STUDY OF FAILURE OF REINFORCED CONCRETE HIGHWAY BRIDGES IN EAST PAKISTAN

A

THESIS

BY

SK. ABDUS SALAM

SUBMITTED TO THE DEPARTMENT OF CIVIL ENGINEERING IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF SCIENCE IN CIVIL ENGINEERING

MAJOR SUBJECT : STRUCTURAL ENGINEERING

JANUARY, 1967

STUDY OF FAILURE OF REINFORCED CONCRETE

HIGHWAY BRIDGES IN EAST PAKISTAN

A
THESIS
BY

SK. ABDUS SALAM

Approved as to style and content by :

Chairman of the Committee

(Dr. W. Ahmad)
Professor & Head of the Deptt.
of Civil Engineering.

Member

(Dr. A. Hasnat)
Professor of the Deptt. of
Civil Engineering.

Member

(Dr.S.H.K.Eusufzai)
Associate Prof. of the Deptt. of
Civil Engineering.

Member

(Mr. H. A. Khan)
Chief Engineer, Roads and Highways,
Government of East Pakistan.

<u>C O N T E N T S</u>

A C K N O W L E D G E M E N T

The author acknowledges gratefully the continued guidance and encouragement given by Dr. W. Ahmad, Professor and Head of the Department of Civil Engineering.

Grateful acknowledgement is expressed to Dr. M. A.Rashid, Vice-Chancellor of this University, who was kind enough to allow the author accompany him during investigation of failure of a bridge in Kushtia.

The author takes this opportunity of acknowledging the encouragement and occasional guidance given by Dr. S.H.K. Eusufzai, and Dr. A. Hasnat.

Sincerest thanks are offered to the Directorate of Roads and Highways, Government of East Pakistan, for helping the author in visiting sites of a large number of bridges.

The author acknowledges endless indebtendness to his dearest parents, to whom this work is dedicated, whose affectionate care and encouragement made this work possible.

SYNOPSIS

Field observation on the nature of the failures and investigation into the causes of four major bridges and ten minor bridges were undertaken. The theoretical studies for apparent causes of failures were made and presented in this thesis.

With a view to forming a general idea about the characteristics of the soil on which any structure is to rest and behaviour of the rivers on which a bridge is to be built, a comprehensive study of the geology and the river system of the province was made. It appears that the physical properties and the mechanical behaviour of the soil differ greatly from place to place attributing to the erratic behaviour and flows of the river system of the country. And on the basis of this study the province may reasonably be divided into four zones.

It appears that the main factor causing the failure of the bridge over the Mathabhanga River in Kushtia district was due to defect in construction arising out of incorrect placement of the reinforcements. Reinforcements in the well-cap were inadequate and placed without concrete covering.

The structural failure of Nayarhat Bridge is due to incorrect placement of top reinforcements in the cantilevered sidewalk and probably due to lack of proper bond in the zone of welded reinforcements. Cracking in piers is possibly due to stress concentration on the thin steining walls of the piers.

The nature of the failure of the Mirpur Bridge indicates lack of bond between reinforcing steel and concrete.

The failure of the Tongi bridge is perhaps due to over-loading. It is difficult to say at this stage whether the failure is also due to inadequate supervision or faulty construction. The review of the design does not show any defect.

Insufficiency of waterway caused scouring at the river beds and the abutments of a number of bridges in Faridpur and Barisal districts. The approaches to Kamaldi Bridge and the abutment of Chokedar Bridge in Faridpur were washed away during flood of 1964. Failure of wing walls is very predominant in this area.

The abutments of Bhangaghat Bridge leaned backwards due to insufficient number of piles under the heel of the abutment and use of cohesive soil as the back-fill. The abutments were later converted into piers by adding extra shore spans.

Horizontal cracks in the masonry abutments of a number of bridges on Demra-Daudkandi Road are due to non-functioning of the expansion bearings.

Settlement of pier was noticed in a bridge near Feni in Noakhali.

CHAPTER - I

INTRODUCTION

CHAPTER - I

INTRODUCTION

1.1. General Remarks :

The province of East Pakistan is intersected with a network system of rivers and rivulets. In order to increase facilities for communication, highways, railroads and water transport system have been developed ; but highways developed more repidly than waterways and railways. The reason is that highways are less expensive than railways and less time-consuming for transportation than waterways. However, the bridging of rivers proved to be the main obstacle in the extension of highways. In spite of it highways developed at a rapid rate during the last two decades. One good example is the Dacca-Aricha Road where there is a large number of bridges and culverts. Another such example is the road between Demra and Daudkandi.

Excepting a few steel bridges, almost all highway bridges in the province are made of reinforced concrete. Although continuous and rigid frame bridges are there in the province, simple span and balanced cantilever bridges are more common.

A number of these R.C. Highway Bridges failed. Some of them had to be replaced by new bridges and some could be repaired. Failure of costly structures like bridges causes wastage of money and materials. It is, therefore, necessary to investigate into the causes of failure of these bridges.

1.2. The Problem :

The problem treated in this thesis is the study of failure of a number of R.C. highway bridges in East Pakistan. Investigations into the causes of failure of some of these bridges were undertaken, the probable causes of failure analysed and suitable measures for probable remedy discussed.

Before independence, there were a few good highways in the province and most of the bridges were in the form of brick arches. There is little scope for study of these old bridges. The bridges under investigation are comparatively young and about fifteen years old at the maximum.

Bridge failure may be of two types : failure of foundation media and structural failure. Failure of foundation may be due to scouring and settlement; structural failure is generally due to faulty design, bad quality of materials, lack of proper supervision and defect in construction.

Cases of bridge failure are scattered all over the province except in the northern districts. In the districts of Faridpur, Barisal and Noakhali, failure in the foundation is probably more prominent. There are stray cases of failure in various districts of the province.

In connection with the present investigation, a comprehensive study of the river system and geology of the province is to be made in order to form a general idea about the characteristics of the soil on which a bridge is to rest and behaviour of the river on which the bridge is to be made.

The objective of this study is to find out why a number of bridges failed and what probable precautions might have saved the catastrophe.

1.3. Programme of work :

The first phase of the work consisted in the study of the geology and river system of the province - the physiographic division, the topography, the formation of the basin, the soil, the ground contour, the distribution of rainfall in the province which influences the discharge from catchment areas in and outside the province, the confluents and tributaries and the influence of floods.

The investigation into the failure of a bridge requires the study of the stratum of soil on which the foundation rests, checking the design of both the superstructure and the substructure of the bridge and an on-the-spot investigation of the failure pattern.

When all stages of investigation are over, a complete analysis of the probable causes of failure and suggestions as to how these failures could have been avoided are then to be made.

After thorough study of causes of failure all over the province, East Pakistan would be divided, if possible, into distinct zones where a particular type of bridge and its foundation may be decided upon.

1.4. Scope and importance :

The province has a large number of railway bridges made of steel girders and trusses. These steel bridges have stood the test of time and there has been no major failure. The present investigation centres round the recent failure of Reinforced Concrete Highway Bridges.

Structures like bridges are very expensive. Failure causes huge wastage of money and materials. The dismantling of a bridge causes an additional expenditure which cannot be avoided when another bridge is to be constructed there.

The engineers or the contractors who were responsible for the construction were not always available during this study, because engineers were transferred in the meantime to a different project. So the difficulties, if any, faced during construction could not be discussed.

The failure of a bridge may be due to insufficient and inaccurate data adopted for the design of the bridge. Hasty construction and lack of supervision might also have contributed to failure.

Bad planning may be a cause of failure of some bridges. The expected loading on a highway increased tremendously in recent years resulting in overloading of the bridges.

Unprecedented high floods flowed over the province for a number of times and it has become almost a yearly occurrence in recent years. The flood water scours the foundation and causes foundation failure.

It appears that no systematic study of the causes of failure of R.C. highway bridges in the province was undertaken before and no literature on it is available.

1.5 RIVER SYSTEM OF EAST PAKISTAN

1.5.1 General:

East Pakistan is a delta excepting the Chittagong Hill Tracts (9). It is formed by alluvial deposits. This deltaic area is formed by deposition of materials carried by the Ganges, Brahmaputra and Meghna rivers. The three mighty river systems have a catchment area of 48,000 sq. miles in East Pakistan (having a total area of 55,000 sq. miles approx) and a still larger catchment area outside the province. The three rivers unite in East Pakistan and discharge their combined waters through a common channel to the Bay of Bengal. The total discharge of the rivers also constitutes the enormous water from the catchment area of the three rivers outside the province.

