D" Type Delmag Hammers. These hammers are one of the most efficient hammers used in the field of pile driving.

Practical experience, however, has shown that the application of all such formulae, give sometimes so approximate results which can hardly be relied upon for big projects. Hence, on large jobs good engineering practice calls for determining the ultimate bearing capacity by means of load tests on full-sized piles.

Pile Driving and Testing Equipment

The most suitable type of pile-driving plant depends much on conditions such as the number, length, spacing, and weight of piles ; depth of excavation, cribbing, or water. Availability and cost of purchase of rental may be a factor in selection of hammer and rig.

The selection of proper type and weight of hammer is of substansive significance for any pile-driving project. Piles shall be driven with the heaviest hammer that can be used to secure maximum penetration without appreciable damage to the piles.

Pile - Driving Hammers

The drop hammers can be improvised and used for driving the piles of shorter lengths. The drop hammer consists of a weight which is raised by rope running over the top of a pully mounted on framework ; and extending to a drum or geared shaft. The hammer is operated by releasing the drum to allow the rope to be unwired. Where a drop hammer is used the striking ram shall weigh not less than 3,000 lbs. The fall shall be so regulated as to avoid injury to the pile.

Steam or Air Hammers

Steam hammers are of three different types. The hammers in which the steam or air is used to raise the striking part which thereafter drops by gravity only are called single-acting steam hammers; while the hammers which employ steam or air to raise the striking part and also to impart additional energy during the down stroke are classified as double acting steam hammers. The third type, that is, differential-acting steam hammers also employ steam or air to raise the striking part and impart additional energy during down-stroke. This type of hammers are claimed by the manufacturers to combine the advantages of both single acting and double acting hammers. The range of ram weight is the same as for single acting hammers, while the number of blows a minute approach those for double acting hammers. Double-acting and differential-acting hammers should be run at full listed speed, since the net available energy at the pile tip falls off rapidly at lesser speed, particularly in the case of undersized hammers. Steam/air hammers have substantiated good performance on the following projects in West Pakistan.

Mekiernan-Terry, Model 9B-3, double-acting, air operated, were used for driving 18 inch square precast concrete piles on Trimmu-Sidhnai and Qadirabad-Balloki Link Canals. Air operated, double-acting Mekiernan-Terry type-C5 was deployed for driving 22 inch octagonal prestressed piles on Rasul-Qadirabad Link Canal. Air-operated, single-acting, Vulcan Model 08 type hammer was also used for driving precast concrete piles on Trimmu-Sidhnai Link Canal. Air operated Mekiernan Terry Model 9B-3-MKT hammer was used for driving 22 inch square precast concrete piles on Chasma-Jhelum Link.

Diesel Hammers

Diesel hammers can be advantageously deployed for driving heavy weight precast or prestressed concrete piles. Principal makes on the United States market are the Delmag, Link-Belt speeder and Mekiernan-Terry. These hammers weigh considerably less than steam hammers, are free of the need of boilers or compressors, are more mobile; use only a gallon or two of fuel per hour, have less maintenance, and operate at low temperatures. Diesel hammers, all the same, being basically single-acting depend on a long drop. Length of Delmag and Meckiernan-Terry diesels compare favourably with correspondingly rated steam/air hammers but are larger than double-acting hammers and also require space for the rising of rams. Link-Belts, being closed on the top, are somewhat shorter in length. Diesel hammers, however, are not fast, although Link-Belt are considerably faster than Delmag or Mckiernan-Terry hammers.

The length of strokes is proportional to the pile resistance. Hence for increased driving resistance, the effectiveness of the hammer will be increased. This is particularly true for the Delmag and Mckieranan-Tarry hammers, which depend entirely on ran impact for ignition ; in this fact also lies the greatest weakness of diesel hammers, namely, difficulty of rating the energy and operating in soft driving. Delmag D-22 type hammers have been successfully used for driving 18 inch square precast concrete piles, varying from 70 to 90 feet in length, for the construction of all types of bridges on Qadirabad-Balloki, Sulemanki II and Taunsa-Panjnad Link Canals. 22 inch octagonal prestressed concrete piles used for founding roadway bridges on Rasul-Qadirabad Link, were driven with Link Belt Model 520.

In the selection of a pile-driving hammer, the ideal solution is a small, light, self-contained and self activated hammer. The diesel hammer is bidding for this field.

Operating data for some hammers popular in the field of pile driving is given in Tables 5 through 8.

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PAKISTAN ENGINEERING CONGRESS

LAHORE.

TABLE - I

PILE DRIVING RECORD

Project :—Taunsa-Panjnad Link Canal.
Location R.D. 29 + 766.75 • Date : 4.4.1968
Structure A.R. Bridge. Location of Pile in Bent. D/S Ist Pile
Bent No. R/S Abutment. Length of Pile : 85 ft.
Pile Serial No. 3
Type of Driving Hammer Delmag D - 22
Initial Depth of Jetting. 85 ft. N.S.L. 438.75

Template R.L. = 444.15

Time	Feet	Blows per Foot	Remarks
14.48	—	—	Initial water jetting started
16.15	85	—	Initial water jetting stopped
16.18	—	—	Pile dropped.
	55	—	Free Fall.
16.19	60	—	Hammering started.
	62	5	
	63	14	
	64	20	
	65	24	
	66	25	
	67	53	
16.23	67.5	35	Hammering stopped.
16.26	—	–	L/S Jetting started.
16.45	—	—	R/S Jetting started.
16.58	68	10	Hammering started.
	69	14	
	70	25	
	71	31	
	72	38	
	73	46	
	74	57	
	75	60	
	76	62	
	77	60	
	78	60	
	79	55	
	80	55	
	81	56	
	81.75	40	
17.13	—	—	Hammering stopped.
17.17	—	—	Hammering started again.
	82	23	
1	82.25	65	
17.20	—	—	Hammering stopped.

Note :—Hammering stopped because the pile refused further penetration.

Incharge Overseer.

