It is not the object here to dilate extensively on various signalling and interlocking systems being practised on the technically advanced and developed railways of the world, details of which have been given in the relevant chapter of Signalling and Interlocking of this book. A mention, however, in the context of projected speed increases which can be achieved and which have been achieved in the world is necessary. Speed limits can be attained signalling wise as under:—

Speed limit	Signalling and Interlocking
75 MPH/120 KMH	Mechanical Signalling (conventional) Signals being electrically lit.
75 MPH to 100 MPH (120 KMH to 160 KMH)	Automatic signalling coupled with some approach warning devices but without automatic braking.
100 MPH to 125 MPH (160 KMH to 200 KMH)	Automatic Signalling with intermittent train running control of location type.
125 MPH to 200 MPH (200 KMH to 300 KMH)	Automatic signalling with continuous train running control. Remote control can eliminate the need of driver even.

Since signalling and braking are inalienably interlinked, improvement in braking is a 'Must' to keep the block sections as small as possible. This will give a desired increase in capacity besides ensuring safety.

(b) Level Crossings:

The introduction of projected speed increases will require the elimination of level crossings by providing over-head or under-bridges for the road traffic. Till such time when the level crossings stand eliminated, fool proof signalling-cum-Interlocking and approach warning devices will have to be provided to introduce speeds above 75 MPH/120 KMH.

(v) SAFETY INSTALLATIONS

Safety installations and signalling/interlocking devices go hand in hand. 80% of the safety installations so far devised on the Technically developed railways are in the fields of signalling/Interlocking and approach warning which are installed on the line and within the locomotive cab. No high speed increases can be projected or achieved without having adequate safety installations whose efficacy will have to be proven through sustained trials simulating the actual conditions that are to be faced and experienced by the running staff of the Railway system on the one hand and the equipment/gadgets on the other.

ESSENCE

A perusal of the above details concerning studies/investigations for long term planning of the target speed will enable us to say that a target speed for the primary main line from Lalamusa to Lahore—Karachi can be fixed as 100 MPH/160 KMH for passenger traffic and 60 MPH/100 KMH for goods traffic which is planned to be achieved, say by the year 2000 A.D. This means that all the sectors and disciplines of the railway system will plan, design and execute the future schemes and projects including introduction of new technologies in a co-ordinated manner to attain the above mentioned target speeds fixed after thorough and comprehensive joint consultations and studies. In nutshell, the over all target speeds will have to be adhered to by all the sectors once they are settled, in the manner prescribed in the foregoing paras.

A schematic chart as an aid to group the strategy of long term planning duly hiteched to 'Speed up' philsophy is given below for the benefit of the readers.

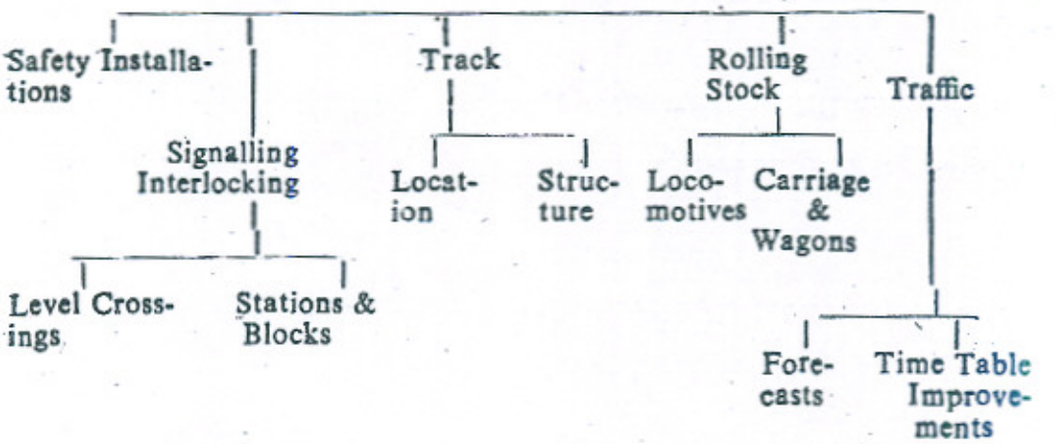
PERSPECTIVE PLAN FOR 'SPEED UP' OF RAIL TRANSPORT

TARGET SPEEDS

Section : Lala Musa — Lahore — Karachi (Main Line)

Speeds , *(i)* 60 MPH/100 KMH for goods trains

(ii) 100 MPH/160 KMH for passenger trains



II. Short Term Planning:

Various projects, schemes, tasks and items of work required to be executed under the overall frame-work of the above referred perspective plan have to be now dove-tailed and synchornised with the National Short Terms plans which normally extend from 4 to 6 years. Naturally the perspective plan

prepared for the period 1980-2000 will be split up into four 5 year plans which should fall in line with the National 5 year plans prepared and approved by the National Planning Commission and Government. Priorities for various tasks and works will be planned and fixed most carefully, keeping in view the technical considerations and availability of resources. A work earmarked for completion during the plan period 1995-2000 will not be attempted to be executed during the period 1980-85. The over-all target speeds can also be sub-divided into 4 steps to render achievement of over-all target in a smooth, orderly and evolutionary fashion as indicated in the table given below:—

Phase	Period	Speeds to be achieved for passanger	Speeds to be achieved for goods
1.	1980–85	70 MPH/115 KMH	42 MPH/65 KMH
2.	1986–90	80 MPH/130 KMH	48 MPH/77 KMH
3.	1991–95	90 MPH/145 KMH	54 MPH/88 KMH
4.	1996–2000	100 MPH/160 KMH	60 MPH/100 KMH

Note — Existing speed limits are: $\frac{60\ \text{MPH}}{100\ \text{KMH}}$ and $\frac{35\ \text{MPH}}{53\ \text{KMH}}$

for passenger and goods trains respectively.

5 Year speed up plan:

Under the short term plan the various measures to achieve the objectives are described as under:—

(a) Increase in the existing maximum speed which may or may not necessitate;

1. Track strengthening.
2. Improved suspension system for coaches.
3. Locomotives of higher horse power.
4. Improved signalling.

(b) Improvements in brake power:— Improvements in the existing brake power or bringing in a new technology.

(c) Locomotives with higher acceleration and declaration . . . . . .

(d) Reduction in stoppages-their number and/or duration.

(e) Elimination of bottlenecks: (Spatio-temporal, operational etc.) which form the raison d'etre for the existing speed restrictions.

Naturally in a developing economy, just like Pakistan, the first choice falls for increase of speed with existing facilities. By adopting this course of action to achieve speed up for existing operations, low capital invest-

ment involving insignificant foreign exchange will be required. Besides, labour intensive methods will be practised instead of opting for capital intensive technologies. This will create a spirit of self-reliance among the railway technicians and staff. A sense of achievement by deploying domestic know how and resources will prevail on the Railway and the Railway and will act as an harbinger for generating self-confidence amongst Railway personnel to tackle technical problems locally without a call for foreign expertise or aid.

A schematic chart showing the tasks to be performed during the five year plans is given below:—

FOUR PHASES OF 'SPEED UP' PLAN

Speed to be achieved		Period
for passenger traffic	for goods traffic	
70 MPH/115KMH	42 MPH/65 KMH	1980-85
80 MPH/130KMH	48 MPH/77 KMH	1986-90
90 MPH/145 KMH	54 MPH88 KMH	1991-95
100 MPH/160 KMH	60 MPH/100 KMH	1996-2000

Five Year Speed Up Plan

- Reduction in stoppages, their Number/And or Duration
  - Locos with Higher Acceleration & Deceleration
- Improvements in Brake power
  - Increase in speed with or without
    - (*i*) Track strengthening
    - (*ii*) Improved suspension for coaches
    - (*iii*) Locos of higher brake power
    - (*iv*) Improved signalling
- Elimination of Bottlenecks *i.e.* Spatio Temporal, operational etc. which cause speed restrictions.

At the end of every five year plan period when all the departments of the railway have duly executed the works pertaining to their sectors and the necessary improvements in the infrastructure have been affected, high speed trials will be carried out to prove efficacy of the improved installations,

equipment, gear, gadgets, methods and techniques before pressing them in service on the section prescribed for increase in speed.

III. HIGH SPEED TRIALS:

The subject of high speed trials is in itself a vast subject of science and technology. Descriptions of detailed procedures and methodologies to be followed are beyond the scope of this article. However, in brief, it is mentioned that, in nutshell, the vibrational characteristics of the rolling stock and track structure will be recorded and analysed in detail in the perspective of standards already fixed in this connection after extensive research and experiments on the technically advanced railways through-out the world. Besides, adequacy and effectiveness of the tolerances and para-meters regarding rolling stock structure, track structure, signalling/interlocking, Electric traction etc. will be critically appraised during these trials. A similar appraisal will be made in respect of maintenance tolerances of the various parts and components of the above mentioned equipments, gear and installations. These critical appraisals may necessitate further restriction, narrowing down and revision of the proposed sets of various tolerances and parameters in order to achieve the standards of comfort and safety prescribed and to fulfil the required yard stick evolved and based on theory of vibrational characteristics.

In the realm of analysis of vibrational characteristics the following fundamentals will be assessed and identified by extensive runs of the locomotives/ coaches over the tracks on which the speeds are to be increased, simulating conditions analogous to real service circumstances.

(i) Lateral acceleration: A continuous graph indicating the lateral acceleration during each run will be prepared. The world 'Standard' at present in this connection is that the static lateral acceleration should not exceed 0.35 g. (g being gravitational acceleration i.2. 32.2 ft. per second per second). The value of lateral acceleration exclusively due to unbalanced centrifugal force during run over a curve, will not exceeds 0.1 g; it being deemed to have been included in the over all limit of 0.35 g. Lateral acceleration is of paramount consideration from the stand point of safety, the derailments of the nature of wheel climbing on rail head being caused by the aggravation of this factor mostly.

(ii) Vertical acceleration: The limit prescribed so far in respect of vertical acceleration is not as dangerous from the safety point of view as lateral acceleration.

(iii) Riding Index: From the records of the lateral as well as vertical acceleration, riding index will be worked out and if the facilities are available riding graph can also be prepared just like the graph of the lateral and vertical acceleration. Based on riding index one can know the quality of riding. The standards fixed on the Japanese National Railways and the European Railways are indicated in the tables given below:—

Japnese National Railways

Riding Index	Riding Quality
1 or less	Excellent
1 to 1.5	Good
1.5 to 2	Fair
2 to 3	bad
3 or above	Extremely bad

European Riding Quality Standard

Riding Index	Riding Quality
1.0	very good
1.5	Nearly very good
2.0	Good
2.5	Nearly good
3.0	Satisfactory
3.5	Still satisfactory
4.0	Fit for operation (Serviceable)
4.5	Unfit for operation
5.0	Dangerous for operation.

Instrumentation

An instrument called Accelerometer is placed on the floor of the locomotive or carriage whose vibrational characteristics are required to be recorded. This is the simplest way to ascertain the riding quality. The science of modern instrumentation, simulation, testing and trials is so vast and complex that no body of individuals or single organisation or a group of industries can boast of perfection.

Contd. from Page 48

utilized by placing an impervious blanket on the u/s, a protection layer and a free draining shell on the d/s side. The modified X-section of the dam fulfills the criteria of stability and seepage. The locally available material shall be used economically in the dam section.

REMODELLING OF DAMAGED SATPARA DAM IN SKARDU VALLEY

By

*Dr. Izhar-ul-Haq**

1. Introduction

Satpara Dam Project is located about 10 KM south of Skardu Town on the outlet of Satpara lake at an average altitude of 9000 ft. above mean sea level. The lake is fed by the snow melt water from the high altitude Deosai Mountains. The inflow to the lake is maximum during the months of June to August while it is minimum from December to March. The scheme envisages to augment the storage of the lake to about 9000 A.F. in order to ensure the supplies during the lean period. The Satpara Dam is a multi purpose project which shall irrigate about 4000 acres of land and shall generate 4000 KW electricity for Skardu Town.

Northern Area Works Organization (NAWO) had constructed an earth fill homogeneous embankment. The review of the available data and field reconnaissance revealed that the existing embankment has been constructed of random fill material without proper compaction and moisture control. High permeability values of the embankment were anticipated. This paper outlines the studies carried out for the existing embankment and to develop a compatible design encompassing the existing embankment.

2. Brief History

The history of Satpara dam goes back to sevententh century. The dam was constructed by Lord Ali Sher, which probably breached about 100 years ago, after a long service. The remanants of the dam with stone masonry spillway are seen in the outflow channel section of the lake.

In 1958 Mangla Dam Organization carried out some studies and proposed a 30 ft. deep gullet with a gate utilizing 30 ft. drawdown discharge of the lake for irrigation and power purposes. The scheme however was considered unfeasible as it involved excavation to a length of about one mile.

In 1963 the small dam organization also carried out preliminary studies for a 30 ft. high dam.

The defunct Northern Area Public Works Department constructed a stone masonry wall with some pipes embedded in it without carrying out proper investigations and design studies. The project did not work.

In 1976 NA.W.O. carried out studies and constructed a 20 ft. high earthfill embankment over the remnants of old dam with an outlet sluice 12 ft. below the existing water level in the lake.