East Pakistan has a flat slope towards the south (Map.1.5). So the river systems have developed meandering courses in the same direction. In general, most of the rivers have shallow depth, soft mud bottom and small slopes. The banks remain unprotected and the rivers cut new channels deserting the old ones, particularly during the floods.

1.5.2 Origin of the Basin of East Pakistan:

Geologists are of the opinion that long ago, the whole of the Bengal Basin was under water. The rivers deposited sediments year after year. Gradually land emerged out of water. Lowlying marshy lands like the "beels", "haors" etc. were formed. Gradually levees were formed and the higher plains originated. More load carried by the rivers was then deposited on the plains to raise the surface of the basin. The finer silts are carried by the rivers and deposited on the mouths of the sea giving birth to new triangular land called delta ; but the coarser sediments are deposited off from the sea. As more and more load comes and gets deposited, the natural compaction is achieved. Thus the whole of the Bengal basin of which East Pakistan is a part was born. Though natural compaction was achieved, the land made from alluvial deposits remained weak and bearing capacity of such land is naturally low.

1.5.3 River Behaviour during flood :

All the rivers are tidal near the sea. The effect of tide goes a long way in the upstream of the rivers.

Floods of exceptional magnitude sometimes flow over the province. There is flood in East Pakistan every year due to the over-flooding of the banks of the main rivers. But a high flood will require the synchronous flood peaks of the Ganges and the Brahmaputra rivers due to simultaneous heavy rain-fall in their catchment basins. From experience of very recent floods it is clear that the mighty rivers are overflooding every year and the high floods have also become a yearly occurrence. The flood current scours the river beds and deposits sediments on higher land immersed in water during flood (6).

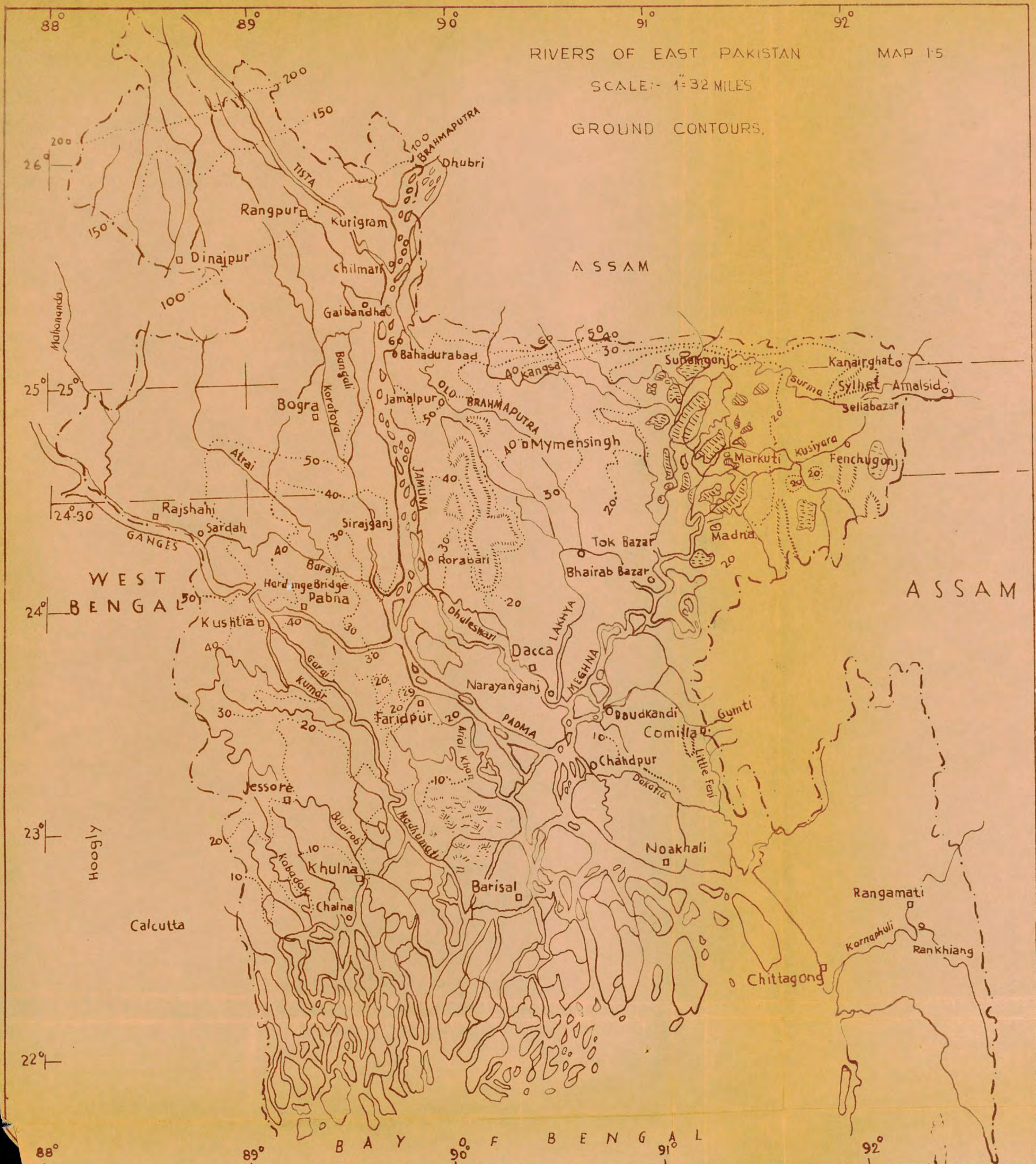
1.5.4 Classification of Rivers :

The rivers of the province (Map 1.5) can be classified into three distinct groups :

1) Perennial : Rivers belonging to this group originate from the Himalayas and has a very large catchment area. The principal members are the Ganges, the Brahmaputra and the Meghna with their innumerable tributaries and spill channels which ultimately discharge their combined waters in the Bay of Bengal. The Ganges and the Brahmaputra are the two most extensive delta-builders with a high flood discharge. The Meghna is not that active and lively.

2) Torrential : These rivers originate from low hills of eastern part of East Pakistan and neighbouring states. The principal members are Gumti, Feni, Halda, Karnaphuli, Sangoo and Mathamohoory. Unlike the perennial rivers they have smaller and localised catchment areas. During heavy rain-fall their flow is torrential and short-lived. During the dry season the flow is very low.

3) Tidal : The lower reaches of the rivers of the above groups within the tidal zone form this group. The



RIVERS OF EAST PAKISTAN
MAP 1·5
SCALE:- 1"=32 MILES
GROUND CONTOURS.
ASSAM
WEST BENGAL
BAY OF BENGAL
Rangpur
Kurigram
Dinajpur
Chilmari
Gaibandha
Bahadurabad
Jamalpur
Bogra
Mymensingh
Rajshahi
Sardah
Sirajganj
Rorabari
Tok Bazar
Bhairab Bazar
Hardinge Bridge
Pabna
Kushtia
Dacca
Narayanganj
Daudkandi
Comilla
Chandpur
Faridpur
Jessore
Khulna
Chalna
Barisal
Noakhali
Chittagong
Rangamati
Rankhiang
Calcutta
Dhubri
Sunamganj
Kanairghat
Sylhet
Amalsid
Seliabazar
Markuli
Fenchuganj
Madna
BRAHMAPUTRA
TISTA
OLD BRAHMAPUTRA
JAMUNA
GANGES
PADMA
MEGHNA
LAKHYA
Dhuleswari
Atrai
Karatoya
Bangali
Mahananda
Baral
Garai
Kumar
Arial Khan
Madhumati
Bhairab
Kobadak
Hoogly
Kangsa
Surma
Kusiyara
Gumti
Little Feni
Dakatia
Kornaphuli

rivers of this group are very active in building delta on account of distribution of the vast unconsolidated silt deposit at their mouths carried by upland floods.

1.5.5 The Ganges River (Map 1.5):

The river flows mainly in the Indo-Gangetic plain, bounded by the great Himalayas on the north and the central Indian plateau on the south. After flowing for a distance of about 100 miles from its source near the Gangatri glaciers down the steep slope of the Himalayas, the river flows in a south-easternly direction till it enters the delta of Bengal.