TABLE - 2 PILE LOAD - TEST RECORD

Project :- Taunsa-Panjnad Link Canal
Location : V.R. Bridge-cum-Aquednct RD. 39+380
Make and Model of Pile-Driving Hammer Delmag D-22
Test Pile Tip Below N. S. L. 55 feet

Date of Commencement 29-8-68
Date of Completion 1-9-68
Observation Recorded by Ghulam Hussain
Observation checked by S. N. H. Mashhadi

Description of Loading	Gauge Reading U/S Anchor Pile	Test Pile	D/S Anchor Pile	Time	Date	Remarks
(i)	(2)	(3)	(4)	(5)	(6)	(7)
1. No. Load 56 Tons	0.5580	0.0500	0.5350	09.32	29-8-68	56 Tons is the Designed load, 28 tons on each jack.
(i)	0.5660	0.1350	0.4900	12.00	-do-	
(ii)	0.5550	0.1350	0.4890	12.15	-do-	
(iii)	0.5490	0.1350	0.4900	12.30	-do-	
(iv)	0.5550	0.1350	0.4900	12.45	-do-	
(v)	0.5550	0.1350	0.4910	13.00	-do-	
(vi)	0.5560	0.1350	0.4910	13.15	-do-	
(vii)	0.5565	0.1360	0.4925	13.30	-do-	
(viii)	0.5580	0.1375	0.4925	13.45	-do-	
(ix)	0.5565	0.1360	0.4912	14.00	-do-	
2. Unload	0.5500	0.1220	0.5200	14.00	-do-	
After 15 Minutes	0.5502	0.1220	0.5215	14.15	-do-	
67.50 Tons	0.5555	0.1580	0.4932	14.20	-do-	After adding first increment of 21.50 tons total load became 67.50 tons 33.75 tons on each jack
—	—	—	—	—	—	
—	—	—	—	—	—	
—	—	—	—	—	—	
—	—	—	—	—	—	

Continued:-

Addition and release of loads, and observation of the behaviour of piles from the gauges connected, therewith were continued on similar pattern till the ultimate load capacity of the test-pile was determined.

TABLE - 3 TAUNSA-PANJNAD LINK CANAL-PILE LOAD TEST-STATEMENT-SHOWING CROSS AND PERMANENT SETTLEMENT UNDER VARIOUS LOADING CONDITIONS WITH 55 - FOOT PILE PENETRATION.

Description of Operation	Continuous Gauge Reading	Cross Settlement (inches)	Permanent Settlement (inches)
(1)	(2)	(3)	(4)
Initial gauge reading with no load	0.0500	—	—
Gauge reading after applying 56 tons for 2 hours	0.136	—	—
Gauge reading after unloading 56 tons	0.122	—	0.072
Gauge readtng after applying 67.50 tons for 2 hours.	0.1575	0.1075	—
Gauge reading after unloading 67.50 tons	0.1545	—	0.1045
Gauge reading after applying 78.75 tons for 2 hours	0.1645	0.1145	—
Gauge reading after unloading 78.75 tons	0.1598	—	0.1098
Gauge reading after applying 90 tons for 2 hours.	0.1935	0.1435	—
Gauge reading after unloading 90 tons	0.066	—	0.016
Gauge reading after applying 100 tons for 2 hours	0.1990	0.1495	—
Gauge reading after unloading 100 tons	0.069	—	0.019
(a) Gauge reading after applying 112 tons for 48 hours.	0.2370	0.187	—
Gauge reading after unloading 112 tons	0.093	—	0.043
Gauge reading after applying 120 tons for 2 hours	0.5153	0.4633	—
Gauge reading after unloading 120 tons	(c) 0.296	—	(b) 0.246
Gauge reading after applying 125 tons for 2 hours	0.7334	0.6834	—
Gauge reading after unloading 125 tons	0.5515	—	0 5015

(a) 112 tons is design test load.

(b) Permanent settlement 0.246 inch, being very near to 0.25 inch. indicates that 120 tons load is failure-load or ultimate load bearing capacity of the test pile.

(c) This reading indicates the position when the pile has completely ceased to rebound. After unloading, the pile may continue to rebound for two-three hours depending upon the characteristics of the surrounding soil.

(d) In addition to the above loads, the test pile was also subjected to the self-load, loads of two jacks, and jack-blocks, which was about 10 tons. Accordingly, the ultimate load bearing capacity of the test pile is reckoned as 125 tons, being on the safer side.

TABLE - 4 PILE DRIVING AND LOAD-TESTING EQUIPMENT
TAUNSA-PANJNAD LINK CANAL

Quantity	Equipment	Manufacturer	Country of Manufacturer
(1)	(2)	(3)	(4)
1	Crane, P & H 430, T.C.	Kobe - Steel Works	Japan
1	Pile Driving Hammer, Delmag Model D-22, Diesel Operated	Delmag	U. S. A.
1	Telescopic lead alongwith other Attachments	Kobe - Steel Works	Japan
1	Two-Stage Centrifugal Pump	Aurora	U. K.
1	Electric Motor for Pump 40-H.P.	Crompton Parkinson	U. K.
2	Hydraulic-Jack-120 tons	—	U. S. A.
1	Electric-Operated Winch	—	Pakistan
1	Electric Motor for Winch 10-H.P.	Decent	Pakistan
1	Load Application Assembly Improvised from Steel Girders	Site Workshop	Pakistan
1	Welding Set, Petbow	Petbow	U. K.
1	Oxy-Acetyline Torch	—	Germany
1	Ruston Generating Set, KW-50	Ruston & Hornsby Ltd	U. K.
1	A set of Soliddrawn seemless Steel 3 inch dia. Jetting Pipe 90 feet long, with Helically Reinforced Rubber Hoise Pipe 3 inches dia., 150 feet long.	—	—

TABLE - 5 MCKIERNAN-TERRY DOUBLE - ACTING STREAM/AIR HAMMERS

Item	Size of Hammer			
	9-B-3	10-B-3	11-B-3	C5
(1)	(2)	(3)	(4)	(5)
Normal stroke (h), in.	17	19	19	18
Ram, Weight (W_r), Ib.	1,600	3,000	5,000	5,000
Casing, weight (Wc), Ib.	5,070	7,225	8,182	6,880
Manufacturer's rated energy per blow (E_n) ft. Ib.	8,750	13,000	19,150	16,000
Stroke per min.	145	105	95	100-110
Anvil, weight, Ib. Flat	330	625	818	
Cup or bell	342	726	991	
Recommended steam pressure at boiler or air pressure at compressor, psi.	125	125	125	125
Steam or air pressure at hammer, psi	100	100	100	100
Size of boiler, hp	45	50	60	45
Compressed air, cfm	600	750	900	585
Hose size, in.	2	$2\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{2}$