In 1980 N.A.W.O. engaged Central Designs Office, WAPDA to review and design the dam and spillway.

At present the local inhabitants of Skardu have constructed a 4 feet high Coffer dam about 100 ft upstream of the existing embankment for impound-

* Director International waterlogging and Salinity Research Institute.

ing snow melt water in summer for using it in winter for irrigation purposes.

3. Project Functions

It shall be a storage dam for impounding surplus supplies during spring.

The stored water shall ensure perennial irrigation of 4000 acres of land.

It shall generate 400 KW of electricity. A drop of 600 ft. is available which shall be utilized for power generation.

4. Geology

Skardu area lies in the high altitude Deosai mountains of Ladakh Range in the Central Himalayas. The rocks belong to Paleozoic group (Permocarboniferrous) Darkot group, mostly metamorphic and volcanic in origin. However, at the project site no volcanic rock exposures are met with. The intrusive rocks exposed belong to early Tertiary age.

4.1 Geology of the area

The Satpara dam has been proposed at the outlet of a natural lake in the Satpara Valley. The Satpara lake in the valley had been formed during the glacial activity when, glaciers descended from high altitude Deosai plains and a valley glacier encroached the Satpara tributary. The Satpara lake came into existence when advancing glaciers dammed up the flowing streams. The lake is about 250 ft. deep and has an overflow channel which probably had acted for a long time as an outlet to the lake. An island in the lake is formed, due to the presence of more resistant Hornfels rock which is hard to very hard in nature.

Fig. 1 shows photograph of the lake.

The rocks exposed in the vicinity of the lake on both banks of the valley are slates. However, on the right valley slope near the embankment, the slates have been intruded by intrusive igneous rocks (Biotite Granodiorite). Due to contact metamorphism the slates have been converted into Hornfels during the igneous activity. The presence of Hornfels rock in the lake island indicates that granite and hornfels rock contact may be present midway in the valley section of the embankment under the galcial drift deposits. However, in the exploratory bore holes the rock was not encountered upto 60 ft. depth showing that glaciers have deeply eroded the rock in the valley. The exposures of granodiorite rock are limited to the right hill of the valley or may extend in the alley at greater depth on the left abutment.

4.2 Dam Site

The Satpara valley is a typical U shaped valley modified by the advancing glaciers in the geological past. The valley fill deposits are the glacial drift deposits laid down by the retreating glaciers. The morainic deposits lie well up the valley slopes. The material is hetrogeneous consisting of boulders and gravels with sand and appreciable amount of silt. Boulder and gravels are well graded ad consist of Hornfels, granodiorite and slates.

A two feet thick layer of clay has

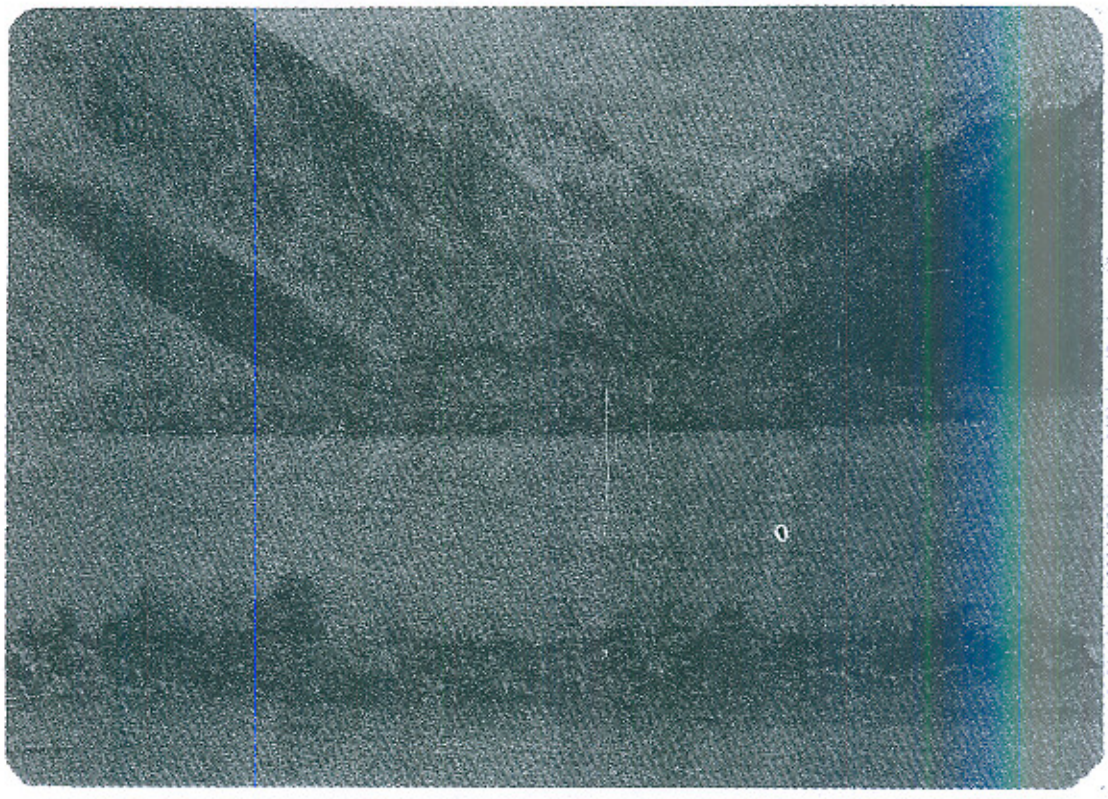


Fig: I SATPARA LAKE LOOKING U/S

been found downs tream of the lake. This layer is also seen in the cuts under the embankment. The clay layer has eithe1 been deposited by the lake which might have quite a large extent in the past and receded with the passage of time or has been deposited by the out-wash which emerges along the ice front of a glacier. The clay layer upstream of the existing embankment is not seen, probably it has been washed away. The surface material of the outflow channel of the lake where spillway has been located consists of sandy gravels, some boulders with little or no silt.

4.3 Abutments

The rocks exposed on the left hills of the valley near the lake are slates. The lower reaches of the valley slopes are covered by scree material and the morainic deposits. The rock exposures are more than 1000 ft. away from the left bank of the embankment.

The instrusive igeneous rock grando-diorite is exposed near the right abut-ment. The lower reaches of the valley slopes are covered with glacial morainic deposits and scree material. The rock exposures are more than 200 ft. away from the right abutment of the existing dam.

4.4 Rocks

4.4.1 Grandiorite: It is Coarse grained, crystalline, light grey and brownish on weathering. The rock constituents are Quartz, Feldspar mica, biotite and magnetite. The petro-graphic analysis shows that it is mica rich, compact, granatoid looking and coarse grained granodiorite. Its minerals show strain extinction. It contains 3% hydromuscovite which is a harmful material for concrete aggregate. Since its amount is small, the rock may as a whole be considered safe for concrete aggregate as far as alkaline activity is concerned. Although quartz has under-gone strong strain yet it has not reached a stage where it becomes reactive.

4.4.2 Slates : There are Brownish grey to grey, moderately hard, fine grained. The beds are striking NE 82° SE with one prominent joint set, N 70 W 55 NE. The rocks are imperfectly inter-locked and easily break along the bedding planes. The slaty cleavage is poorly developed. It contains 20% minerals belonging to Illite Illite group. Most of this appears to be hydromu-scovite. The rock is therefore unsuitable as a concrete aggregate.

4.4.3 Hornfels : The country rocks (slates) near the contact with granite have been converted into Hornfels due to contact metamorphism. They are exposed in the island within the lake. The hornfels rock is brownish grey to dark grey, hard, fine grained and com-pact. Three sets of joints are present which are tight to slightly open. The rock contains biotite and a andalusite. The deleterious clay minerals have recrystallised into non deleterious mic minerals.

4.4.4 Scree: Scree consists of angular to sub- angular boulders and gravels comprising of slates mixed with sand and silt. It disintegrates on weather-ing.

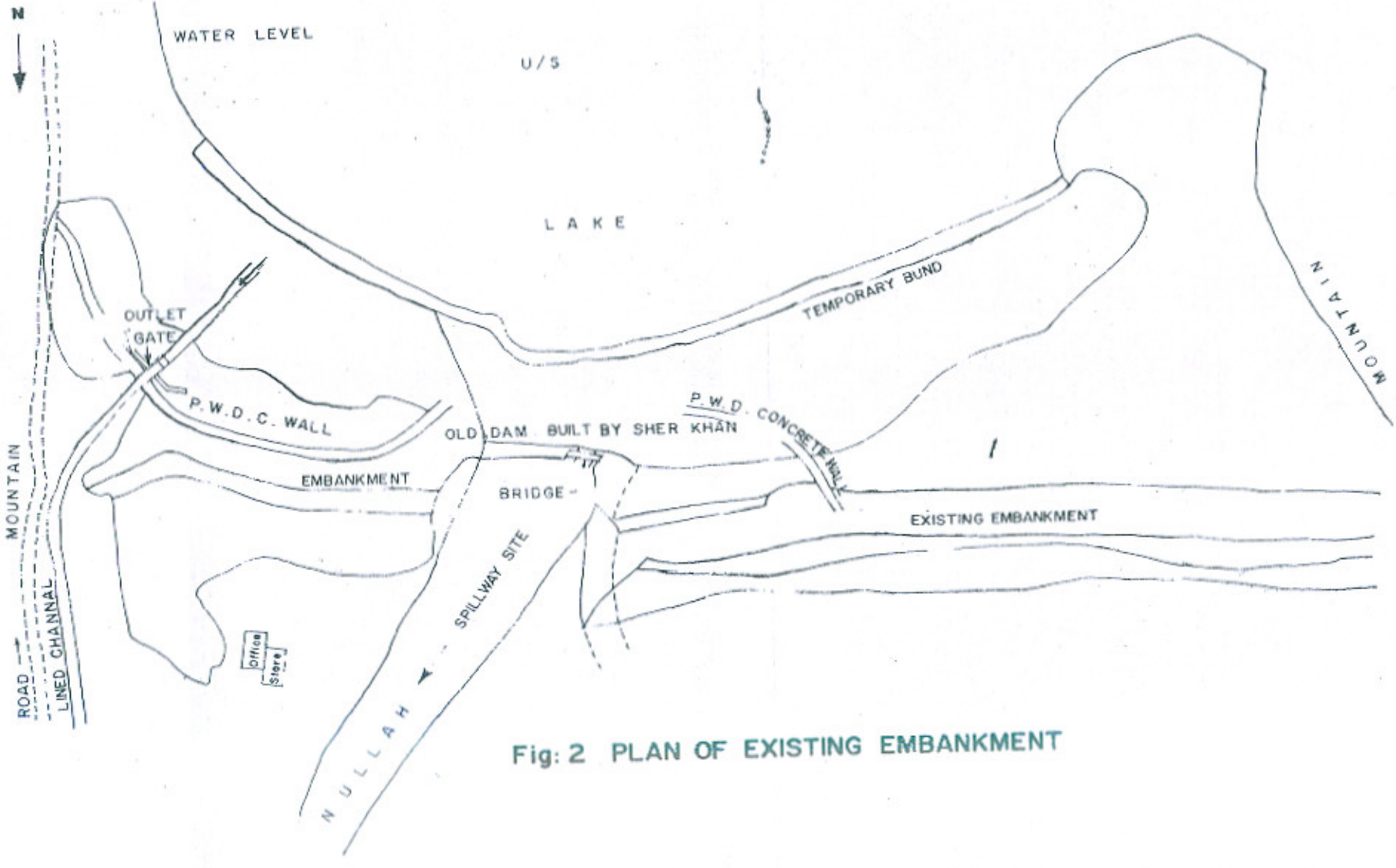


Fig: 2 PLAN OF EXISTING EMBANKMENT

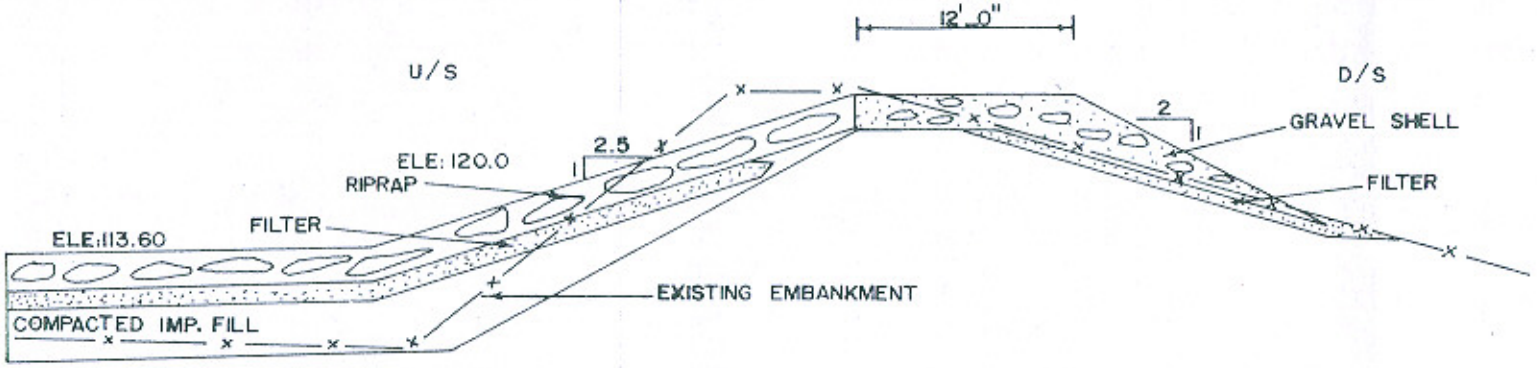


FIG. 3 BLANKETING OF EXISTING EMBANKMENT

5. Hydrology

The water supply to the lake is from snow melt water from streams like Satpara and Kachma which emanate from the high altitude Deosai mountains. The snow melt starts in the catchment area in the middle of May and ends in middle of November.