After following a south-easternly course for about 140 miles, it meets the Brahmaputra near Goalundo at latitude $23^{\circ} - 50'$ and longitude $89^{\circ} - 50'$. Some speculate that four hundred years ago the Ganges followed the Bhagirathi-Hooghly course and was entirely separate from the Brahmaputra. The present combined course follows a south-easternly direction for about 65 miles in the name of Padma river and meets the Meghna river at Chandpur. The combined river flows almost south with the name of the Meghna.

On its way in East Pakistan it receives discharge of only one important tributary, the Mahananda meeting the river near Godagari, but throws off a number of spill channels of which the Mathabhanga, Gorai and Arial Khan are worth-mentioning. According to some, the Ganges flowed through these rivers successively.

The Ganges has a total length of about 1600 miles and a drainage area of about 35,000 sq.miles of which only 12,000 sq. miles are in East Pakistan.

The maximum recorded flood level of the river at Hardinge Bridge was 49.36 P.W.D. on August 24,1910 and the maximum recorded peak discharge was 2.160 million cusecs on August 29,1962 (7). The minimum recorded water level was 19.70 on April 29,1936 and discharge was 0.047 millinn cusecs on the same date.

1.5.6 The Brahmaputra River (Map. 1.5):

The river Brahmaputra has a total length of 1800 miles and a catchment area of 224,000 sq. miles. Born in a glacier in Tibetan plateau under the name of T sang - Po, the Brahmaputra flows for half of its length in a trough more or less parallel to the Himalayan range along the northern foot-hills of the Himalayas. It then veers northeast, and after taking a hairpin bend in Assam turns southwest. After receiving two of its confluents, the Dibang and Luhit, the combined stream takes the name of Brahmaputra and flows for about 450 miles through the Assam valley and enters East Pakistan where it flows about 170 miles in a southernly direction till it joins the Ganges river near Goalundo. In East Pakistan, it receives discharge from a large number of tributaries of which

Dud Kumar, Dharla, Teesta and Karatoa, Atrai, Hurasagar are worth-mentioning.

The old Brahmaputra and the Dhaleswari are the only two spill channels of the river in East Pakistan. Total drainage area above the junction with the Ganges is about 224,000 sq. miles of which 18,000 sq. miles lie in East Pakistan. Of the remaining drainage area, about 113,000 sq. miles lie in Tibet and 93,000 sq. miles in Assam (India).

The river recorded the highest flood level of 65.65 (P.W.D.) at Bahadurabad on August 28,1958 with corresponding maximum discharge of 2.519 million cusecs. The minimum recorded water level at Bahadurabad is 38.4 on February 24-26, 1952, while the minimum discharge is 110,000 cusecs on 30th March,1960 (5).

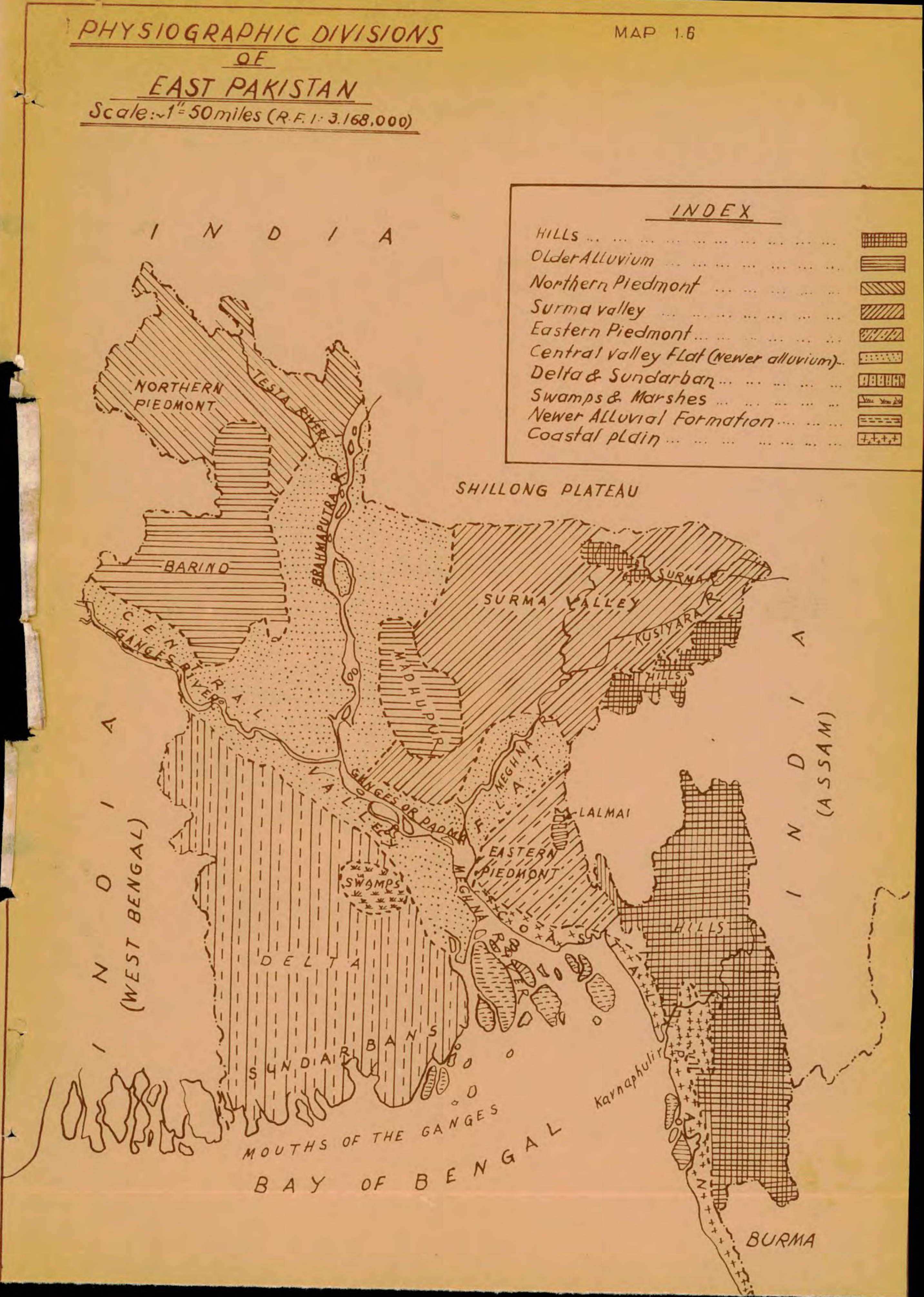
1.5.7 The Meghna River (Map - 1.5):

The Barak river rising in the Assam Hills enters into East Pakistan and bifurcates into the Surma and Kushiyara near Amalsid. The Surma which flowing by the northern part of the district of Sylhet receives discharges from a number of tributaries flowing south, of which Sarainadi, Singer Khal and Piyaingang are worth-mentioning. The Kushiyara on the other hand flowing by the south of the district receives supplies from a number of tributaries flowing north via Jurinadi and Manu River.

The Surma and Kushiyara again join together at Markuli and take the name of Kalni river and flows in a southernly direction. Near Kuliarchar the Ghorantra river meets the Kalni river and the combined course takes the name of Meghna and flowing in a southwesternly direction meets the combined flow of Ganges-Brahmaputra river system at Chandpur.

The Barak Meghna system is about 500 miles long of which about 260 miles are in East Pakistan. Total drainage area above Bhairab Bazar is about 25,000 sq. miles of which 8,000 sq. miles lie within the province.

The maximum recorded flood level of the Meghna river at Bhairab Bazar was 25.10 (P.W.D.) on August 21,1955 and the maxm. discharge was 456,000 cusecs. The minimum recorded water level was 3.00 on February,1952. Occasional tidal discharge measurements in dry season indicate the influence of back water from the Padma.


PHYSIOGRAPHIC DIVISIONS
OF
EAST PAKISTAN
Scale:- 1" = 50 miles (R.F. 1: 3.168,000)
MAP 1.6
INDEX
Hills
Older Alluvium
Northern Piedmont
Surma valley
Eastern Piedmont
Central valley Flat (Newer alluvium)
Delta & Sundarban
Swamps & Marshes
Newer Alluvial Formation
Coastal plain
INDIA
NORTHERN PIEDMONT
TESTA RIVER
BRAHMAPUTRA R.
BARIND
CENTRAL VALLEY FLAT
GANGES RIVER
MADHUPUR
SHILLONG PLATEAU
SURMA VALLEY
SURMA R.
KUSIYARA R.
HILLS
MEGHNA
LALMAI
GANGES OR PADMA
EASTERN PIEDMONT
MEGHNA R.
SWAMPS
DELTA
SUNDARBANS
COASTAL
INDIA (WEST BENGAL)
INDIA (ASSAM)
Karnaphuli
MOUTHS OF THE GANGES
BAY OF BENGAL
BURMA

1.6 GEOLOGY OF EAST PAKISTAN

1.6.1 Physiographic Divisions of East Pakistan :

The whole of East Pakistan can be divided into a number of physiographic divisi ns (8) depending on the land and river pattern (Map. 1.6).