TABLE - 6 LINK BELT SPEEDER DIESEL HAMMERS

Item	Size of Hammer		
	105	312	520
(1)	(2)	(3)	(4)
Maximum stroke (h), in.	to 38	to 36	to 48
Idling strokes, no impact, in.	14	16	22-24
Ram, weight (W_r) Ib.	1,460	3,800	5,000
Casing weight (W_r), Ib.	1,845	5,010	6,120
Manufacturer's rated energy per blow (E_n), ft-Ib.	Upto 7,500	Upto 18,000	Upto 30,000
Stroke per min.	90-98	100-105	80-84
Over-all length ft.	10.17	10.75	13.50
Min width of leads usable with hammer, in.	18.5	26.5	26.5
Diesel fuel, gal. per hr.	0.9	2.5	3.5

TABLE - 7 MCKIERNAN-TERRY DIESEL HAMMERS

Item	Size of Hammer	
	DE 20	DE 30
(1)	(2)	(3)
Stroke (h), in.	48-96	48-96
Ram, weight (W_r), Ib.	2,000	2,8000
Casting weight (Wc), Ib.	3,750	5,325
Manufacturer's rated energy per blow (E_n) ft-Ib.		
Min....................................	12,000	16,800
Max....................................	16,000	22,400
Stroke per min.	48-52	48-52
Over-all length, ft.	11.67	14
Diesel fuel, gal. per hr.	1.6	2

TABLE - 8 DELMAG DIESEL PILE HAMMERS

Item	Size of Hammer		
	D-5	D-12	D-22
(1)	(2)	(3)	(4)
Total weight, Ib	2,220	4,940	8,800
Ram weight (W_r), Ib.	1,100	2,750	4,850
Manufacturer's rated energy (E_n) ft-Ib. per blow	9,100	22,500	39,700
Stroke per min.	50-56	50-60	50-60
Over-all length, ft.	11.25	12.25	12.83
Diesel fuel, gal. per hr.	0.67	1.75	3.33

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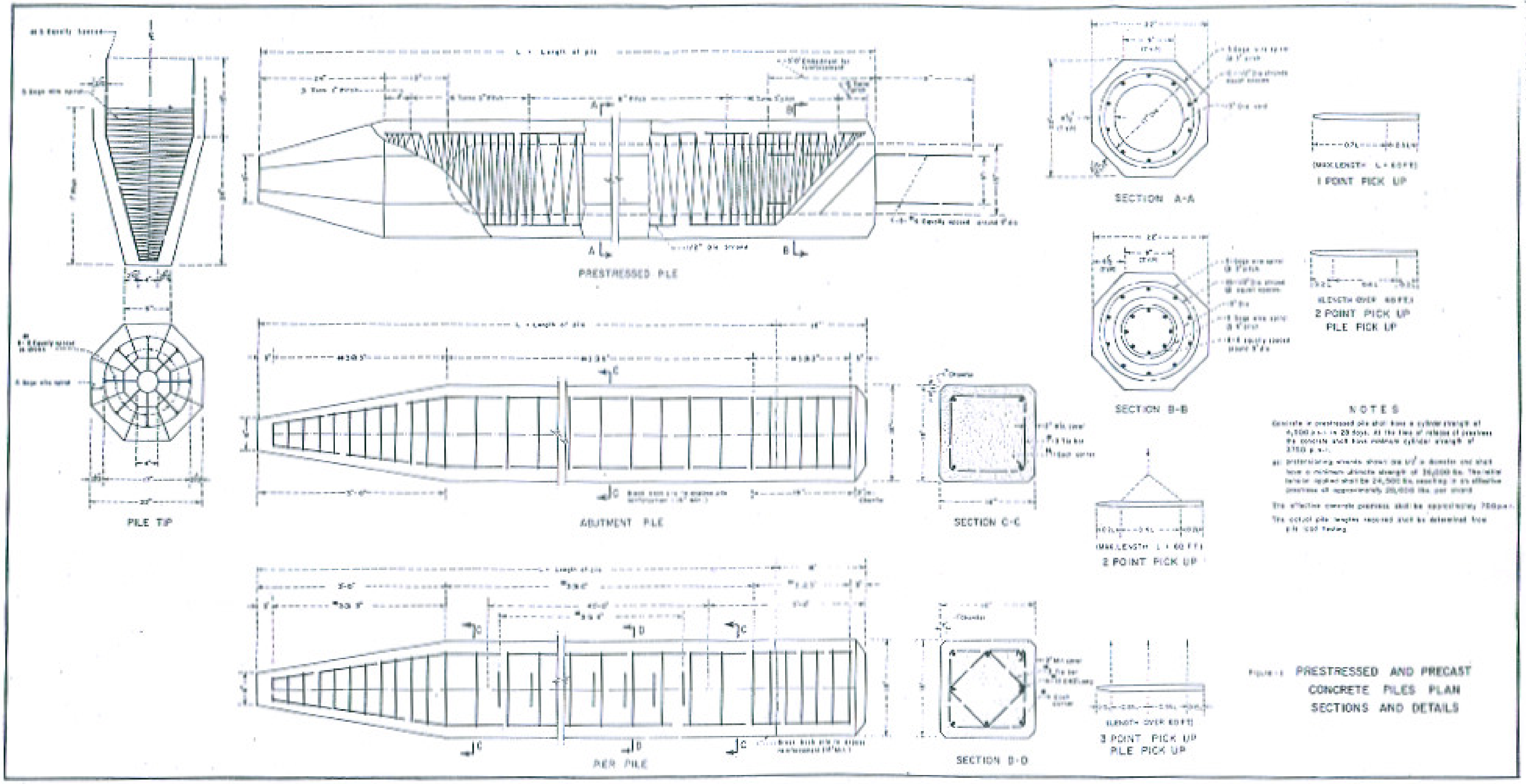