The area receives occasional snow, fall from mid December to March and minimum temperature occasionally drops below $0^{o}C$. The minimum temperature recorded is $31^{o}C$ in December, 1985. The region is outside the effective influence of the Monsoons and the rainfall is extremely low and erratic. Based on limited data available, average rainfall varies from 3'' to 18''. The discharge of Satpara nullah varies from 20 cusecs in winter to about max of 3000 cusecs in summer.

6. Existing Embankment

Fig. 2 shows the plan of the existing embankment. The embankment has been constructed on the remants of an old dam as a random fill without proper compaction and moisture control. No records were available of foundation preparation or presence of any cutoff underneath the embankment. In situ-density and permeability tests were conducted in open pits excavated in the existing embankment. Samples were taken and the following laboratory tests were carried out, (i) complete gradation analysis, (ii) Atterberg Limits and (iii) Proctor tests. The embankment material consisting of loose, uncompacted poorly graded sandy silty gravels with some cobbles and occasional boulders is considered to be quite pervious. The results of permeability tests conducted in pits in the existing embankment indicate that the embankment material is fairly pervious.

The embankment material is quite hetrogenous. The percentage of fines varies from 10 to 40%. The average in-situ wet density is 108 lb/ft^3 while average dry density is 102 lb/ft^3. The Proctor density of the material is 110 lb/ft^3 (dry) and optimum moisture content is of the order of 12%. Comparison of the in-situ density with the Proctor density reveals that the embankment is not compacted upto 95% Proctor density.

From the economic considerations it was decided to develop a design encompassing the existing embankment.

7. Design of Modified Embankment

7.1 Body of Embankment: The modified embankment consist of jacketing the existing embankment with an impervious apron and a protective layer on the u/s side. A sandy gravel section with a transition filter blanket shall be added on the d/s side (Fig. 3). An embankment having impervious section, filters and d/s pervious section shall be built in the gap left open. Fig. 4 shows typical cross section of the embankment to be constructed in the gap. The salient features of the embankment are :

Length at crest	=	1150 ft.
Width at crest	=	12 ft.
Maximum height above lowest foundation level		35 ft.

About 1.5 million cubic feet of construction material will be required for the construction of the embankment.

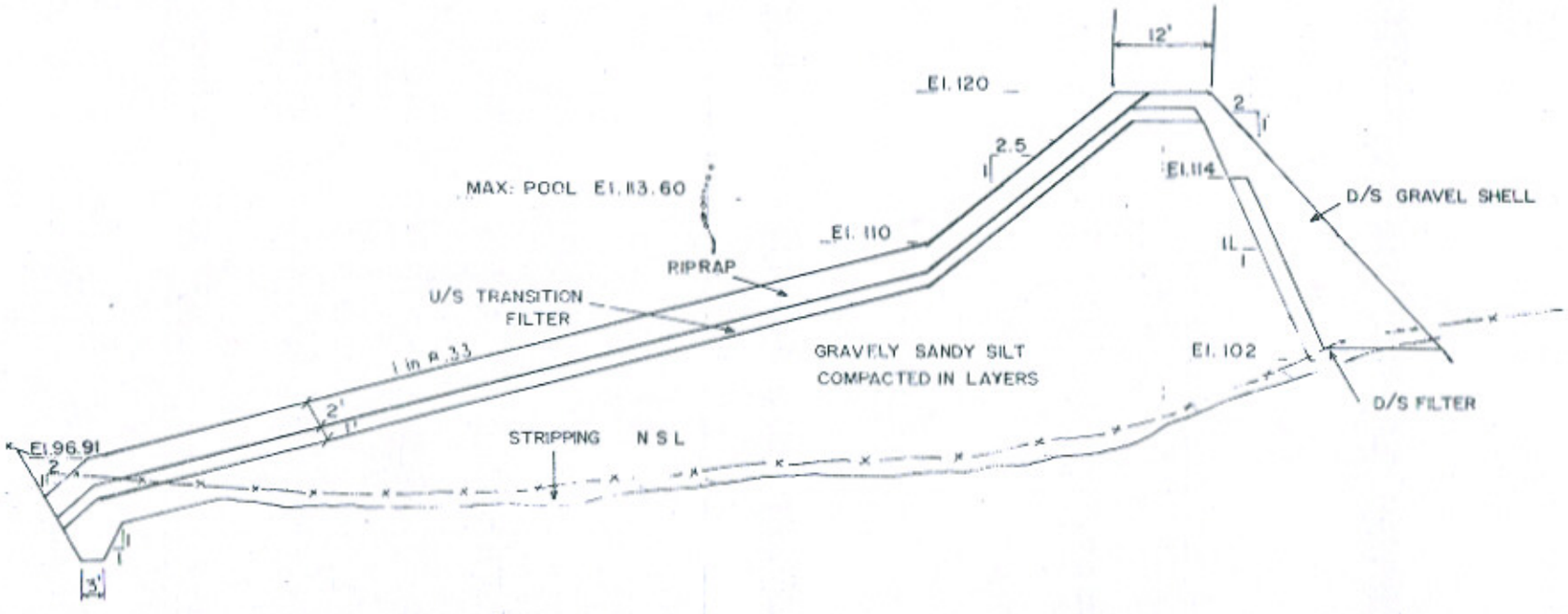


Fig: 4 TYPICAL CROSS SECTION SATPARA DAM (To be constructed in the gap)

Whole of the material will become available from nearby borrow areas without much of quarrying.

The dam will consist of following zones :

i) An existing embankment consisting of silty sandy gravels, loosely compacted.

ii) An upstream blanket and raised portion of embankment will be of gravelly sand silt properly compacted.

iii) A layer of riprap on upstream side of blanket above the minimum drawdown level.

iv) An upstream transition below rip rap.

v) A downstream rock shell.

vi) Transition layer on the downstream side to protect the core and fine grained rock materials.

The upstream face of the dam will have a uniform 2.5 H : IV slope, the downstream face will be constructed 2 H t IV.

7.2 Slope Protection : Rip rap protection will only be required on the upstream slope and blanket. The riprap will consist of a layer of dumped will graded rock, the thickness and gradation of material is based on estimated height of the waves. For a wind velocity of about 60 miles per hour and u/s slpe of 1 : 2.5 the maximum size of stone required is 24 inches and the minimum 12 inches in a total of thickness of 24 inches.

7.3 Impervious Blanket : High seepage is anticipated through the existing embankment. In order to limit the seepage an impervious blanket will be constructed with gravely sandy silty material. The blanket will have upstream slope of 2.5

H : IV and at toe it tapers to 8.33 H : IV to join the NSL. This material will be compacted to 95% Proctor density in 6 inches layers.

7.4 Upstream Transition : A 1.0 ft. thick transition between the blanket and the rip rap will be provided to prevent migration of blanket fines. This transition material will be obtained from the borrow area. The material will be well graded and will have floowing sizes :

Material size	% Finer by Weight
3"	100
¾"	100–65
No.4	40–72
No.200 not exceeding	5

7.5 Downstream Rockfill: The downstream part of the embankment will be constructed of sound, durable and fresh granite rock or compacted phyllite. The material will be graded to have maximum size of 12 inches. The gradation of material will be as follows :

Size	% Passing
12"	100
3"	100–63
No.4	45
No.200 not exceeding	10

The material shall be rolled and compacted in 12 inch lifts at natural moisture

content of the materials.

7.6 Downstream Transition: The downstream transition has been provided between the existing Embankment and the downstream rockfill. The material shall be well graded within the limits specified for upstream transition.

8. Embankment Stability

8.1 Stalility Requirements : The proposed embankment section has been adopted as an integrated design to satisfy seepage, settlement and stability requrements and utilization of materials available near the dam site at an economical distance. The slope stability analysis was made for the most critical loading conditions which will be imposed during construction and reservoir operation. The resultant embankment section provided for upper part of upstream slope 2.5 (H) : 1(V) and downstream slope 2.0 (H) : 1(V)

8.2 Design Criteria : The possibility of shear failure of an embankment is most critical for conditions under which it is subjected to shear stresses in addition to those impsed by its own weight. Several cases which take these forces into account are designated as! sudden drawnown, steady seepage and earthquake. The embankment section has been selected to provide a sufficient factor of safety which is prescribed for the most critical condition.

Load Condition	Factor of Safety
Sudden Drawdown without earthquake	1.20
Sudden Drawdown with earthquake	1.00
Steady seepage without earthquake	1.40
Steady seepage with earthquake	1.10

The upstream slope was analysed for dual loading conditions, under simultaneous applications of instantaneous drawdown and horizontal earthquake acceleration of 0.2g. Earthquake loading was also combined with seepage forces for full reservoir conditions in analysis of downstream face. The pore pressures were determined from flownet analysis in both the cases of sudden drawdown and steady seepage. Soil parameters used were as follows :

Material	Bulk lb/ft^3	Sat lb/ft^3	C lb/ft^2	Degree Ø
Rip Rap and Transition	150.0	156	0.00	35^o
Existing Fill Material	102.5	126.23	0.00	30^o
Downstream Gravel fill	130.00	143.30	0.00	35^o
Compacted Imp.fill Material	110.00	131.00	0.00	30^o

9.0 Conclusions :

Satpara dam project would augment the storage of the lake. It would supply perennial irrigation to 4000 acres of land. It would also help to generate 4000 KW of electricity by utilising 600 ft. of drop from the lake to the river Indus.

The existing embankment shall be

Contd. on Page 38

HYDERABAD BY-PASS BRIDGE DOWNSTREAM OF KOTRI BARRAGE HYDRAULIC MODEL STUDY

By
*Abdul Shakoor**

Introduction :

The National Highway Board undertook the feasibility and design studies for providing a by-pass road to super highway at Jamshoro. It was intended that the by-pass road would connect super highway to national highway. Thus, the traffic would have the facility to by-pass Hyderabad City. The length of this road would be about 20 kilometers and would cross river Indus about 3 miles downstream of Kotri Barrage.

The National Highway Board approached the Irrigation Research Institute (I.R.I.) in 1983, to conduct model studies and finalise the hydraulic features of the bridge design across river Indus on the following hydraulic aspects :-

i) Bridge location
ii) Optimum waterway of the bridge
iii) Shape and alignment of guide Bank.
iv) River Training measures upstream of the bridge location if required.

The model experiments were therefore, carried out at the Hydraulic Research Station, Nandipur of IRI on distorted scales of 1 : 200 & 1 : 36 as horizontal and vertical respectively. The model layout and its limits are shown in (Fig. 1). It incorporated Kotri Barrage, stone pitched Jamshoro Bund along left flank and Kalari Bagar Feeder along the right flank below the bridge. The two bridges further down were also represented.

Bridge Location :

The bridge location was finalised with the Consultants (A.A. Associates, Planners and Consulting Engineers) by mutual discussions keeping in view that the afflux caused by the bridge should in no way affect the hydraulic performance of Kotri Barrage. Also, the road detours should not become expensive. It was decided that the bridge would be located about 3 miles downstream of Kotri Barrage on the main channel with left abutment tied to the existing Jamshoro Bund.