A) The northern piedmont plain:

The alluvial plain ies between the most northern boundary of East Pakistan and a line running south of Dinajpur and Rangpur towns.

The rivers like the Mahananda, Purnabhaba, Teesta, Atrai and Karatoya flow over the area and drain the southern foot-hills of the Himalayas. The streams and rivers of this area have constantly changed their courses in the past. Many old beds of rivers are found in this area. Like other alluvial plains the channels here are also shallow and they have developed meandering courses. The Teesta valley of this area is liable to frequent floods. During a disastrous flood of the Teesta river, the Brahmaputra was forced to change the original course (which is now known as the old Brahmaputra) and flow into its recent channel.

Various types of soil are encountered in this region, especially old alluvium mixed with kankar, clay and loam.

B) The Barind :

The Barind, the lateritic highland, extends south eastwards upto the Jamuna. It appears again in the Madhupur jungle between the districts of Dacca and Mymensingh. It is intersected by the Meghna valley. Again, a small tract is found in the Lalma i-Maina mati Hills ; two isolated tracts are found in Sylhet district, one in the north-east of Sylhet and other in the Chattak Hills. The characteristic of the Barind area is massive argillaceous mound which is a pale reddish brown soil mixed with kankar. Here again a number of rivers has developed meandering courses.

C) The central valley flat :

This area includes the flood plains along the Padma and the Jamuna (Brahmaputra). Meghna flat is also a part of this plain. From the confluence of these two great rivers, the central valley flat extends in three directions along the three courses of the main rivers. The deposit is made of fine silt. This plain is flooded each year and major portion remains under water during the monsoons.

D) The Surma Valley :

This extends from south of the Shillong plateau in the district of Sylhet upto the old Brahmaputra. This area is a triangular depression bounded by the Patharia Hills and hillocks of the Tripura State in the east, composed of lime-stones and sandstones of past origin, and hillocks composed of sand and clay. Vast low-lying areas called "Haors" are the characteristic of this valley. The central and eastern portion of this valley presents the usual characteristics of alluvial tract. The Surma, Kushiyara and the upper course of the Meghna are the main rivers of this valley.

E) The delta of the Sundarbans:

The vast area lying north of the Bay of Bengal upto the central valley is delta, the southern portion of which is a dense forest called the Sundarbans. Innumerable rivers discharge into the Bay of Bengal through this delta. A number of islands is formed at the mouth of the rivers by the silt and sediment carried down by them. The compaction of original formation and new formation of land is very active in this area. In the Faridpur district of this area there is a number of swamps or low marshy lands. The lower half is saline and there is a network system of small "khal" in the Sundarbans.

F) Eastern piedmont:

This area is surrounded by a succession of low range of hills composed of sand-stone and clay of geologic past, while the central portion has been formed by alluvial deposits of sand and clay.

G) Coastal plain:

This extends from south of the eastern piedmont along the coast upto the end of the Chittagong Hill Tracts.

The stretch of land from Cox's Bazar to Taknaf is formed of sand and shales. Huge deposits of sand-stones and lime-stones are found in St. Martin's Island which marks the southwest point of the province.

H) Hills:

The area of the Sylhet valley bordering the Tripura State, some portion of Chittagong and the whole of Chittagong Hill Tracts are hills. These hills have developed forest of small and big trees. These hills are composed of rock of various types, viz. quartzite, laterite, sandstones, lime.tones etc.

1.6.2 Geologic Formation of East Pakistan :

The land of East Pakistan can be divided into three broad geological divisions (1) :

1) Old and new alluvial deposits
2) Deltaic coastal deposits
3) Hills

The piedmont alluvial plains in the north Bengal, a patch of this in the Lalmai-Mainamati hills in the east, and in the Madhupur Jungle are old alluvium. This old alluvial deposit is also found in some places of Sylhet and the foot-hills of the Himalayas. In the clay beds of the basin are encountered massive beds of calcareous clay ; but boulders and gravel with sand are obtained near the hills. A few beds of compact sand or gravelly conglomerate occur in some places at depths. These indicate higher gradient of rivers in the past.

1) Older Alluvium : These are older deposits of lateritic origin. This highland is in northern Bengal with patches found in Madhupur and Lalmai-Mainamati.

Newer Alluvium: These are deposited by the sides of the channels. The river valleys in the older alluvial tracts are deposition of newer alluvium. The south part of the East Pakistan delta is intersected with network system of rivers, and the channels and channel-sides, in fact, most of the area of the basin, are formed of newer alluvium.

It is formed of repeated alternations of clay, sands and marls with recurring layers of peat, ligmite and forest beds.

2) Deltaic coastal deposits:

The Bay of Bengal is the stage of sedimentation of the rivers falling in it, because the valley flattens and the velocity of rivers diminish. The process of sedimentation takes place before and after each tide. The downflowing fresh water confronts the denser saline water and sedimentation starts as water over a vast area gets stagnated.

3) Hills:

The gently rolling hill area of the eastern part of the country rises from the alluvial valley to elevation of about 1000 ft. above sea level. The geological formations in this area are the materials that have been weathered to their present surface configuration. This region, because of its general elevation, is free from floods and has good load bearing capacity.

C H A P T E R - II

FAILURE OF MATHABHANGA BRIDGE

2.1 Location and description

2.2 Bridge failure

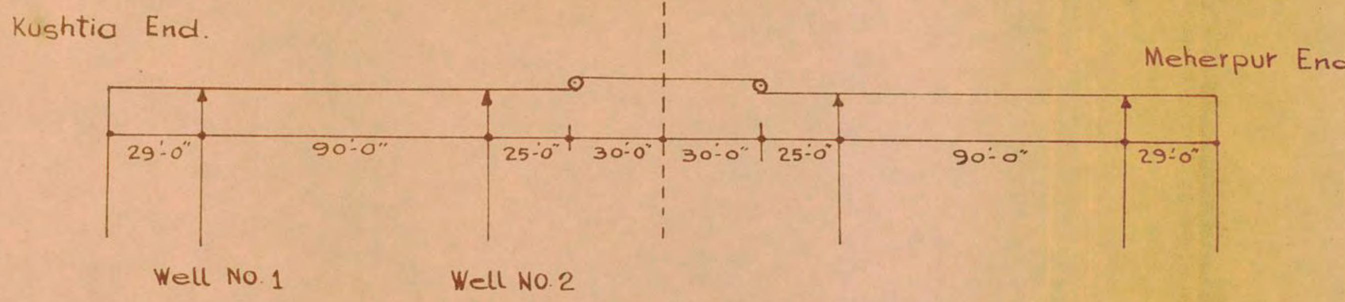
2.3 Preliminary to investigation

2.4 Preparation for investigation

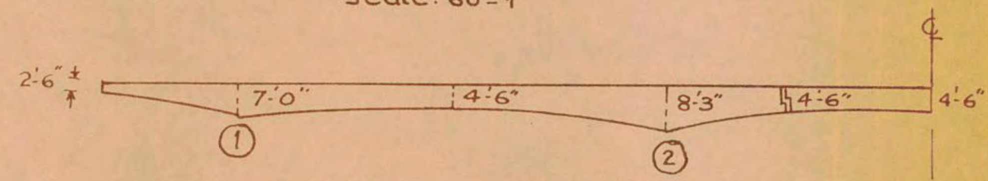
2.5 Observations

2.6 Causes of failure

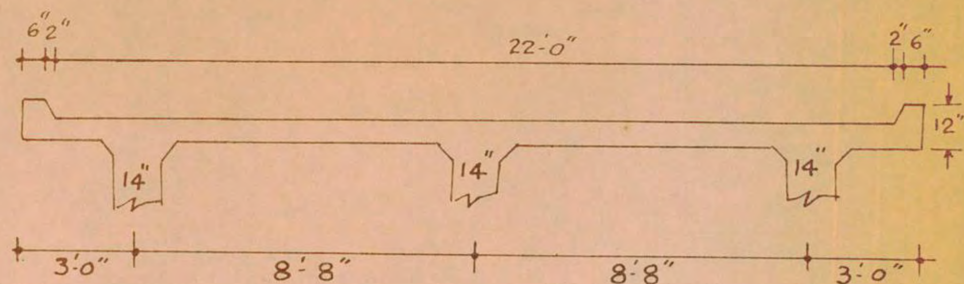
MATHABHANGA BRIDGE



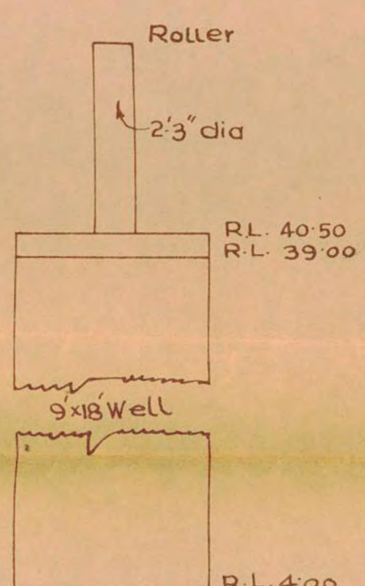
SPAN ARRANGEMENT
scale: 60'=1"