Figure PRESTRESSED AND PRECAST CONCRETE PILES PLAN SECTIONS AND DETAILS

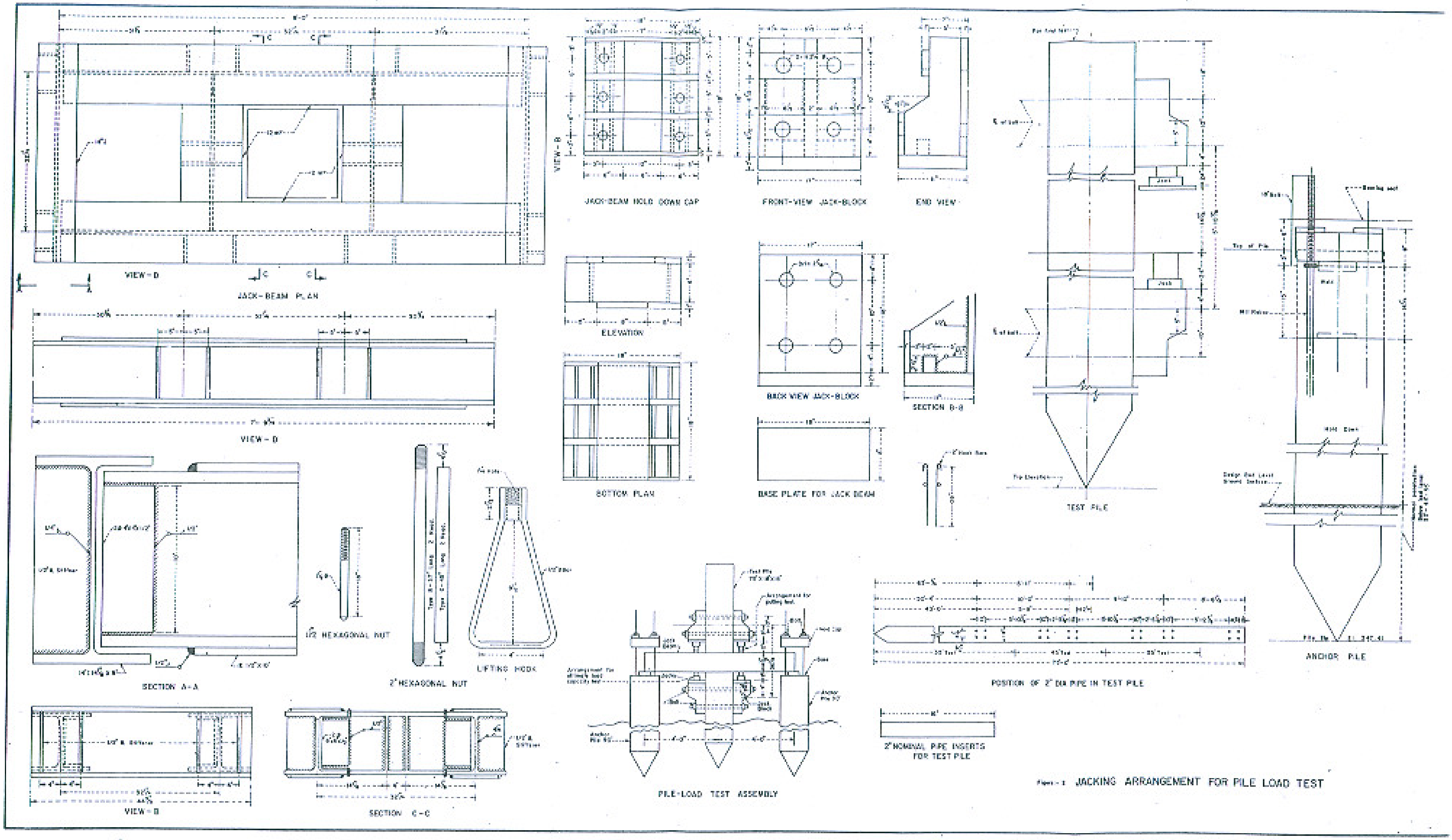

Figure - 1 JACKING ARRANGEMENT FOR PILE LOAD TEST

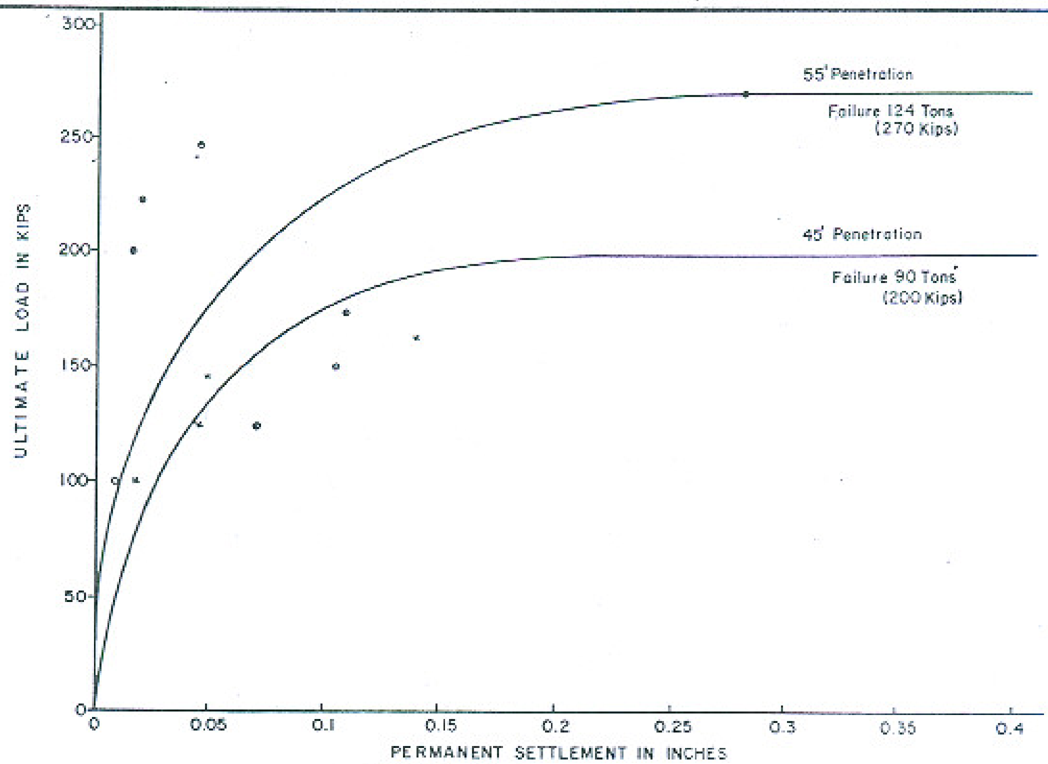

Figure:- 3 TAUNSA-PANJNAD LINK CANAL
PILE LOAD TEST-LOAD-SETTLEMENT CURVE

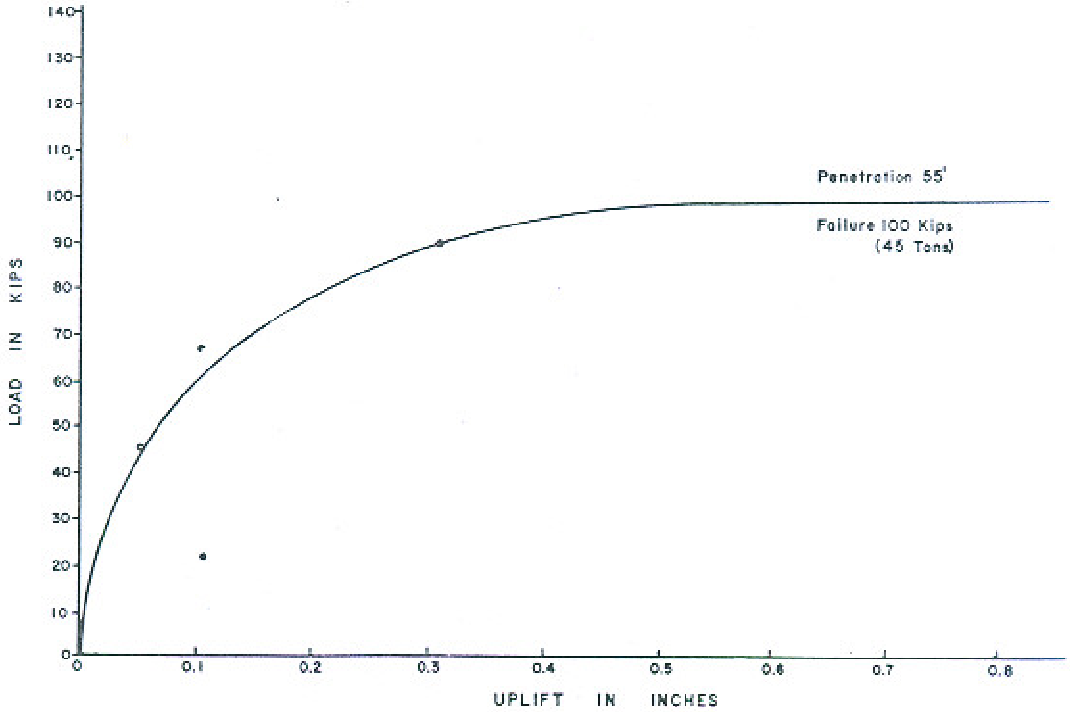

Figure:- 4 TAUNSA-PANJNAD LINK CANAL
PULLING TEST-PULL-UPLIFT CURVE

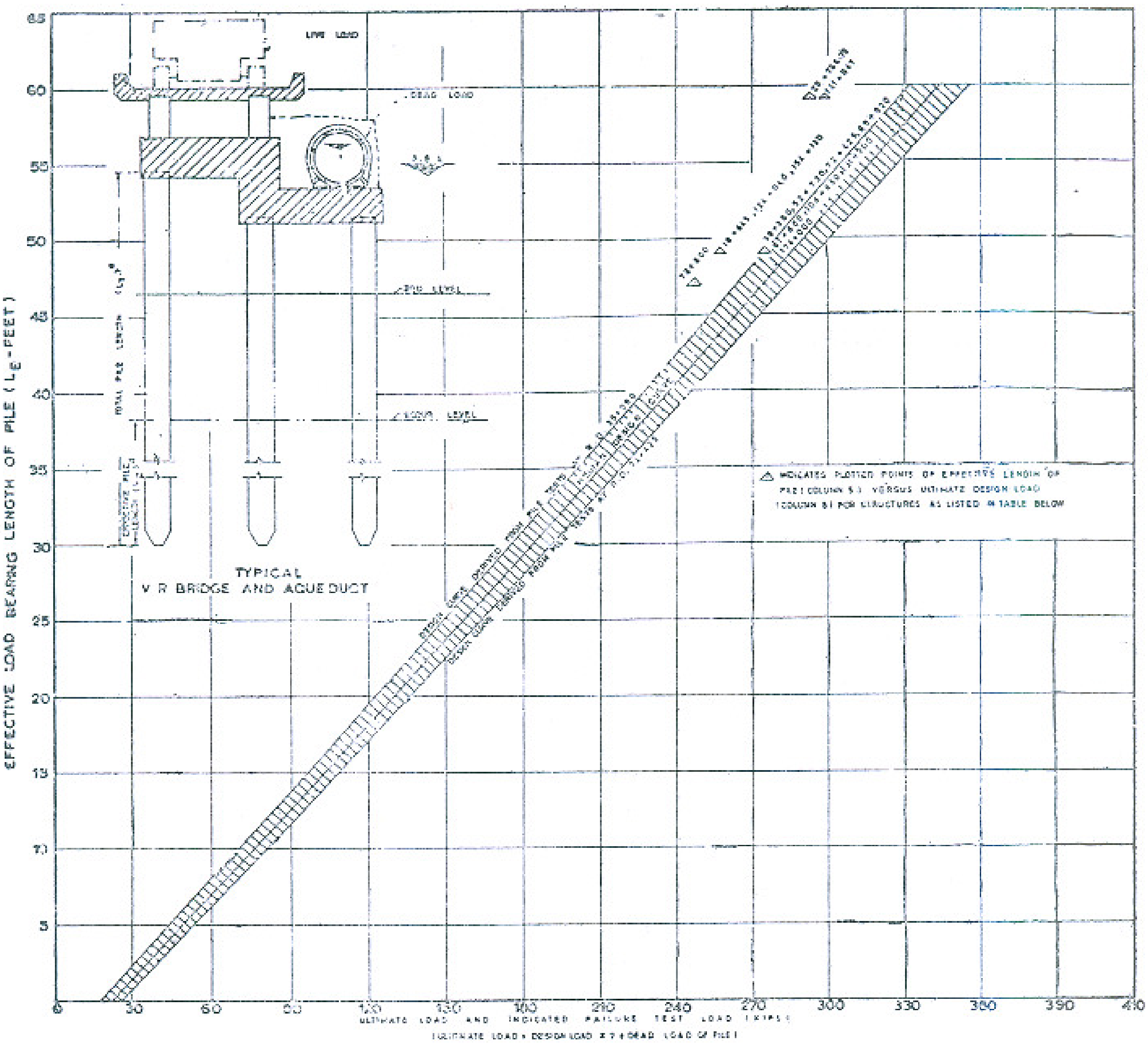

TABLE - A. DESIGN, INDICATED FAILURE TEST- AND ULTIMATE PILE LOAD CAPACITY

LOCATION R D	TYPE OF STRUCTURE	NO OF PILES PER PIER	PILE LENGTH (L_T)[a]	EFFECTIVE LENGTH (L_E)[a]	DESIGN LOAD PER PILE[d]	DEAD LOAD OF PILE[b]	ULTIMATE LOAD 2×COL(6)+COL(7)	INDICATED FAILURE TEST LOAD[c]	EQUIV. NET TEST LOAD COL(9)-COL(7)	FACTOR OF SAFETY COL(10)÷COL(6)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
10+645	V R BRIDGE	2	75	49	130 KIPS	17 KIPS	257 KIPS	286 KIPS	269 KIPS	2.1
39+766.75	A R BRIDGE (A)	4	85	59	137	19	293	330	321	[illegible]
33+380	V R BRIDGE CUM AQUEDUCT	3	75	49	129	17	275	286	269	[illegible]
[illegible]+710	FOOT BRIDGE CUM [illegible]		75	49	129	17	275	286	269	2.1
72+125	V R BRIDGE CUM AQUEDUCT	3	75	49	129	17	275	286	269	2
78+500	V R BRIDGE CUM AQUEDUCT	5	75	47	115	17	[illegible]	275	256	2.2
84+920	FOOT BRIDGE		75	49	129	17	275	286	269	2.1
97+683	FOOT BRIDGE	1	75	49	129	17	275	286	269	2.1
106+850	FOOT BRIDGE		75	49	129	17	275	286	269	2.1
124+910	V R BRIDGE	2	75	49	120	17	257	286	269	2.1
131+847	A R BRIDGE CUM AQUEDUCT (A.A.)	5	85	58	140	15	295	319	321	2.3
152+390	V R BRIDGE	2	75	49	125	17	257	286	285	2.1
167+700	FOOT BRIDGE		75	49	129	17	275	286	269	2
174+000	FOOT BRIDGE	1	75	49	129	17	275	286	269	2.1