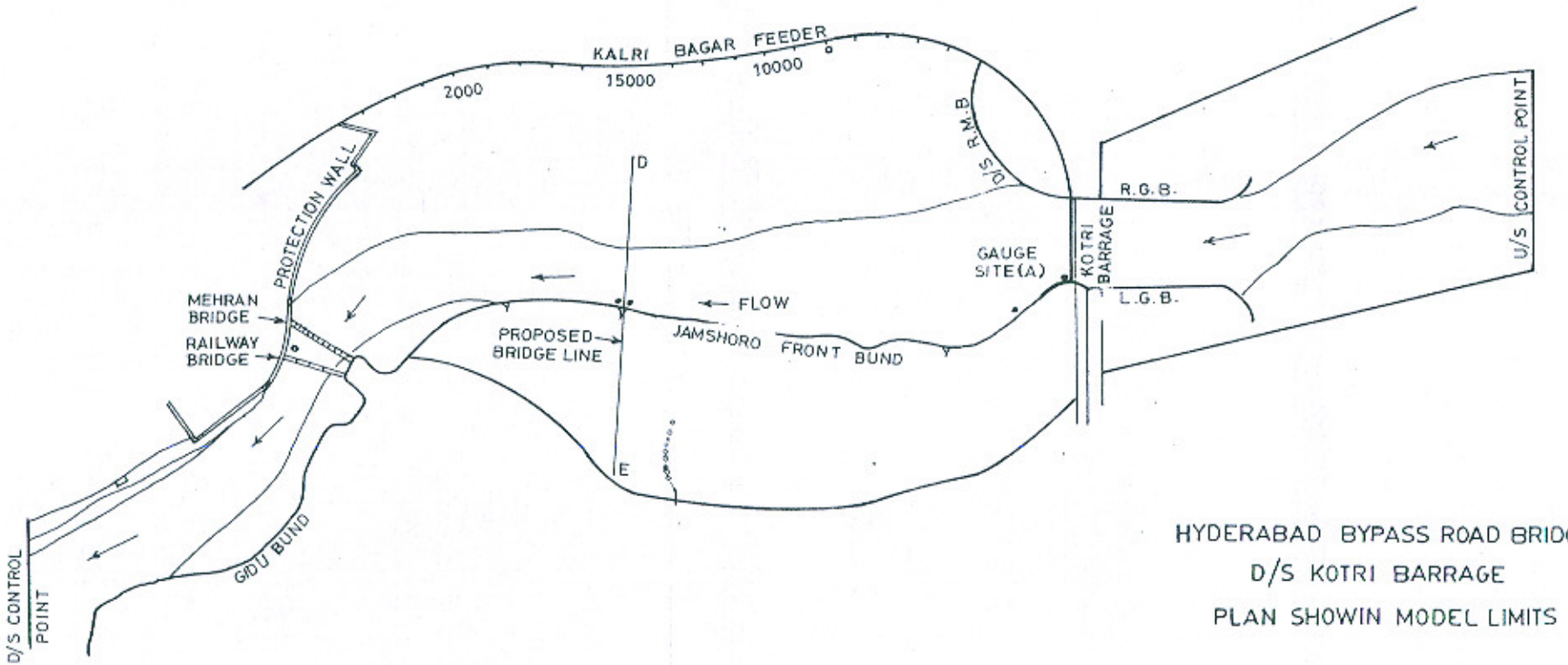
Existing Hydraulic Features at the Bridge Site :

The river Indus downstream of Korti Barrage runs straight along stone pitched Jamshoro Bund upto a distance of 4 miles. It, then turns towards left approaching the two railway bridges. The flood plain of the river is towards right which is bounded by Kalari Bagar Feeder and lower down by right protection wall guiding the flows towards the existing railway bridges.

The initial test runs with hydrograph of the river at Kotri Barrage

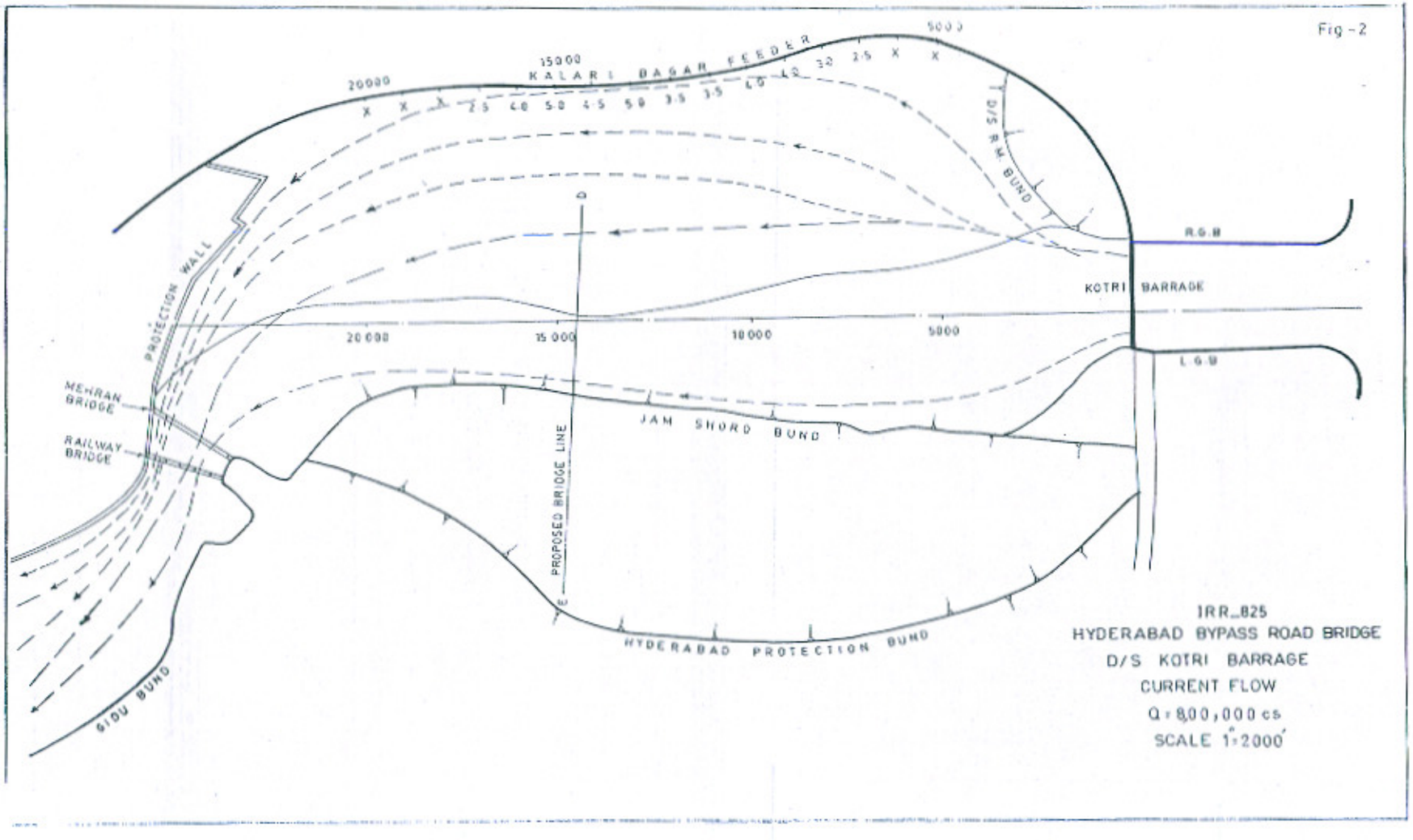
* Principal Research Officer (Hydraulic) Irrigation Research Institute, Lahore.

Fig-1



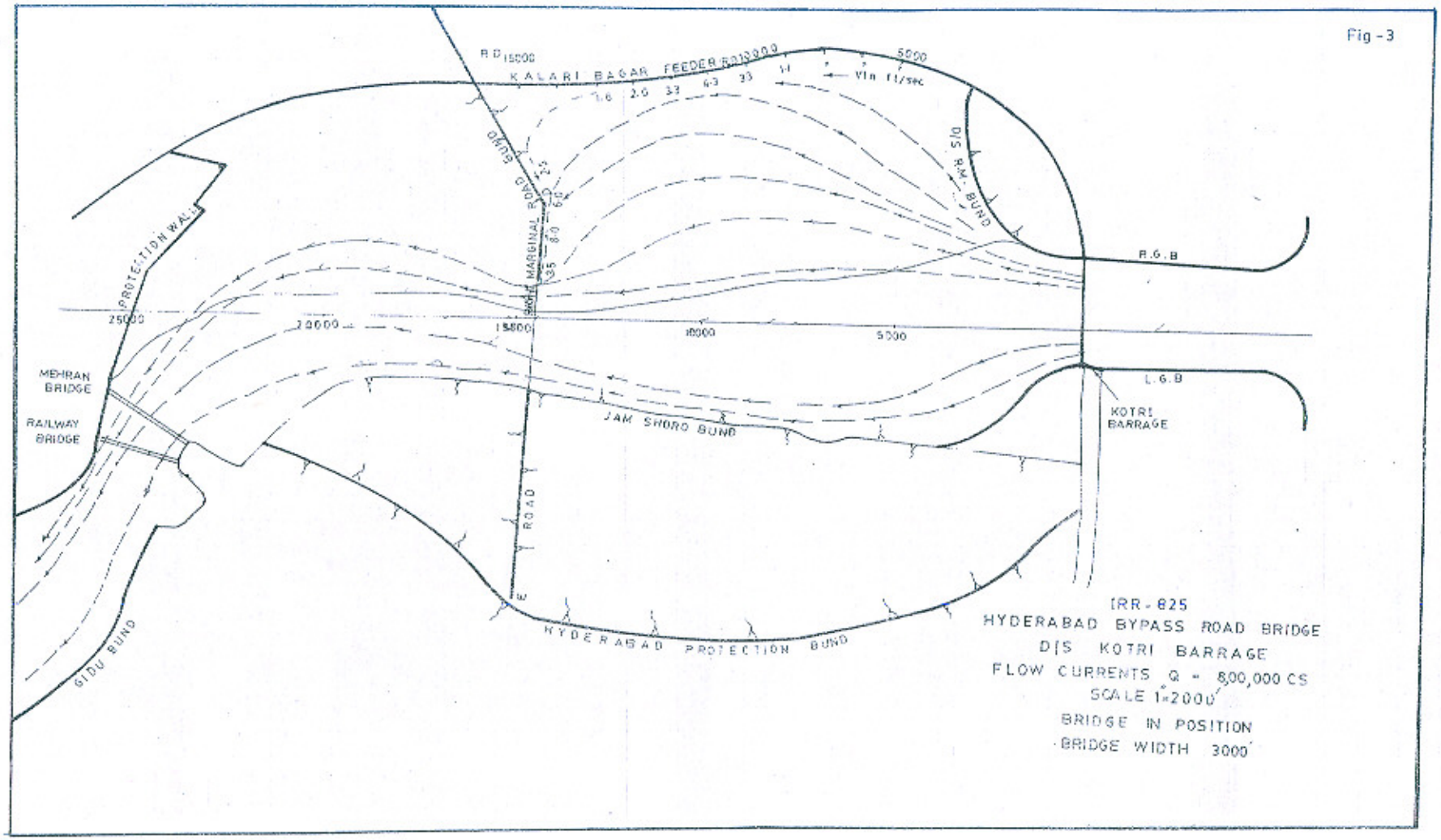
HYDERABAD BYPASS ROAD BRIDGE
D/S KOTRI BARRAGE
PLAN SHOWIN MODEL LIMITS

Fig - 2



IRR_825
HYDERABAD BYPASS ROAD BRIDGE
D/S KOTRI BARRAGE
CURRENT FLOW
Q = 8,00,000 cs
SCALE 1"=2000'

Fig - 3



IRR - 825
HYDERABAD BYPASS ROAD BRIDGE
D/S KOTRI BARRAGE
FLOW CURRENTS Q = 800,000 CS
SCALE 1"=2000'
BRIDGE IN POSITION
BRIDGE WIDTH 3000'

peaking upto 12.5 lakhs cusecs indicated that the river remained within the banks upto 5 lakh cusecs river stage. Above this discharge, the river spilled over the right flood plain bounded by Kalari Bagar Feeder. The spill flow started flowing along Kalari Bagar Feeder with erodible order of velocities (Fig. 2). It then followed the right protection wall upstream of Mehran and Kotri railway bridges with concentration entirely towards right half of these two bridges.

Hydraulic Features After the Introduction of Bridge of Various Widths :

According to drawings supplied by A.A. Associates, the proposed bridge was 3000 ft. wide abutment to abutment, with pile diameter of 6.56 ft. and 26 rows of piles. The bridge had 18 no wider bays of 147.60 ft and 8 no smaller bays of 21.73 ft. The clear waterway as worked out was 2832.3 ft. All the sub-structure elements comprised of reinforced concrete piles taken down 39.36 ft. in the bed rock. The elevation of bed rock at the location of the bridge was at R.L. 0.00.

The proposed width of 3000 ft. of bridge and other widths were tried in model experiments. The test runs made with the bridge in position of 3000 ft. width with looseness factor of 0.95, indicated that the velocity order along Kalari Bagar Feeder reduced due to blocking of the right flood plain by right road embankment. It caused an afflux of 1.7 ft. downstream of Kotri Barrage at 13 lakh cusecs river stage. But, it was noted at the same, time that the parallel velocities of damaging order appeared along the upstream face of right road embankment of the new bridge (Figs. 3-4). This suggested that the provision of right guide bank of suitable length and shape was necessary. As mentioned earlier stone pitched Jamshoro Bund on the left flank would constitute the left guide bank for this bridge. Further tests performed on the shape of right guide bank indicated that the shape of right guide bank tested on model with minimum length of 2540 ft. as shown in (Fig. 5-6) would be suitable. The minimum length of guide bank of 2540 ft. was tested with the objective that any saving in the length of guide bank would be utilised in the form of training works if found necessary for tying with the Kalari Bagar Feeder upstream.

At the same time, it was suggested that the clear waterway of the bridge equal to 3465 should be adopted with looseness factor of 1.1 and width between abutment to abutment equal to 3674 ft. as it reduced the afflux downstream of Kotri Barrage and practically restored the existing conditions.