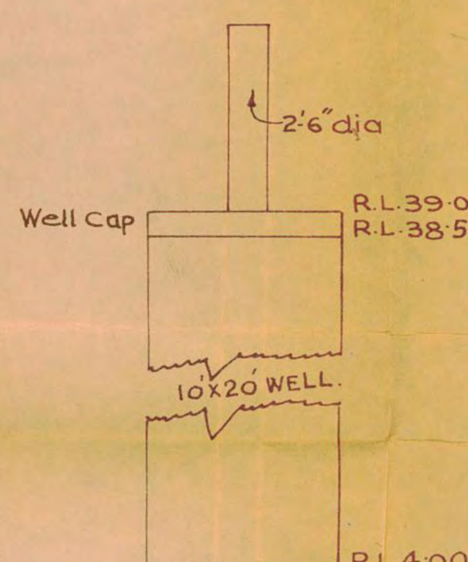
LONGITUDINAL GIRDER
Scale: 40'=1"



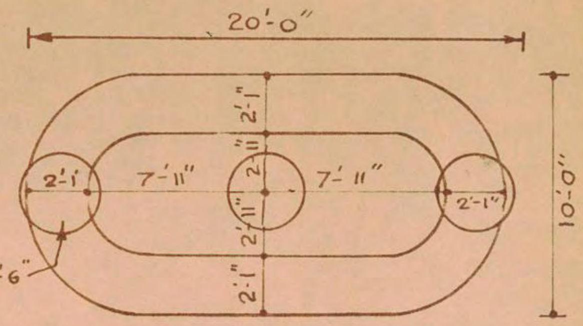
CROSS SECTION
Scale: 5'=1"



WELL NO. 1.



WELL NO. 2.



PLAN WELL NO. 2.

FIG. NO. 2·1

CHAPTER - II.

FAILURE OF MATHABHANGA BRIDGE

2.1 Location and description:

The bridge is at the seventeenth mile on Kushtia-Meherpur Road across the river Mathabhanga.

This is a balanced cantilever two-lane highway bridge of total length 348 ft. ; the suspended span is at the middle. The piers rest on well foundation (fig.2.1).

River Mathabhanga takes off from the Ganges and flows through the district of Kushtia (Map 1.5). Once the Ganges flowed through Mathabhanga as one of its intermediate channels before forming the Padma-Meghna system. In the monsoons the river carries a huge load and overfloods the banks, whereas in winter the river flows through a very narrow channel. The bridge is on a bend of the river. The Meherpur side of the river bank has been brickpitched and wire-netted to stop erosion during flood.

The bridge is in the deltaic zone of East Pakistan south of the Ganges. The soil is alluvial.

2.2 Bridge failure

The bridge was opened to traffic in August,1965. The bridge failed two months later at the time when a state

SOIL INVESTIGATION OF MATHABHANGA RIVER.

Scale:-

Hori. : 1" = 40'

Ver. : 1" = 20'

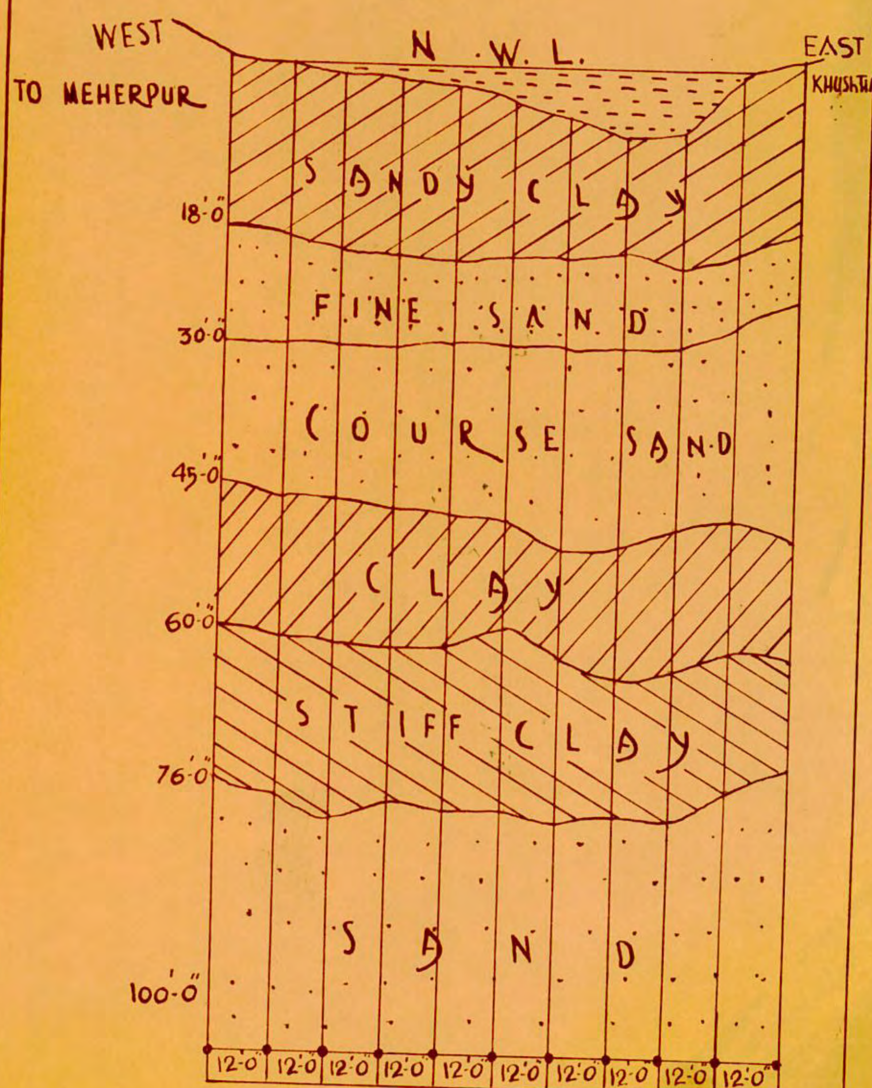


FIG. 2.3

of emergency was prevailing in the country. Heavy military trucks and equipments were carried over the bridge in September during this period.

In the dawn the bridge collapsed and fell down in the river with three to four loud reports. The second pier on the Kushtia side went deep into the river through the well-cap (Plate 2.4).

2.3 Preliminary to investigation:

a) The design of the bridge was checked. The design loading was $H_{20} - S_{16} - 44$. There was no defect in the design of the superstructure. The well-cap was not designed. Cap thickness and reinforcements were provided from experience.

b) The soil boring data were collected (Fig. 2.3). The test boring shows the different strata of soil encountered at different depths. The depth of the well of the second pier below the river bed was 42'-0" indicating that the well rested on coarse sand (11).

c) Some deviations from specifications were noticed. The roller was shown in the drawing to be of 5" diameter. The AASHO specifications put the minimum diameter at 6".

If 5" dia. rollers are used, the length should be:

Permissible bearing on expansion rollers, according to AASHO -

$$= \frac{p - 13,000}{20,000} \times 600d$$

[where d = diameter of roller in inches;

p = the lesser yield point in tension of steel in the roller or the base, = 33,000 psi]

$$= \frac{33,000 - 13,000}{20,000} \times 600 \times 5 = 3,000 \text{ lbs/in.}$$

So length of rollers should be $\frac{110,000}{3,000} = 37$ in.

where 110^k is the total load (including dead load and live-load) on each column.