(a) TOTAL AND EFFECTIVE PILE LENGTH SHOWN ARE MEDIAN LENGTHS, INDIVIDUAL PILE LENGTHS AT EACH STRUCTURE VARY ± 10 % FROM LENGTH SHOWN

(b) DEAD LOAD OF PILE TAKEN AS DRY WEIGHT, 0.135 KIPS/FOOT AND WEIGHT BELOW S.W.L. 0.15 KIPS/FOOT

(c) INDICATED FAILURE TEST LOAD (COLUMN 9) IS DERIVED FROM THE MEDIAN DESIGN CURVE ABOVE FOR THE EFFECTIVE PILE LENGTH, L_E (COLUMN 5) SHOWN FOR THE STRUCTURE

(d) TOTAL LENGTH (L_T) = EFFECTIVE LENGTH (L_E) + SCOUR DEPTH + LENGTH ABOVE BED LEVEL + ALLOWANCE FOR EMBEDMENT IN THE CAP

Figure 5 DESIGN SUMMARY- PILE LOAD BEARING CAPACITY

General Section

Cost Benefit Analysis

by
MR. ASRAR AHMED QURESHI
Deputy Chief Engineer, WAPDA

1. **Introdcution :**

Engineers are always concerned with the design, construction and management of projects which vary widely in nature, size, cost and benefits. It is very important, therefore, that due consideration is given to the costs and benefits from the very inception of a project ; to the evaluation of various alternatives which are considered in its initial stages ; and to the control of its costs when being implemented or constructed. It is important that the significance of carrying out a soundly-based evaluation and of knowing the sensitivity of the evaluation to possible changes in future costs of the work through many causes, is properly understood.

It is at the student stage of a project manager's development that the seeds of "Cost consciousness" should first be sown if they are to bear the most prolific crop. This article is meant for the young practicing engineers and managers incharge of projects both in public and private sectors to make them cost conscious. An effort has been made to give some idea of the existing methods of project evaluation alongwith the past practice.

2. **Economics of Projects :**

All business enterprise is governed by certain economic laws which are the result of interaction between people and wealth. *Economics* is a Social Science which deals with people and their money. Alternatively, it is the study of wealth, its value, creation and distribution. Economics can also be defined as means to obtain the best value or making the best use out of the limited resources that are available.

One of the outstanding characteristics of the work of an Engineer is that it results in products or services within cost limitations to be utilized economically by mankind. Economics and basic economic laws are, therefore, important factors in many economy studies. Engineers frequently have difficulty with economics. This is because of the fact that economics is not an exact science like mathematics, physics, etc. with which they are accustomed to deal with. Since economics involves the reactions of *people*, its laws are based on the behaviour or reactions of *groups* of people described in general terms. Thus economic laws are generalised laws or generalizations.

For a successful Engineer, knowledge of certain economics is a "*Must*". Engineers, as a group, probably do more than any other to better the economic status of the people through their inventions and techniques. In their work they utilize capital in

The author delivered this lecture at National Institute of Public Administration, Lahore on 13.7.1974.

various forms, the most important being the *money*, other being labour and. materials.