Provision of Right Side Training Works :

With the finalised shape and length of right guide bank with width of the bridge of about 3674 ft, the experiments also indicated the flow still embayed at the right guide bank nose and the erodible order of velocities still existed along Kalari Bagar Feeder (Figs. 7-8). This suggested thatsome training work from right side was necessary. The provision for this was made by curtailing the length of right guide

bank. The training work was found useful for streamlining the flow entry from the right side flood plain through the bridge. A training work in the form of a J-spur tied to Kalari Bagar Feeder was finalised. The layout plan of bridge project thus finalised is shown in (Fig.9). The provision of this J-spur in the layout plain was recommended on the basis that it would eliminate the cost of (i) stone pitching of the right road embankment in its full length and (ii) extra strenthening of Kalari Bagar Feeder by stone pitching in the affected length.

To sum up, the model experiments aimed at providing sufficient width of the bridge so as not to affect the hydraulic performance of the Kotri Barrage. The width of bridge should therefore be increased form 3000 ft. to 3674 ft. Secondly, the proposed shape and length of right guide bank should be adopted so that it should protect the entire length of right road embankment. It was also intended to keep the length of right guide bank minimum so as to provide out of its savings, a training work upstream tied to Kalari Bagar Feeder to protect it from parallel velocities. This would save the cost of stone pitching in the affected length of Kalari Bagar Feeder and also the cost of stone pitching on the upstream face of the right road embankment in the flood plain. The project is at present, in execution stage and incorporates the modifications suggested by the Irrigation Research Institute Punjab, Lahore in their Report on "Hyderabad By-Pass Road Bridge Downstream Kotri Barrage Hydraulic Model Study, Indus River July, 1984 IRR-825/Hyd/By-Pass Bridge".

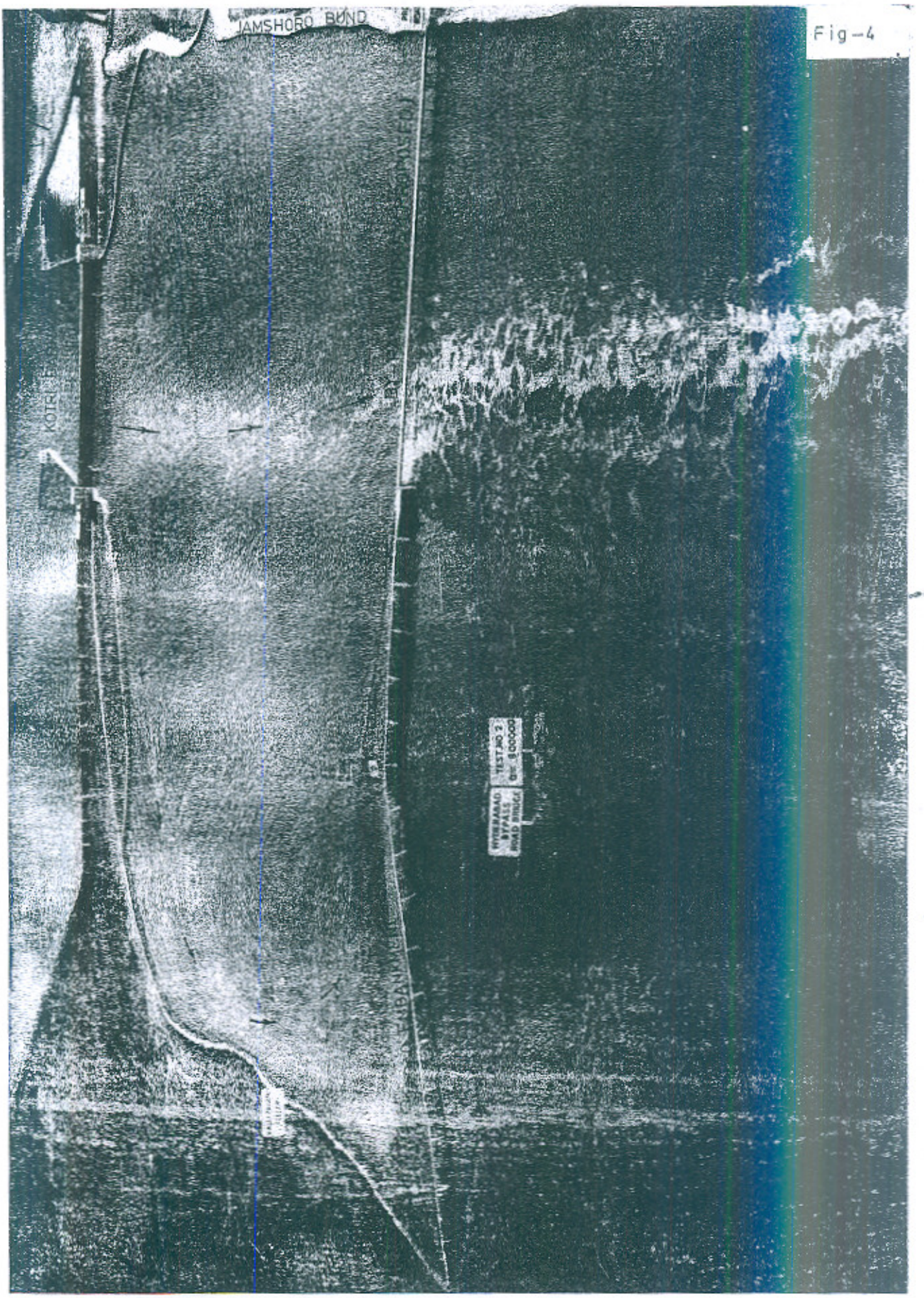
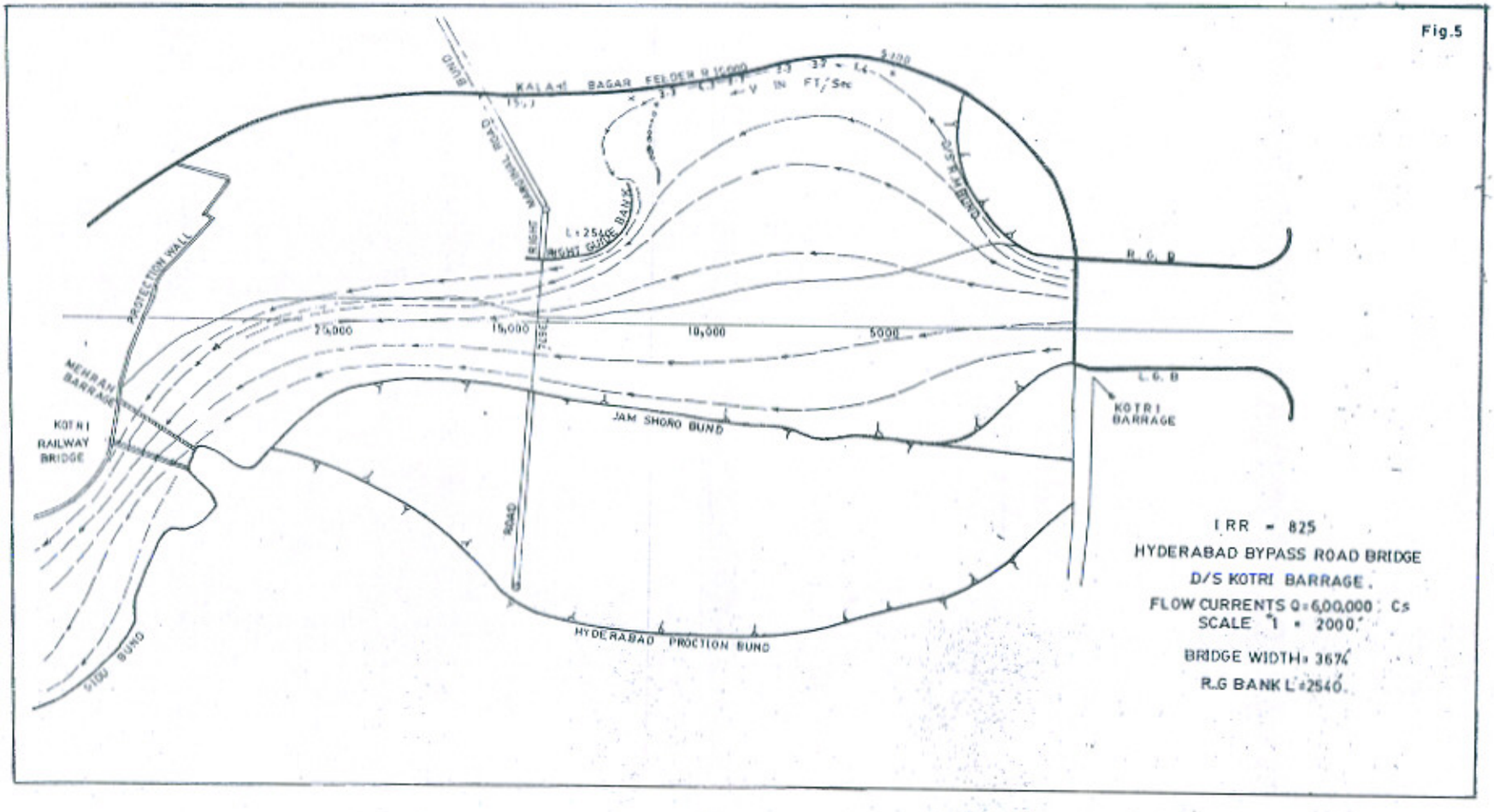
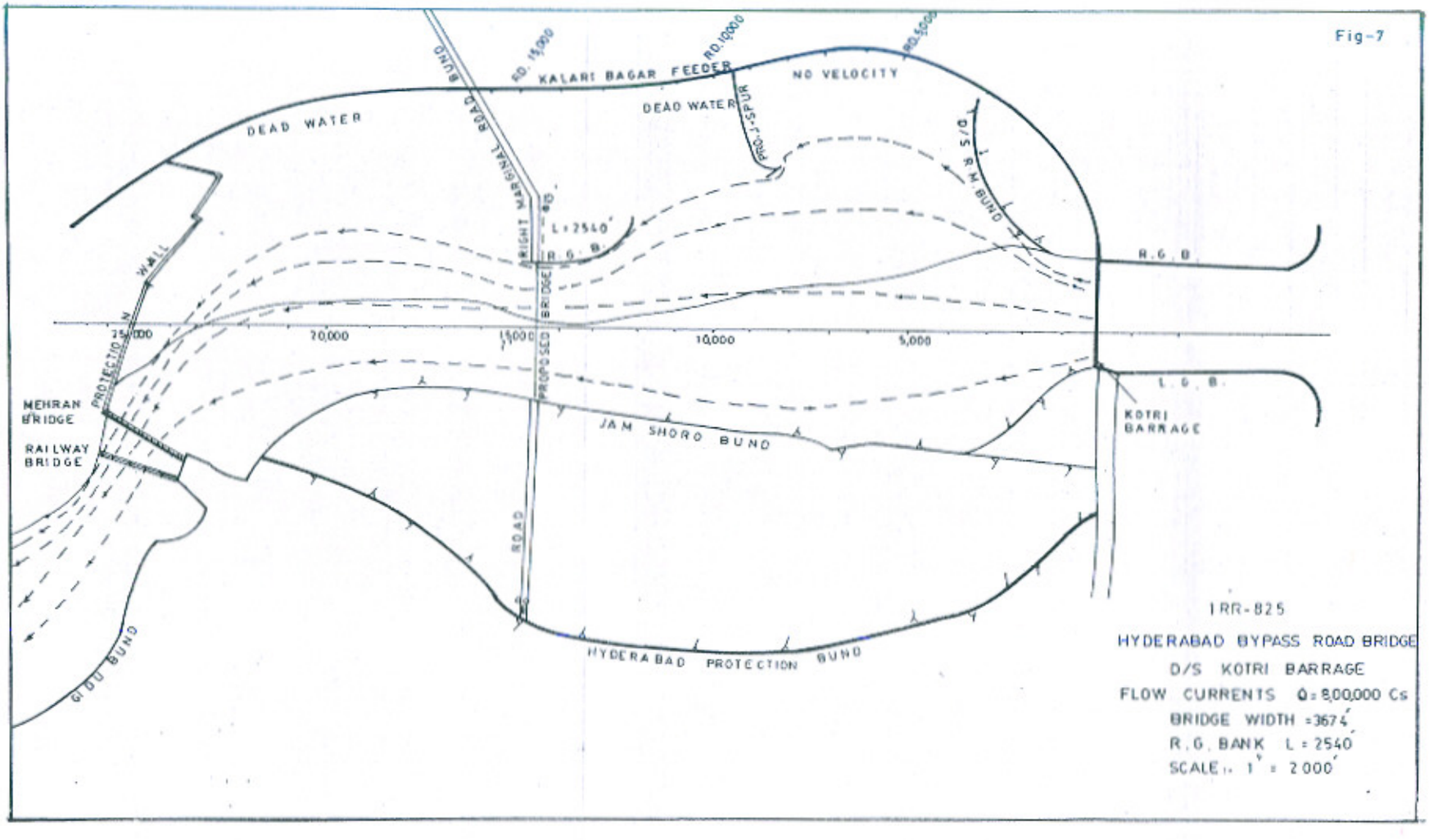