So the length provided (10 x 3 = 30 in.) was less than required.

2.4 Preparations for investigations:

An embankment was built round the damaged well No.2 in the river bed on the Kushtia side. As water was pumped out of the well, water seeped through the ring embankment. So the river bed round the well could not be seen and thorough investigation was not possible. Thereafter a second embankment was built and water could be pumped out completely (Plate 2.4).

The cantilevers carrying the short suspended span was removed by cutting concrete to small fragments. The concrete fragments were removed from the bridge site to facilitate efficient investigation of the river bed.

2.5 Observations :

a) The suspended span lies in the river towards the downstream side of the alignment. This deviation is a few feet. The fallen superstructure has swung towards the downstream side by nearly seven inches.

b)The rollers are displaced from their seats.(Plate 2.3).

c) The half of the bridge on the Meherpur end is standing alright. No damage has been noticed anywhere in that part of the structure.

d) There was no top plugging of well No. 2 which failed and caused the catastrophe.

e) The well-cap is shown in the drawing as 1'-6" thick, but it was found to be 2'-0" thick.

The rollers provided were each 4½ in diameter and 10 inches long.

f) The cracks in the girder (Plate 2.5) have developed possibly due to the impact on falling.

g) The cap of the well was found to have split longitudinally into two parts and the half towards the river has broken into two parts transversely. The half towards the bank is quite intact without further damage.

h) The well-cap was doubly reinforced ; the reinforcements were 3/8" dia rods at a spacing of 6 inches longitudinally and transversely ; but the spacing was not maintained throughout the entire cap. The spacing was as high as 10 inches at some places. The top layer of reinforcement in the well-cap was about 6 inches from the top surface of the well-cap. The bottom reinforcements were just spread over the well at the bottom of the well-cap without any protective concrete covering. Moreover, the spacing was not always maintained and cut pieces of short lengths were used with or without hooked ends.

i) The three circular columns of pier No. 2 have pushed through the river bed and are resting inclined to the vertical. The column on the upstream side has fallen in the adjacent pocket of the well, and after proceeding to a certain depth through the sand-filling, has pierced through the steining wall towards the river, and has smashed the wall upto a considerable depth before it went deep in the river bed.

j) The middle column has fallen in the downstream pocket of the well, and it has pierced through the steining wall on the river side and gone deep in the river bed exactly in the same manner as the upstream column.

k) The downstream column has fallen outside the well and gone deep into the bed of the river.

l) The tie beam of the columns is now resting on the well-cap and the column tops are in contact with the tie beams, though the tie beam is damaged at the junctions with the column heads. Though the concrete at the column head has been crushed the reinforcements in the column head are in contact with the tie beam.

m) The steining wall towards the bank has been damaged only in a few upper layers of brickwork. Except three to four layers of brick at the top, the rest is in position. The middle wall could be located deep down into the well because the mid-steining wall has crushed to pieces particularly in the upper portion. The steining wall on the river side could not be located because the columns have broken it into blocks during penetration and blocks of masonry are found scattered in the river bed.

n) The size of well No. 2 was designed to be 10 ft. x 20 ft. and 43 ft. deep from the top of the well cap. A tube well was sunk into the well. The bottom plug was reached at 42'-6" below the ground level. So the wells were sunk up to the required depth.

o) There was no scouring at the well. There was no evidence of settlement of the well.

p) The strength of concrete used was verified at a number of places by a concrete hammer in different parts of the structure which revealed that the concrete developed the required strength.

2.6 Probable causes of failure:

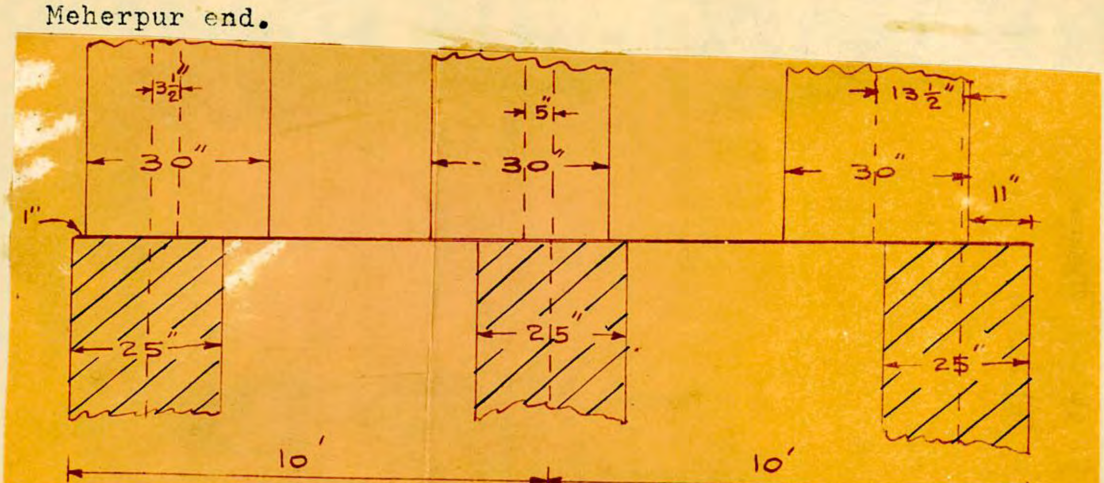
1) The reinforcements placed at the bottom of the well-cap

could not develop bond with the concrete; the cap was, as if, made of mass concrete without reinforcing steel. As a result these reinforcements could not play any part against failure of the cap in bending.

2) From the above observations, it may be said that the failure started from the well cap ; the longitudinal splitting shows that the well cap failed due to bending caused by the eccentricity in the position of the columns.

3) From an examination of the position of well No. 2 on the Meherpur end it is evident that the downstream end of the downstream column is 1 inch from the end of the well cap and the upstream end of the upstream column is 11 inches away from the end of the well cap. So the eccentricity in the position of the middle column is 5".

On the Kushtia side the well and well cap that failed have broken and the columns have been displaced; it may be said that the eccentricity of the middle column with respect to the centreline of the middle wall was 5 inches as in the Meherpur end.



4) This eccentricity in the position of the middle column produced a stress of 15 tons/sft. in the brickwork of the midsteining wall and put the well-cap in bending.

Load = 176 kps including live load

e = 5 inches

M = 176,000 x 5 lb-in.

$$\text{Total comp. stress} = \frac{P}{A} + \frac{Mc}{I} = \frac{176,000}{25 \times 70} + \frac{176,000 \times 5 \times 12.5}{70 \times 25^3/12}$$

$$= 221 \text{ psi} = 14.7 \text{ T/sft.}$$

The stress in the brickwork was possibly much higher due to stress concentration just below the column. The brickwork could not keep this stress for a long time and crushed into pieces.

5) The failure of the middle column resulted in redistribution of the total load on the two columns for which these columns failed. The suspended span was thrown on the river towards the downstream side which indicates clearly that the downstream column failed before the failure of the upstream column. The fallen Kushtia side half of the bridge is also deflected downstream which can verify the earlier failure of the downstream column.

C H A P T E R - III

FAILURE OF NAYARHAT BRIDGE

3.1 Location and description

3.2 Foundation failure

3.3 Structural failure

CHAPTER - III

FAILURE OF NAYARHAT BRIDGE ACROSS RIVER BANGSHI

3.1 Location and description:

The bridge is at Nayarhat on the Dacca-Aricha Road across the River Bangshi.

The bridge could not be completed because of the failure of the bridge in the foundation during construction. There was structural failure in the superstructure .

It was a two-lane balanced cantiliver bridge. The deck slab was supported on two main girders and a number of cross beams. The piers were on well-foundations. The abutment rested on R.C. piles and superstructure was supported on rollers upon the abutments.(Fig. 3.3).

No information on soil exploration was available.

3.2 Foundation failure:

The construction of the foundation structures was started late in the working season. The wells were sunk up to a certain depth when suddenly the flood came. In that year the flood was of a devastating nature. It completely submerged the portion under construction. The current in the river was so high that the well tilted.

In the next working season when the flood completely subsided, the remaining portion of the well could not be traced. Divers were brought to the site who, after repeated attempts, located the portion of the well and reported that the whole of the well had gone down to a certain extent and there was no damage to the well.

A hollow reinforced concrete box was made, floated to the site and was placed on the well below. Water was pumped out from the inside of the box and then the hollow space was filled with concrete.

The R.C. hollow box was of 3" thickness only. When it was lowered down on the existing well, water leaked through the wall of the box.