Money is recognized as an engineering material like Steel, Cement, etc. and Engineers and Project Managers are held responsible and accountable for its use. An Engineer is required to examine all aspects, technical, economic, financial, managerial, commercial, etc. before embarking upon a project. To make a decision if a project is viable or not, he calculates the rate of return and compares it with the minimum rate that is acceptable.

When a young engineer after completing his education enters upon his professional career in industry, one of the most immediate and sometimes difficult, adjustments he must make is the recognition that money is an essential material in almost every design that he creates. Decidedly, he cannot hope to achieve a high level of professional success unless he deals with money as effectively as he does with the other materials he uses in his projects.

The importance of money as an engineering material may be illustrated by the fact that in order to furnish the plant, equipment and tools which are necessary to provide a job for one worker in industry in the United States, someone must accumulate and invest, on the average, over $10,000. The requirement of a large investment of capital per worker is one of the outstanding characteristics of modern industrial economy.

A construction contractor's site manager is faced with the economic problem in many situations. He almost certainly has limited resources, labour and/or equipment. In the event of a conflict of demand for the resources which are available the manager must make a decision as to how he can distribute them most economically to the various wants.

The use of economic theory and practice in this context is cȯncerned with putting limited resources to their best use in order to produce goods and services which are significantly based to give a good return and prove to be the best cost benefit organ during operation.

3. **Requirements of an Economy Study of an Engineering Project :-**

1. It should be based on consideration of all available factors.
2. When some factors must be estimated, these estimates should be made intelligently, in the light of experience and sound judgment.
3. The study should show a measure of financial efficiency such as the annual rate of return upon the required investment.
4. Insofar as they apply correctly, the study should make use of the same factors that the accountant will use in determining the financial efficiency of the investment after it is made.
5. The study should contain a recommended course of action, together with the reasons for the recommendation.

Studies made in this manner will assure that those who must make investment decisions related to engineering projects will have the necessary facts for optimum utilization of capital funds.

4. **Human Factor in Economic Planning :-**

Relationship of the human individual to the project is a factor which has tremendous effect upon the economy of an engineering undertaking. Engineering Organization and Industry are becoming increasingly conscious of this fact. Human element cannot be written down in an equation or a set of rules, nor can it be accurately predicted at all times, yet it may be so great as to outweigh all other factors. Much of the knowledge of this factor is learnt only through long, and sometimes bitter experience. As long as it is present, the engineer can ill-afford to close his eyes on it. The reaction of public to a new engineering product or the effect of bad relationship between labour and management are just two examples of this factor. An old saying "You can't fool all the people all the time" may be paraphrased as "It is foolish to disregard any of the people any of the time". The engineer must not neglect the human factor in his engineering economy.

5. **Accountant and Economist :-**

It is very important that the functions of the two which are so different and yet frequently confused are clearly understood.

The traditional accountant's role is essentially that of recording historic data which ensures an honest account of showing where from the funds used have been obtained and to where and on what they were spent. His account is displayed in a balance sheet which covers a restricted period of 12 months. The main outcome of the balance sheet is to know whether the accounting periods, trading has resulted in a profit or a loss. In summary, the accountant is concerned with the establishment of the profit or loss for a discreet accounting period in the past.

An economist on the other hand is concerned with taking possible steps to make sure that money invested in a project will bring an acceptable return or yield before the final decision to invest is taken. He is concerned with obtaining forecast of what is likely to happen throughout the life of the proposed investment project in order to aid the decision-making process.

Most people think that the functions of each as being in opposition but it is not so. It is necessary that each understands the other's point of view and his method of working since both within a company, or an organization are concerned to achieve its objectives.

6. **Calculation of Profits :**

Under the capitalistic system profits must be earned to assure that needed capital is available. Profit is defined as excess of income over expenses and thus,

Profit = Income - Expenses . . . (1)

Such a generalized equation is not sufficiently specific to indicate the various factors that must be considered in determining profit. Both profit and expense are complex.

Two classes of profit are of interest in economic studies. The first is profit before income taxes, while the other is profit after income taxes.

Expenses may conveniently be divided into several classes as follows :

O+M = Out-of-pocket expenses for operation and maintenance.

D = Depreciation, always pre-paid through purchase of property, buildings or equipment.

I = Interest paid for the use of borrowed funds.

T = Income Tax (not an ordinary expense, since they depend upon profits remaining after other expenses are paid).

Using this notation, Equation can be rewritten in more specific form as follows :

$$Ea = G-(O+M+D+I) - T \ldots (2)$$

where Ea = Net profits (or earnings) after taxes

and G = Gross income or receipts

Since interest paid for the use of borrowed capital is included as an expense, the profits shown in equation above are those belonging to the owners of the equity capital. Engineering economy studies deal with all of the factors in Equation in order to determine what course of action should be followed to assure optimum efficiency in the use of capital.

7. The use of Cost-benefit Analysis for Evaluation of Individual Projects :-

Cost Benefit Analysis in its present sophisticated form it is a comparatively recent development. Before the country embarked on planned development, the practice in vogue for establishing the justification of an irrigation project for example, was to apply to it what was known as the "Financial productivity test". This test was based on a straight comparison of most elementary items of visible expenditure and revenues (income) of the project. The expenditure was taken as :-

(i) Annual Maintenance and Operation Costs, and

(ii) Interest charges.

The revenue (income) was considered as the yearly water tax and land revenue paid by the prospective cultivators. The project was termed "Productive" and "Justified" if after ten years of its completion it was able to earn revenues sufficient to meet the working expenses plus a minimum of four per cent to cover interest on the "Sum at Charge" or the capital cost. The "Sum at Charge" being defined as the direct cost of construction plus the accumulated arrears of simple interest up to the 10 year of the commissioning of the project. The capital cost of the project was never recovered and was treated as a perpetual debt on the assumption that the project had an unlimited life and could be operated perpetually through normal maintenance. This criterion was perhaps good enough for application to an isolated project at a time when construction costs were low and waters needed for irrigation could easily be obtained from the rivers through simple diversion weirs with almost unlimited life ; when the pace of development was slow, and not many projects were competing for development funds at one time. It has, however, very limited application left in the changed conditions of planned development designed to achieve balance growth of the economy as a whole.