Fig-4

Fig.5


KALAN BAGAR FELDER
V IN FT/Sec
PROTECTION WALL
MEHRAN BARRAGE
KOTRI RAILWAY BRIDGE
SIBBI BUND
MANCHHAL ROAD
RIGHT GUIDE BANK
L=2540
D/S R.M.BUND
R. G. B
L. G. B
KOTRI BARRAGE
25,000
15,000
10,000
5000
JAM SHORO BUND
ROAD
HYDERABAD PROCTION BUND
IRR = 825
HYDERABAD BYPASS ROAD BRIDGE
D/S KOTRI BARRAGE.
FLOW CURRENTS Q=6,00,000 Cs
SCALE 1 = 2000'
BRIDGE WIDTH= 3674'
R.G BANK L=2540'

Fig—6


Fig-7
KALARI BAGAR FEEDER
RD.10000
RD.5000
NO VELOCITY
DEAD WATER
DEAD WATER
RD. 15,000
BYPASS ROAD BRIDGE
RIGHT MARGINAL BUND
L=2540'
R.G.B.
BRIDGE
PROPOSED
WALL
PROTECTION
MEHRAN BRIDGE
RAILWAY BRIDGE
25,000
20,000
15,000
10,000
5,000
R.M. BUND
D/S
R.G.B.
L.G.B.
KOTRI BARRAGE
JAM SHORO BUND
ROAD
GIDU BUND
HYDERABAD PROTECTION BUND
IRR-825
HYDERABAD BYPASS ROAD BRIDGE
D/S KOTRI BARRAGE
FLOW CURRENTS Q=8,00,000 Cs
BRIDGE WIDTH =3674'
R.G. BANK L=2540'
SCALE :- 1" = 2000'

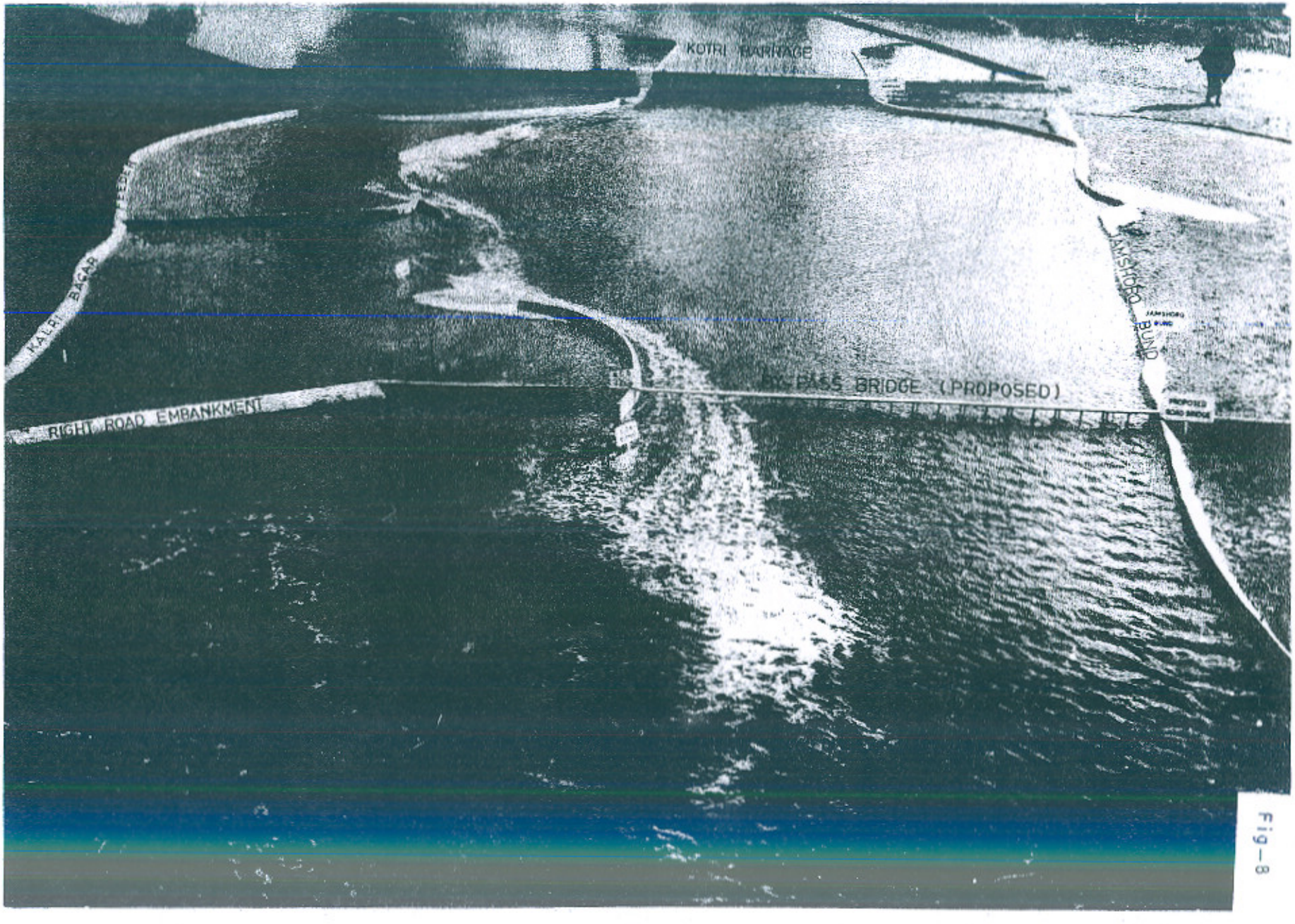
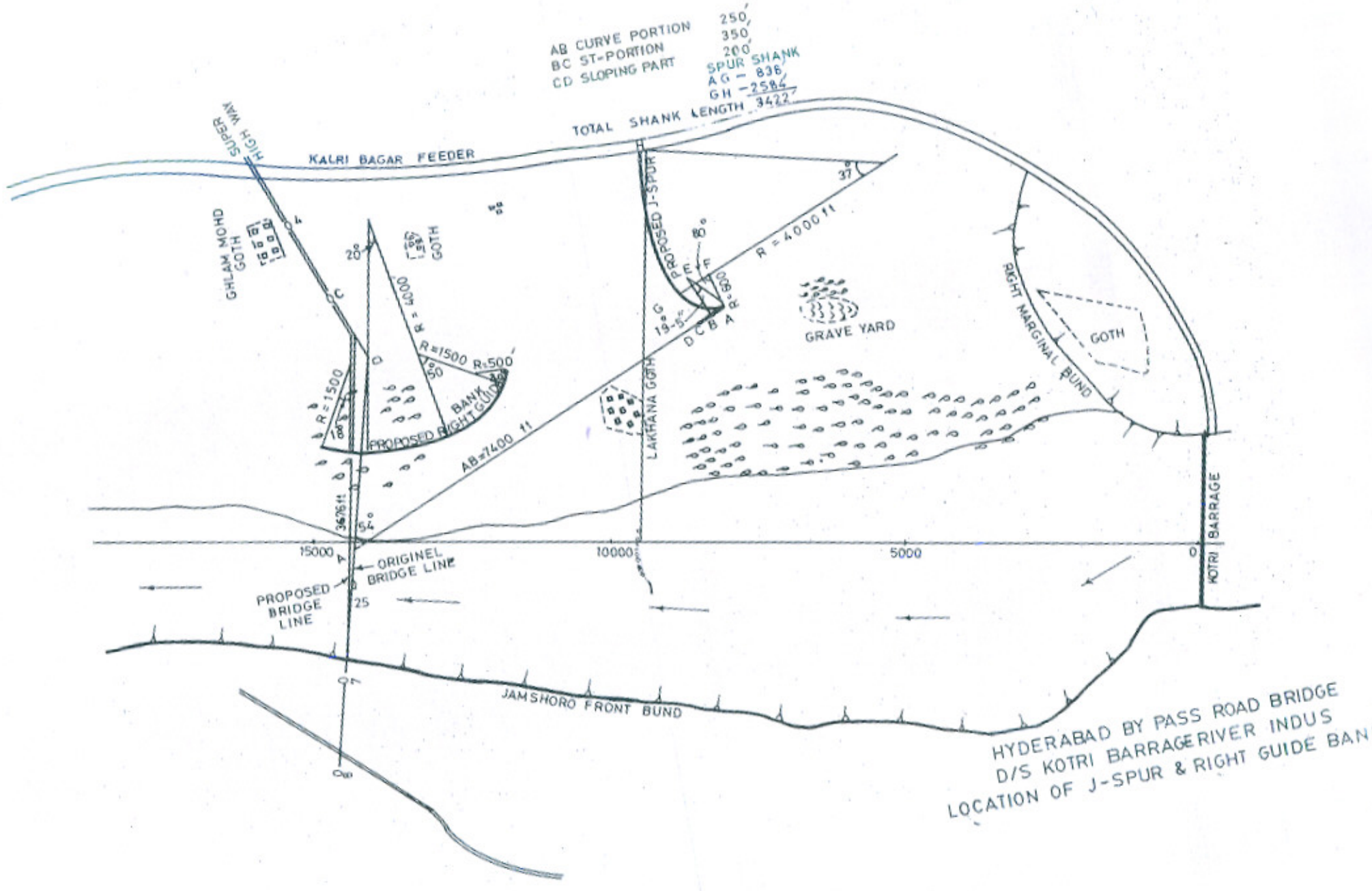

KOTRI BARRAGE
KALRI BAGAR
RIGHT ROAD EMBANKMENT
BY-PASS BRIDGE (PROPOSED)
JAMSHORO ROAD BRIDGE

Fig-8

Fig -9



HYDERABAD BY PASS ROAD BRIDGE
D/S KOTRI BARRAGE RIVER INDUS
LOCATION OF J-SPUR & RIGHT GUIDE BANK

RECLAMATION OF SALINE SODIC LANDS WITH THE APPLICATION OF GYPSUM

By
Ch: Mohammad Altaf Hussain *

Introduction

Salinity and sodicity have been inherrent problems of soils in the arid and semi arid regions. Most of the lands in the canal irrigated areas of Pakistan are also situated within the same regions and face the same problems.

The introduction of canal irrigation system has caused the ground water level to rise to the upper layers of soils which has aggravated the situation further with respect to salinity and sodicity. In the Punjab alone at present 2.66 million acres of the cultivated land (11.11%) is in the grip of salinity and sodicity. This is on the basis of the visual soil salinity survey. The magnitude of salinity and waterlogging problems in Pakistan has been assessed by Planning Division, WAPDA in the country wide survey in the irrigated areas of 34.5 million acres. According to these data 25% of the irrigated land is affected by salinity in varying degrees on the surface. The percentage of salinity and sodicity has been estimated at 38% in the culturable commanded area.

Reclamation of these lands and to keep them in fit condition and to achieve the desired level of production of agricultural crops, is a pre-requisite to ensure the prosperity of the masses in the country.

Reclamation of saline sodic lands involves two components: the removal of salts and the elimination of sodicity. Salts in saline sodic soils are mainly Sodium Chloride and Sodium sulphate which are highly soluble in water. The solubility of Sodium Chloride at the room temperature is 37%; whereas that of Sodium sulphate is 14%. They are easily removed when an adequate amount of irrigation water is applied for leaching but sodicity persists and requires 4 to 6 years to get rid of it by simple leaching which is generally practised in the country at present. It needs replacement of Sodium from the soil with an equivalent amount of some divalent cation.

With the application of a suitable soil amendment which provides the divalent cations the process of reclamation may by completed within a short period of time. For this Gypsum is the cheapest available source in the country.

An experiment has been conducted to demonstrate its usefulness to reduce the period of reclamation of a saline sodic land at the field level and the results achieved therefrom are presented here:-

Review of Literature

1. Kelley (1951) applied gypsum to an alkali soil at the rate of 10–12 and 15 tons per acre. The plots were flooded continuously for three weeks. He found that the exchangeable sodium was

* Land Reclamation Directorate Punjab, Lahore.

effectively removed to a depth of four feet and practically a barren land was converted into a high productive land.

2. Haider (1959) reported that leaching with water alone was in-effective while gypsum in combination with leaching effectively reclaimed a "Bara" soil.

3. R.C. Renord and C.A. Bower (1960) used sea water for reclaiming a sodic soil. The sea water contained 11.6 me1/1 of Ca + Mg. They pointed out that with the use of only 4 feet of Sea water combined with 6 feet of Colorado River water, reduced the initial exchangeable sodium percentage of the soil from 39 to 5 and reduced the time of reclamation from 120 days to 12 days.

4. Bower (1962) recommended that for soils having a gypsum requirement upto 5 meq/1 100 gms of soil growing of rice and green manure crops could be depended upon for reclamation of a saline sodic soil in a reasonable period of time.

5. Zaidi and Qayyum (1968) reported that the rate of reclamation increased considerably with the use of gypsum and farm yard manure and the income obtained with the use of amendments for reclamation is mroe than that with leaching without amendments.

6. Hussain Mohammad (1968). It takes 6–8 years for reclaiming a saline sodic soil by rice culture. Application of gypsum accelerates the replacement of exchangeable sodium.

7. B.K. Khosla and I.P. Abra (1972) studied the effect of using gypsum in fuier grades on the composition of saturation extract of a saline sodic soil. They reportèd that much of the gypsum is utilized in precipitating the soluble çarbonates to form relatively insoluble calcium carbonate. Gypsum of slightly finer than than 0.59 mesh would be more effective than the coarser grades. The amount og gypsum needed to neutralize the soluble carbonates is shown to be indicative to the minimum amount of amendments required to start the reclamation of a saline sodic soil high in carbonates.