However, after a repeated trials the box was set in position and filled with concrete and the columns were made to bear on this.

At this stage it was proposed to perform the load test before placing the superstructure on it. Load test was done by the usual method of dumping brick and blocks of stone on it. As soon as the test load exceeded the design load, the portion of structure under test went down into the river.

Henceforth no attempt was made to do anything for the foundation of the bridge. Moreover the huge quantity of bricks and other foreign materials on river-bed cause hindrance to anything to be done there.

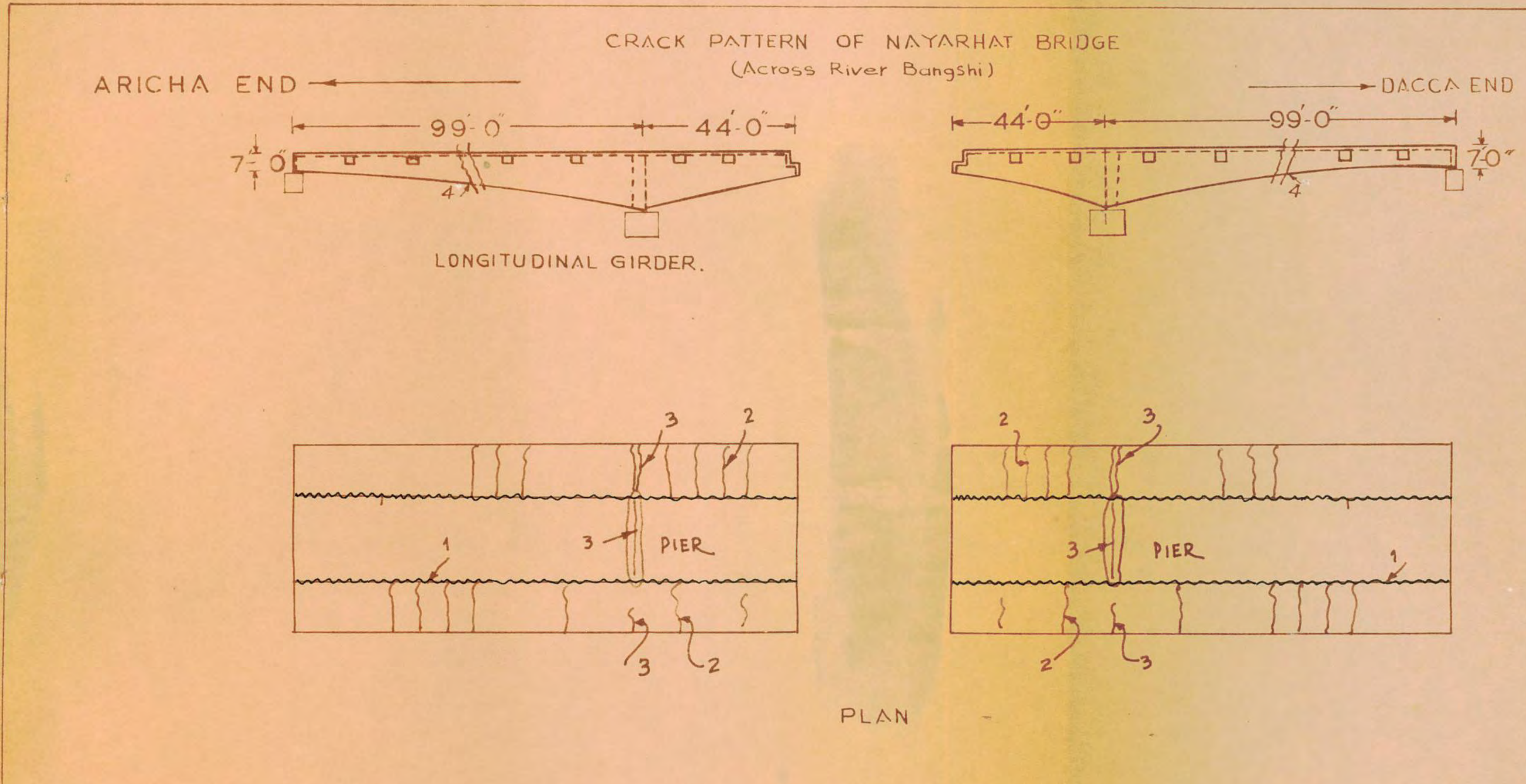

CRACK PATTERN OF NAYARHAT BRIDGE
(Across River Bangshi)
ARICHA END
DACCA END
99'-0"
44'-0"
7'-0"
4
LONGITUDINAL GIRDER.
1
2
3
PIER
PLAN

FIG. 4·3

3.3. <u>Structural failure</u> :

The two existing end portions of the Bangshi Bridge have failed structurally. The failure is seen in the deck-slab and in the girders.

The crack pattern in the deck slab are of two types :

a) Longitudinal cracks.

b) Transverse cracks.

Longitudinal cracks (Fig. 3.3) are type I cracks. Absence of top reinforcements in the cantilevered side-walk was the cause of the failure. The top rods were displaced down during casting. This displacement of reinforcements occurred due to the displacement of steel chairs and movement of labours on reinforcements during casting of the slab.

The thickness of the cantilevered side-walk at the cracks is about 3". Downward displacement of reinforcements causes reduction in effective depth and lowers the moment resisting capacity of the section. The displacement was probably quite high to cause failure.

Transverse cracks are of two types as shown in Fig. 3.3.

i) Type - 2

ii) Type - 3

Type-2 cracks are in the cantilevered side-walk.

The cantilever is only $3''$ thick and 144 ft. long. Absence of steel in the top, because the reinforcements are displaced down, contributed to failure on account of shrinkage and stresses arising out of variations in temperature.

Type-3 cracks are in a transverse direction and are situated near the supports which is the zone of maximum negative stresses. The development of these cracks may be due to the following reasons :

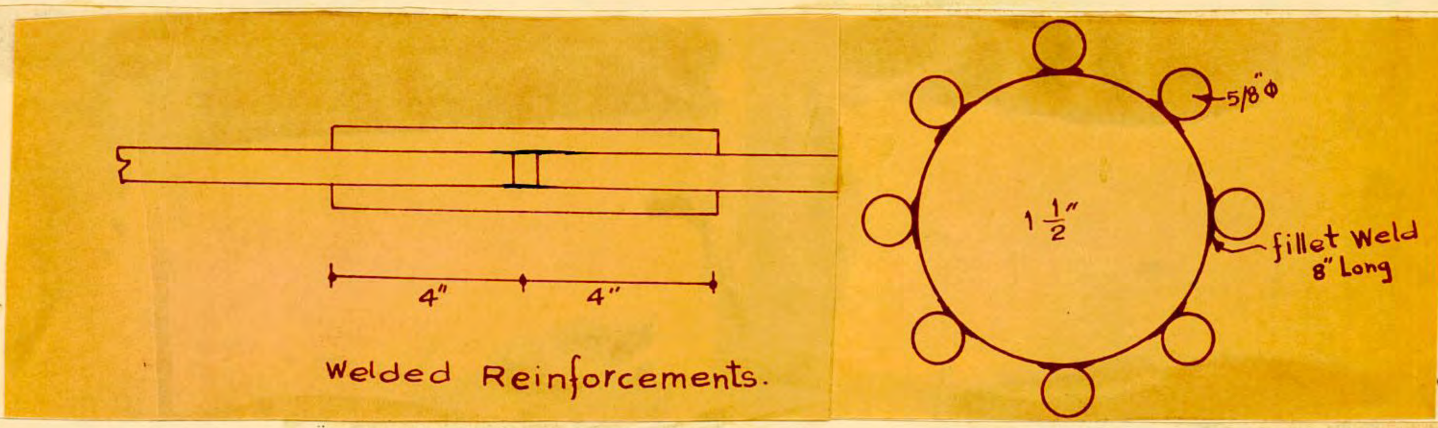
a) The cracks are assumed to have developed in the thin wearing course which is not monolithic with reinforced concrete deck slab. These are probably limited to the wearing surface and not extended upto the deck slab. So the cracks are only surface cracks, which do not indicate failure.

b) The flanges of girder ribs are monolithic with them so as to act like a T-beam. Heavy negative moments in the ribs cause tensile stresses in the entire flange. The provision of negative steel is almost concentrated in the girder rib and there is almost no negative steel in the slab. This has resulted in cracks in region where no negative steel has been provided. A certain percentage of negative steel should have been given in the flanges.