Cost benefit ratios of projects, properly worked out, enable us, unlike the old "Rate of Return" formula, not only to assess the economic worth of individual projects but also to determine which of the several projects designed to serve different purposes would confer the largest net benefits on the economy as a whole. Cost benefit analysis, as a technique, would also permit us to assign priorities or to decide on the relative economic ranking of projects in the same field, to compare the economic worth of projects in different fields, and to determine what should be the optimum size of projects or programmes.

It is not always easy to determine what constitutes the total benefit of a project and what its total cost. In quite a number of fields like Education and Health the benefits of a project are not quantitatively measureable. In the case of other projects like those in the field of Water and Power, Transport and Communications there may result external economics in addition to direct benefits.

The costs are usually divided into Capital Costs, Fixed Annual Charges and Annual Operating Costs which are briefly explained as under :-

Capital Costs
Machinery, Plant, etc.
Structures
Land

Fixed Annual Charges
Interest
Replacement ;
  Structures
  Generating Units
Insurance

Annual Operating Costs
Fuel
Operating Staff
Supplies
Maintenance

8. **Example :**

The foregoing can be better understood by the following example :-

It is proposed to install a 75 KVA Generating Set at a certain location. There can be two alternatives. Install a Hydro-Electric Generating Set, if the required quantity of Water and all are available or alternatively a Diesel Generating Set can be installed. The Capital Costs, Fixed Annual Charges and Annual Maintenance and Operation Costs for each type of installation are estimated on present day prevailing prices as under :-

Description		75 KV Hydro-Plant Rs.	75 KVA Diesel Plant Rs.
Capital Costs :			
Generating Unit		1,47,000	90,000
Structures		4,59,000	25,000
Land		12,000	5,000
Total :		6,18,000	1,20,000
Fixed Annual Charges :			
Interest @ 4%		24,720	4,800
*Replacement :	Structure @ 0 6%	- 2,760	
	Gen. Unit @ 1.3%	1,910	
	@ 6%	-	150
	@ 2 4%	-	2,175
Insurance @ 9.25%		370	225
Total :		29,760	7,350
Annual Maintenance and Operation Costs :			
** Fuel		—	2,50,000
Operating Staff		7,500	5,000
Supplies		1,000	3,000
Maintenance		1,240	2,000
Total :		9,740	2,60,000
Total Annual Charges		39,500	2,67,350

Notes :-

Hydel Plant :

* Generating life @ 35 years and structures life @ 50 years with equal annual payments into sinking fund receiving 4% interest.

Diesel Plant :

* Generator life @ 25 years and structure life @ 50 years with equal annual payments into sinking fund receiving 4% interest.

** Based on 3,23,000 KWH generation during the year operating at 50% load factor.

Annual Production of Energy :

Assuming a load factor of 50% within 5 years, the total number of units produced

$$\text{annually} = \frac{75 \times 30 \times 12 \times 24 \times 50}{100} = 3{,}24{,}000 \text{ RWH}$$

(Units of Energy)

Cost Per Unit of Energy Generated :

	75 KVA Hydro-Plant Energy	75 KVA Diesel Plant Energy
Fixed Annual Charges	Rs. 29,760	Rs. 7,350
Annual Maintenance and Operation Costs	Rs. 9,740	Rs. 2,60,000
Total Annual Charges	Rs. 39,500	Rs. 2,67,350
Unit Cost (for producing 3,24,000 KWH annually)	Rs. 0.12	Rs. 0.83

From the above, it is abvious that in the case of a company or organization, producing electricity at Rs. 0.12 rupees (Paisa Twelve) per unit, and selling it to the consumers @ Rs. 0 25 (Paisa Twenty-five) the benefit-cost ratio

$$= \frac{\text{Total Earnings}}{\text{Total Cost of Production}}$$

$$= \frac{81{,}000}{39{,}500} = 2.05 \text{ approx.}$$

as Total Annual Earning

$$= \frac{3{,}24{,}000 \text{ KWH} \times 0.25}{100} = \text{Rs. } 81{,}000$$

Total Annual Costs Rs. 39,500

The above illustration takes into account only direct benefits, which can be easily written down in figures. There are numerous other socio-economic benefits which cannot be put down in terms of money such as improvement in the living comfort of the people by providing a steady supply of electricity for lighting and heating, setting up of cottage industries, providing water for irrigation thereby increasing farm products, increased supply of consumer goods to the area in place of kerosene oil not needed any more : and several other allied benefits.

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2. Any change in the name, address or Telephone number of any firm may also be promptly notified for carrying out a correction in the next issue.

٥- وَلَا تَأْكُلُوا أَمْوَالَكُمْ بَيْنَكُمْ بِالْبَاطِلِ وَتُدْلُوا بِهَا
إِلَى الْحُكَّامِ لِتَأْكُلُوا فَرِيقًا مِنْ أَمْوَالِ النَّاسِ
بِالْإِثْمِ وَأَنْتُمْ تَعْلَمُونَ ○

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

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

٦- وَقُولُوا قَوْلًا سَدِيدًا ○

"And speak straight words." xxxiii : 70

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

٧- اجْتَنِبُوا كَثِيرًا مِنَ الظَّنِّ إِنَّ بَعْضَ الظَّنِّ إِثْمٌ
وَلَا تَجَسَّسُوا وَلَا يَغْتَبْ بَعْضُكُمْ بَعْضًا

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

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

٨- وَلَا تَقْفُ مَا لَيْسَ لَكَ بِهِ عِلْمٌ إِنَّ السَّمْعَ
وَالْبَصَرَ وَالْفُؤَادَ كُلُّ أُولَٰئِكَ كَانَ عَنْهُ
مَسْئُولًا ○

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

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

٩- وَتَعَاوَنُوا عَلَى الْبِرِّ وَالتَّقْوَىٰ وَلَا تَعَاوَنُوا
عَلَى الْإِثْمِ وَالْعُدْوَانِ وَاتَّقُوا اللَّهَ

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

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

١٠- وَأَمْرُهُمْ شُورَىٰ بَيْنَهُمْ

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

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

١١- وَاعْتَصِمُوا بِحَبْلِ اللَّهِ جَمِيعًا وَلَا تَفَرَّقُوا

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

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