8. G.R.Dutt: et al (1972) pointed out that the amounts of water and gypsum required for reclaiming the soil were highly dependent upon the quality of water used for leaching.

9. Mohammad and Khaliq (1975) reported taht gypsum sulpher combination with manure were quite efficient in reclaiming a saline sodic soil.

10. Hussain and Asghar (1985) found out that with the application of Gypsum even though it is applied to the extent of 1/2 or 1/4 of the total requirement of the soil the reclamation of saline sodic soil is accelerated and can be achieved within a limited period of time. The increase in infiltration rate of the soil is almost equal with 1/2 and 1/4 of gypsum requirement applications. They further concluded that rice is a highly sodic resistant crop.

Material and methods

The experiment was conducted in the field at the Mianchannu Reclamation Research Station. The following treatments were tried :–

1. Control (No gypsum application)

2. Application of half dose of gypsum requirement of the soil.
3. Application of full dose of gypsum requirement of the soil.
4. Application of double dose of gypsum requirement of the soil.

The plot size was 1/40th of an acre (49.50'x22'). The soil was fine sandy loam in texture. The soil samples were taken before starting the experiment. The pH of the soil saturation paste was determined and the Electrical Conductivity of the saturation extract was found out. The saturation extract was analysed to know the SAR of the same. The pH and the SAR values of the soil samples showed that the soil was highly sodic in nature.

55.0 Cm canal water was applied within 63 days to leach mist of the salts present in the soil and later on Gypsum was applied on the basis of Gypsum requirement of the soil and was mixed with the upper 15 Cm depth of the soil. The rice crop was transplanted on 4th & 5th Aug. 1980. Ninety 94.0 Cm irrigation delta was applied to the crop and 2.5 Cms rainfall was received during the growth period of the crop. The crop was removed on 26.11 1980 and the soil samples were again taken and tested. The analysis results are given in Table 1. No fertilizers were applied to the crop.

Results and discussions

The results (Table – I) shows that originally the soil was highly saline and sodic in nature. The application of Gypsum and providing irrigation delta for leaching and growing the crop removed most of the salts present in the root zone of the crop. It reduced the pH level of the soil to less than 8.3 in all the cases of gypsum treatments. It shows that Gypsum application in all the treatments has been equally useful for reclaiming the Saline Sodic land. However the sodicity persisted in case of control.

The yield results of the crop (Table–2) indicate that the highest yield of the rice crop was obtaned in case of which control which needs to be explained here. It was because of the reason that originally in two replications the soil was only marginally saline. In the other two replaications where salinity was high, the application of 70 Cm water for leaching removed most of the salts from the root zone of the crop. The sodic condition which persisted caused the dispersion of the soil and reduced its infiltration rate. Thus the water losses from the root zone of the crop would have been less which was a favourable condition for the rice crop. Moreover the rice crop is a highly sodicity tolerant crop. The same has been proved by Altaf and Asghar (1985). In case of double dose of Gypsum, the infiltration rate of the soil would have increased to the extent that it proved harmful for the crop. Hence the crop yield was low. The full and half dose gypsum requirement applications have been useful to remove sodicity. The infiltration rate of the soil in these cases would have improved to the benefit of the soil and the crop did not give better yield in case of the double dose application.

Conclusions

The reclamation of a saline sodic land

is highly accelerated with the application of gypsum. There is no need to apply gypsum to meet the full gypsum requirement of the soil. Even to provide half of the requirement is equally useful to convert a sodic land to a normal productive land.

TABLE – 1

ANALYSIS RESULTS OF THE SOIL SAMPLES.

Sr. No.	Treatment	Plot No.	Depth in cms	Before starting the experiment SP	pH	EC x 10^3	SAR	GR	After completing the Experim SP	pH.	Ecx10^3	S/
I.	Control (Without application of gypsum)	6	0–15	40	8.7	2.2	6.7	1.37	35	7.5	0.7	
			15–30	35	8.6	1.45	4.6	2.75	34	7.4	0.6	
		12	0–15	39	8.6	1.20	4.5	3.44	40	7.4	2.2	2
			15–30	38	8.6	1.00	4.2	2.06	38	7.5	0.8	
		18	0–15	27	9.1	3.00	24.0	6.9	37	8.8	1.30	
			15–30	30	9.3	4.8	52.5	8.94	37	8.3	1.50	1
		24	0–15	29	9.5	5.50	36.5	10.32	44	8.6	3.40	2
			15–30	48	10.0	8.00	53.7	12.38	39	8.7	2.30	1
II.	Gypsum Half Dose	2	0–15	33	8.7	3.4	9.9	3.44	31	7.9	0.82	
			15–30	36	8.8	4.5	7.7	2.75	33	7.8	1.60	
		8	0–15	39	8–5	1.55	3.65	2.75	38	7.5	0.7	
			15–30	41	8.6	0.8	1.70	3.44	37	7.5	1.2	
		14	0–15	33	9.6	5.5	16.4	10.32	30	7.5	2.6	2
			15–30	36	9.8	7.5	66.5	10.32	30	8.5	2.2	2
		20	0–15	25	9.1	2.3	20.9	8.25	38	7.9	2.3	1
			15–30	26	9.7	3.00	28.1	12.38	35	8.2	4.4	3
III.	Gypsum Full Dose.	1	0–15	25	8.9	7.2	64.0	6.88	27	7.6	0.7	
			15–30	33	8.5	8.8	89.0	9.63	32	8.0	1.15	
		7	0–15	39	8.6	.8	2.1	3.44	36	7.5	0.8	
			15–30	39	8.6	.45	1.2	1.37	36	7.5	1.2	
		13	0–15	33	9.7	3.00	20.3	11.00	30	7.6	2.6	2
			15–30	45	9.9	5.5	35.0	11.00	31	8.5	2.2	2
		19	0–15	30	8.8	5.2	49.0	8.25	34	8.9	2.6	
			15–30	32	9.4	1.4	11.40	11.0	37	8.9	1.9	
IV.	Gypsum Double Dose.	3	0–15	33	8.6	1.6	6.5	2.75	31	7.5	1.35	
			15–30	35	8.8	.7	3.0	2.75	29	7.7	.8	
		9	0–15	36	8.3	1.75	9.0x 9.0	6.88	39	7.9	2.5	
			15–30	32	9.2	4.50	22.5	11.0	33	7.7	1.40	
		15	0–15	31	8.5	2.8	16.1	6.19	40	7.4	1.6	
			15–30	32	8.8	3.6	27.2	11.69	36	7.7	1.0	
		21	0–15	27	8.7	7.7	48.8	8.96	38	7.7	3.00	
			15–30	32	9.4	1.25	19.0	6.88	39	8.3	2.6	

TABLE NO. II

AVERAGE YIELD RESULTS OF PADDY IN KGS/ACRE IN CASE OF EACH TREATMENT.

Control	Gypsum ½ Dose.	Gypsum Full Dose.	Gypsum Double Dose.
53.65	32.08	33.85	12.83

REFERENCES

1. Kelley W.P. 1951 ... Alkali soils, their formation properties and Reclamation Rein Hold Publishing corporation, New York.

2. Haider G. 1959 ... Effect of water and amendments on Physical and Chemical properties of alkali soils. MSc. thesis Pb., Univ Lahore.

3. Renord R.C. and Bower C.A. 1960 ... Use of sea water for the reclamation of a sodic soil, soil Sc. V, 90, 139 to 193.

4. Bower C.A. and Fireman M. 1962. ... Soil water chemistry, compilation of studies for the Indus Plan (A report by Harza Engg: Company International and WASID).

5. Zaidi H.S. and Qayyum 1968 ... Reclamation of Saline Alkali soils in the Upper Indus Basin.

6. Hussain Mohammad 1968 ... Reclamation of Saline and Alkali soils in West Pakistan Res. Pub. Vo. II No.19.

7. Khosla B.K. Abra I.P. 1972 ... Effect of gypsum fines on the composition of saturation extract of saline sodic soil, soil sci. Vo. 113, 1972.

8. Dutt G.R. Terkeltoub and Roa Shkoll R.S. 1972 ... Amount of water and gypsum required for reclaiming sodic soils, soil Sci. V. 114 No. 1

9. Mohammad Shah and Khaliq A. 1975 ... Comparsion of various chemicals and Organic Amendments for Reclamation saline sodic soils (Bulletin of ID & FCRC Vo. 5 June, 1975.

10. WAPDA 1979 ... Revised action Gypsum for Irrigation Agricultural Main report.

11. Hussain Mohammad Altaf and Asghar Mohammad 1985 ... Gypsum as an Amendment for the reclamation of saline sodic lands. Pakistan Engg: Congress proceedings Annual Session (1984-85).

ABSTRACT OF PAPERS PRESENTED AT INTERNATIONAL COLLOQUIUM ON CONCRETE IN DEVELOPING COUNTRIES MATERIAL, DESIGN & CONSTRUCTION

The International colloquium on concrete in Developing Countries was held in Lahore, Pakistan from December 14–20, 1985. These colloquium proceedings have been published in two volumes and were made available at the colloquium to all registrants. These include 44 papers from 8 different countries covering all aspects (materials, design, construction and others) of concrete. This International Forum was jointly sponsored by the Institution of Engineers Pakistan, the Canadian Society for Civil Engineering, the University of Engineering and Technology, Lahore.

Abstract of the 23 papers published in volume I relating to materials were put forth for the General interest of the readers and fellow Engineers. The remaining papers of volume II relating to analysis and design are printed in this issue.

24. Recent Development Design of Prestressed concrete structures.

T.I. Camppell
Professor of Civil Engineering
Department Queen's University
Kingston, Canada.

Recent developments in the design of prestressed concrete structures are described from the point of view of developments in constitutive materials, construction techniques are related to bridge structures are examined and the application of the prestressing techniques as a remedial measure for structure is outlined briefly. The use of truss models based on both the compressor field theory and the theory of plasticity is discussed and finally, recent work on continuity in post-tensioned structures is described.

25. Analysis and Design of Reinforced and prestressed concrete slabs — A state-of-the-art report

M. Saeed Mirza
Professor
Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, Canada.

This paper presents a state-of-the-art report of some of the available procedures for design of reinforced and prestressed concrete slabs in North America and the United Kingdom. Different types of slab construction and some essential features of slab behaviour are summarized along with the computer programs available for complete analysis for monotonically increasing loads from zero load until failure. Principles of elastic and limit analyses are summar-

ized. The analytical development and the experimental work leading to the development of code clauses for design of reinforced and prestressed concrete flat slabs are summarily reviewed.

The enhancement of slab strength due to in-plane forces is discussed briefly along with the recent developments leading to the use of drastically reduced slab reinforcement in bridge decks of slab-girder bridges. The basic provisions of the ACI Code, CSA Standard A23.3 M-84 and CP110 for simplified analysis and design methods are summarized.

26. Two-way slab deflections

Andrew Scanlon
Professor
Civil Engineering Department
University of Alberta Edmonton
Canada

The current state of the art for controlling deflections of two-way slabs is reviewed. Computation procedures, loads to be considered, use of minimum thickness relationships, and the variability that can be expected in deflections of actual slab systems are covered.

27. Effects of High Construction Loads on the short and long term deflections of plat slabs

N.J. Gardner
Associate Dean and Professor
J.B. FU
Ph. D. Candidate
Department of Civil Engineering
University of Ottawa, Ottawa, Canada

Typical construction procedure for multi-storey flab slab structures involves casting the new slab onto forms which are supported by one or more previously cast floors. The loads occurring on the supporting slabs are large relative to the slabs' design capacity. The supporting slabs have not developed their design capacity. The large construction loads on immature slabs cause large immeidate deflections. Further, the high stress/developed strength due to construction loads will cause large creep deflections.

This paper describes an experimental investigation into the effect of early age construction loads on the long-term deflections of reinforced concrete flat slabs. The measured long-term deflections were 5 to 8 times the 28-day calculated deflections.

The high construction loads in shored and shored/reshore construction cannot be ignored when considering long-term deflection. Consequently, to ensure adequate deflection behaviour the slab designer must specify the construction schedule, including number of forms and reshores and concrete strength at stripping.

A design method which includes a construction load dependent creep term is presented to calculate the long-term deflections of flat slab structures.

28 Concrete Bridges — A State-of-the-art

Barrington Dev. Batchelor
Professor
Department of Civil Engineering
Queen's University, Kingston, Canada.

A brief review of the state-of-the-art of concrete bridges is presented. The

evolution of concrete bridge construction is discussed together with the main types of concrete bridge superstructures, and particular applications to which they are suited. The pros and cons of precast and cast-in-place construction are considered where appropriate.

Special attention is given to the recent general trends, including the materials of construction, exploitation of partial prestressing, and the use of compressive membrane action to reduce slab reinforcement requirements dramatically. The important role of bridge evaluation and rehabilitation, in an era of scarce financial resources, is also discussed.

29. **Reinforcement Savings in Deck Slab by a new design method**
Baidar Bakht
Principal Research Engineer
Svetlana Markovic
Systems Development Project Engr.
Ministry of Transporation and Communications, Toronto, Canada.

Concrete deck slabs of slab-on-girder bridges are customarily designed for transverse bending. Extensive research in three different countries has conclusively shown that these deck slabs are overdesigned by this approach. An internal arching action, present in these slabs, enhances considerably their capacities to sustain concentrated loads; by taking advantage of this, the reinforcement in deck slabs can be considerably reduced. This paper presents a state-of-the-art on the consideration of internal arching effects in deck slabs, together with some case histories and a methodology using which a simple-to-use empirical method of deck slab design can be developed for any jurisdiction. It is shown that large savings in reinforcement can be achieved by designing the deck slabs by this method.

30. **Concrete Design and Construction in Earthquak Hazard Areas**
S. Tanvir Wasti
Professor
Civil Engineering Department,
Middle East Technical University, Ankara, Turkey

An overview of the state-of-the-art in the seismic design and construction of concrete structures is presented. The response of simple structures to vibratory movements caused by earthquakes is analyzed and related design implications are discussed. The equivalent lateral force method employed in the Uniform Building Code is described. The concept of ductility, which is relied upon in current design procedures, is explained and necessary dutility requirements are briefly given. Som information on the use of elastic and inelastic response spectra is also presented.

31. **Analysis and Design of High Rise Buildings**
Mohamed I. Soliman
Assistant Professor
Civil Engineering Department
Ain Shams University, Cairo, Egypt.

This paper summarizes the most commonly used load-bearing high rise building systems. The basic structural elements of a high rise building are introduced and discussed along with the combination of these basic elements to generate the bone structure of a building. A brief summary of the available

mathods for the analysis of a high rise building is presented. The analysis of a shear wall-frane structure is explained. Finally, a comparison is made between the various methods along with comments on the range of workability of each method.

32. **Optimization study of shear wall-rigid frame design for high-rise buildings**

Zhou Ya-Jun
Lecturer
Civil Engineering Department
Tianjin University Tianjin China

Li Zhu-Jing
Professor
Civil Engineering Department
Qinhua University Beijing China

This paper presents the results of a recent investigation in the optimization analysis of a two span shear wall-rigid frame structural system. The total cost of the structural system was taken as the objective function to be minimized, with column and beam sizes, shear wall thickness, and number of shear wall elements as the design variables, subject to the constraints specified in the currently prevailing codes. The properties of the feasible regions were investigated and the uniqueness of the optimum solution was established.

A "quasi-optimization" process for the column size was proposed. The relationship between shear wall stiffness and the total structural cost was studied and a criterion for the optimum combination of shear wall thickness and the number of shear wall, elements was proposed.

33. **Structural systems for high rise buildings**

Tariq S. Cheema
Associate and Project Engineer
Datum Structures, Inc.
Dallas
U.S.A.

The design of a high-rise structure is greatly influenced by the forces and deflections caused by wind or earthquake forces. In the past two decades, Fazalur Khan (1969) developed a menu of optimum structural systems for tall buildings. Following these concepts, early high rise buildings were constructed in either steel or concrete. In the recent years., however, engineers have started to combine steel and concrete into composite systems, thereby taking advantage of the inherent speed of steel erection and the economy and stiffness of concrete.

In this paper, the author presents two case studies where composite systems have proven to be the most economical. The example buildings are 50 stories and 62 stories tall, respectively. A brief description of their structural systems, design procedures, and construction details are provided.

34. **Strength-Ductility of prestressed Beam-columns elements using high strength concrete**

Shuaib H. Ahmad
Assistant Professor
Civil Engineering Department
North Carollna State University
Raleigh U.S.A.

The research desribed in this paper conducts a computerized parametric study for prestressed beams and beam-column elements of high stength con-

crete. The nonlinear computerized model is based on experimentally obtained stress-strain curves of high strength concrete, and a realistic representation of the stress-strain relationship of the prestressing steel.

The ranges of concrete strengths studied are from 5000 psi to 13,000 psi. The model is capable of handling any desired stress-strain relationships of concrete and prestressing steel. The idealization of the prestressed section is done through the use of strip elements. Results of strength analysis are presented in the form of nondimensionalized load-moment interaction curves and the results of deformation analysis are presented through the use of ductility ratio factors. The results of the strength analysis of prestressed beam-columns elements are compared with the PCI procedure.

35. Behavior of confined concrete — design for ductility

Shamim A. Sheikh
Assistant Professor
Department of Civil Engineering
University of Houston
Houston U.S.A.

After briefly reviewing the need for ductile behavior of reinforced concrete structures for seismic resistance, this paper presents results from in extensive experimental and analytical study of concrete confined by rectilinear ties. Results from the tests on 8 in. (203 mm) and 12 in. (305 mm) square columns under monotonic axial compression are presented. The rariables examined are distribution of longitudinal and lateral steel, amount of longitudinal and lateral steel, tie spacing, precracking of concrete and crossties. Behavior of con-.fined concrete under axial and flexural loads can be evaluated from a proposed analytical model. Analytical and experimental results compare well for the tests reported in the literature by various investigators. In order to simplify the application of the proposed model to members under combined loading, a rectangular stress block representing the actual stress-strain curve is proposed. Equations and graphs to determine the dimensions of the stress block are pretented. The provisions of the building codes (particularly ACI 318-83, 318-IBR) for the confinement steel in ductile frame columns are critically examined. Several important variables which affect confined concrete behavior are not considered by the codes which makes their confinement requirements either too conservative in some situations or unsafe in others.

36. Physical modelling of reinforced and prestressed concrete structures

Svetlana Markovic
Systems Development Project Engineer
Ministry of Transportation and Communications of Ontario
Toronto Canada

John Wyzykowski
Intermediate Engineer
Pratt and Whitney Canada Toronto Canada

M. Saeed Mirza
Professor
Department of Civil Engineering and Applled Mechanics
McGill University Montreal Canada

Principles of model analysis are summarized and the similitude relationships for elastic and ultimate strength models are presented. Materials and

construction and testing techniques are briefly reviewed with emphasis on plastics and model concrete. Provisions of the various national building codes and standards permitting the use of physical models as a design tool are presented.

Applications of small-scale physical models in the design of some important structures are presented as case studies. These include complete model studies of the CANDU nuclear reactor, development of prestressed concrete box girder bridges for use on Ontario bridges and post-tensioned fiat plate siab systems, seismic modeiling of vertically irregular structures and wind tunnel studies on the Calgary Coliseum.

37. Ferrocement—design, construction and applications

S.L. LEE
Professor and Head

P. Paramasivam
Associate Professor

M.A. Mansur
Senior Lecturer
Department of Civil Engineering
National University of Singapore
Singapore

This paper summarises the design concepts, construction techniques and some possible applications of ferrocement in building industry and highlights recent research and development work carried out at the National University of Singapore. Several studies have been conducted on the application and performance of prototype ferrocement structures such as water tanks, flume, roofing elements, sunscreens, permanent formwork for slabs, and wall panels. The design, construction and testing of these ferrocement elements are briefly treated.

38. Foundations of concrete structures

Stephen G. Revay
President
Revay and Assciates Limited
Construction Management
Consultants
Montreal Canada

This paper describes some of the better known construction methods available to install deep foundations for concrete structures in general and for bridges in particular. This discussion is limited to the proper application, and the advantages and disadvantages of the various methods, such as :

1. Land cofferdams,
2. River cofferdams,
3. Slury trench walls,
4. Open well caissons,
5. Compressed air caissons, and
6. Sand islands

The application of these methods will be illustrated during the presentation, with slides depicting "real life" examples taken from the experience of both the author and others.

39. Concrete pile foundations — a review

S.Q. Ahmad
Project Engineer
C.Mirza
Principal
Principal
Strata Engineering Corporation
Toronto Canada

Concrete is an abundantly available

material in developing Countries. The use of concrete piles in foundation engineering applications should be encouraged, with a view to learning more about them in the prevailing environment of these countries and also with a view to prevailing environment of these countries and also with a view to establishing local industries for their manufacture and/or installation.

In this paper, various foundation pile types are discussed in general with particular emphasis on concrete piles. For these a general overview is given from well known published sources. In addition, the Paper includes the results of several reliable load tests reported in the literature for various types of concrete piles. Such test data may be found useful in forming judgements about the load carrying capacity of different concrete pile types and in corroborating theoretically derived load carrying capacities for such piles.

40. Rehabilitation of building and bridge structures

Gajanan M. Sabnis
Professor
Civil Engineering Department
Howard University Washington
U.S.A. and President
Technology Transfer Consultants, Inc, Silver Spring Maryland U.S.A.

This paper presents an overview of rehabilitation of structures. The overview is based on the ideas and views of the author with his experience in the United States; however, it is also applicable to other countries. The main topics addressed are : (a) reasons, (b) problems with materials, stresses and loads, (c) techniques, (d) codes and — guidelines, and (e) future potentials for this type of work. In the last five years he was responsible for arranging and coordinating international symposia and seminars on the subject.

41. Towards a code of practice for design & construction of concrete structures in developing countries

K. Mahmood
Professor
Civil Engineering Department
King Abdulaziz University Jeddah
Saudi Arabia.

Z. Mian
Professor
Civil Engineering Department
University of Engineering and Technology Lahore - Pakistan

M. Saeed Mirza
Professor
Department of Civil Engineering and Applied Mechanics
McGill University Montreal Canada

The paper discusses the need for national codes of practice for design and construction of concrete structures in developing countries. The basic philosophy and the essential components of some of the current codes for some developed and developing countries are considered briefly and compared. The recent in many parts of the world towards similired codes is reviewed along with the ideal qualities desirable in a code. Finally some recommendations are proposed for the development of a code of practice for design and construction of concrete structures in developing countries.

42. Craneles precast housing system

Mickhail Linetsky

Consulting Engineer
Toronto Canada

This paper presents proposals for a jobsite method of manufacture and erection of concrete well panels and ceiling/roof slabs to form separate or adjacent precast bays of single storey houses, shelters and other structures.

43. Manufacture of prestressed concrete poles in Pakistan

M.R. Chaudhry
Chief Engineer (Civil)
Wapda Lahore - Pakistan

The Project for the manufacture: of Prestressed Concrete Poles consisting of four Pole Plants for the Provinces of Punjab and NWFP was planned during 1974-75 as two Pole Plants of smaller capacity had already been installed at Kotri and Rohri in the province of Sind. The four new plants are located at Lodhran, Chichoki Mallian, Bullaand Hill and Jehangira. The total cost of the four plants was Rs. 113.13 million including foreign exchange provided by CIDA of Rs. 15.02 million require for the import of plant and equipment not available in Pakistan. The manufacturing of Prestressed concrete poles was started first time in Pakistan during 1964 for the transmission lines from Sukkur Thermal Power Project.

There are several advantages in the use of Prestressed Concrete Poles e.g. resistant to corrosion, no foundation, economical, safe from accidents, saving in foreign exchange as all materials for manufacture of poles are available locally.

The poles being manufactured at the six plants are of 31,36 and 55 feet sizes alongwith cross-arms for 132 KV lines where two 55 feet poles are used. These poles are normally used for 440 Volt, 11 KV and 66 KV transmission and distribution lines. The 31 and feet poles are mainly used for rural electrification project in the provinces of Sind, Punjab and North West Frontier (NWFP).

44. Precast shell pan roofing: An innovation in the concrete structures

B.N. Hira
Assistant Director

S.K. Jain
Scientist
Central Building Research Institue
Roorkee India

The cost of construction of instu flat slab is becoming very high due to the increased cost of shuttering as well as of materials. The speed of construction is also not satisfactory. Innovation in roof by prefabrication, simple shape of units, and ease in construction is therefore needed. This paper describes a recently developed innovative roofing technique which can economically replace the less durable conventional sloped roofing systems. It consists of precast full span RCC shell units which are cast, cured an and transported in stacks one above the other. Due to casting in stacks, the production cost is reduced, the space for site activities are minimised and the curing of concrete is simplified. After placement of units the joints in the roof are filled up with insitu concrete. Due to providing grooves at edges of units, the joints become leakproof and structurally monolithic. Though used mainly as roof, it can however, be converted into a flat floor after filling the haunches with RCC. The experimental

prototype roofs made of precast shell units were found quite strong and yielded considerable saving of cement, steel and cost. On-site prefabrication further reduces the transportation cost and mass factory production of the shell units is commercially viable.

The paper contains the method of production of shell units, construction of roofs and other salient features. It also presents the results of test and experimental investigation of the prototype roofs. Design and construction guidance for mass uses has also been added briefly in the paper.

٥۔ وَلَا تَأْكُلُوٓا أَمْوَالَكُمْ بَيْنَكُمْ بِالْبَاطِلِ وَتُدْلُوا بِهَا إِلَى الْحُكَّامِ لِتَأْكُلُوا فَرِيقًا مِّنْ أَمْوَالِ النَّاسِ بِالْإِثْمِ وَأَنتُمْ 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 مِّنَ الظَّنِّ إِنَّ بَعْضَ الظَّنِّ إِثْمٌ وَلَا تَجَسَّسُوا وَلَا يَغْتَب بَّعْضُكُم 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.