<u>Failure in longitudinal girders</u>: Cracks type 4 in (Fig. 3.3).

The deck system consists of a $6''$ two-way slab supported on two longitudinal girders and cross beams. No cracks have been noted in the transverse floor beams. Quite a large number of cracks are in longitudinal stringers. Most of these cracks in the girder are inclined to vertical and are extended in zones of compressive stresses.

The bars used as reinforcements were 1½" dia. Eight ⅝" dia. rods were welded with these bars for jointing(12).



The exact location of welded joints could not be obtained. The effective diameter of bars at welded joint failure to 1½" + 2 x ⅝" = 2¾". All the reinforcements were welded at one section. This resulted in reduction of concrete round the reinforcements thereby weakening the portion of the structure to bond stresses. Horizontal cracks along the longitudinal reinforcement indicate bond failure.

CHAPTER - IV

FAILURE OF MIRPUR BRIDGE

4.1 Location and particulars

4.2 Case-history of bridge failure

4.3 Analysis of causes of failure

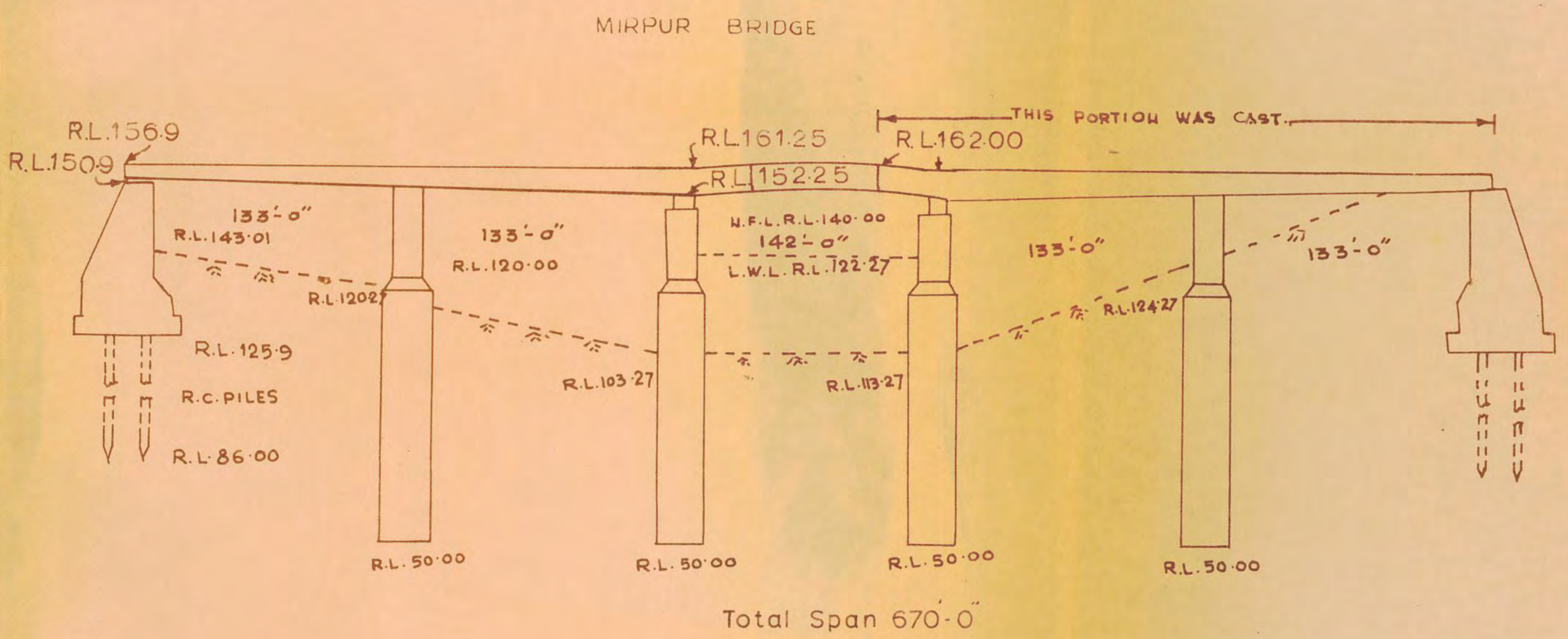

MIRPUR BRIDGE
THIS PORTION WAS CAST.
R.L.156.9
R.L.150.9
R.L.161.25
R.L.162.00
R.L.152.25
133'-0"
R.L.143.01
133'-0"
R.L.120.00
H.F.L. R.L.140.00
142'-0"
L.W.L. R.L.122.27
133'-0"
133'-0"
R.L.120.27
R.L.124.27
R.L.125.9
R.C.PILES
R.L.103.27
R.L.113.27
R.L.86.00
R.L.50.00
R.L.50.00
R.L.50.00
R.L.50.00
Total Span 670'-0"

FIG. NO 4.1

CHAPTER - IV

FAILURE OF MIRPUR BRIDGE

4.1 Location and particulars :

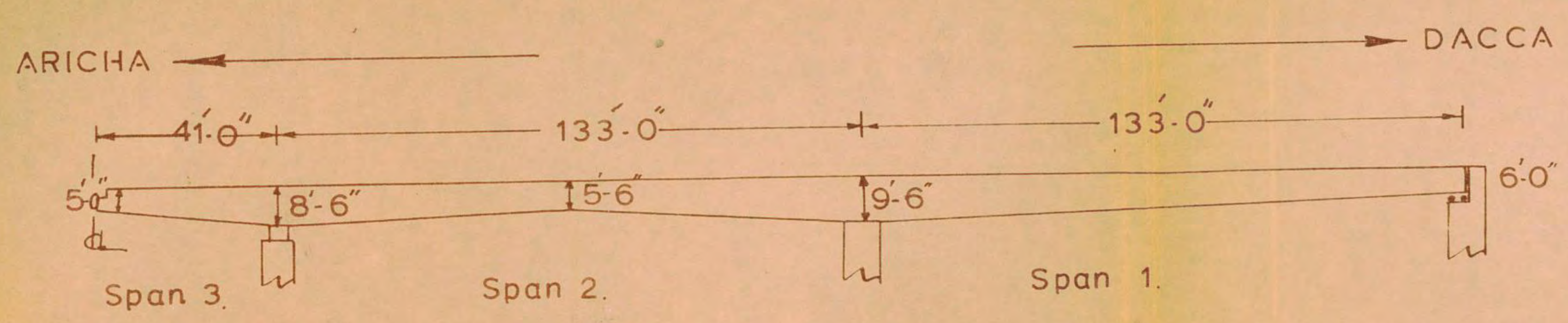
The bridge is across the river Turag at the eighth mile from Dacca on Dacca-Aricha Road.

It is a balanced cantilever type bridge having the suspended span at the middle(Fig.4.1). The abutments rest on R.C. piles of 12" x 12" x 40' size and the piers on well foundation of depth 50'-0" (Fig. 4.1).

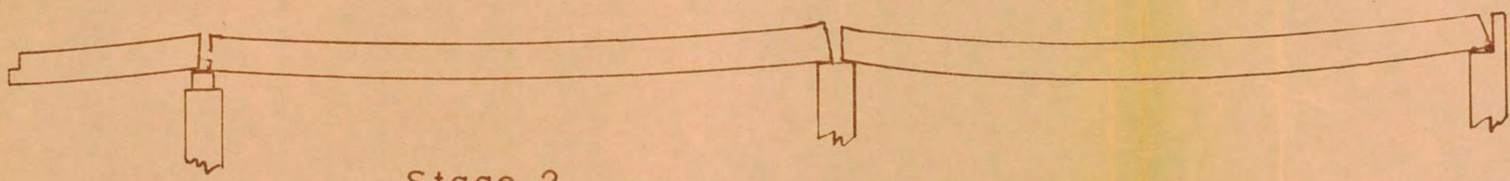
4.2 Case-history of failure :

Foundation of the bridge was complete, abutments and piers were erected and were ready to receive the superstructure. The casting of superstructure was planned in two stages - the casting of the girder ribs at the first stage and that of the slab at the second stage. About the time when the casting of the girder ribs were completed, it was noticed that the form-work for the girder deflected laterally due to incorrect and insufficient propping and the work had to be stopped. Subsequently, the salballah props were worn out and damaged causing the spans to sag. It was apprehended that the girder would fall and block the waterway. The girders were, therefore, dismantled slowly by cutting concrete to small fragments.

Stages of failure of Mirpur Bridge Girder

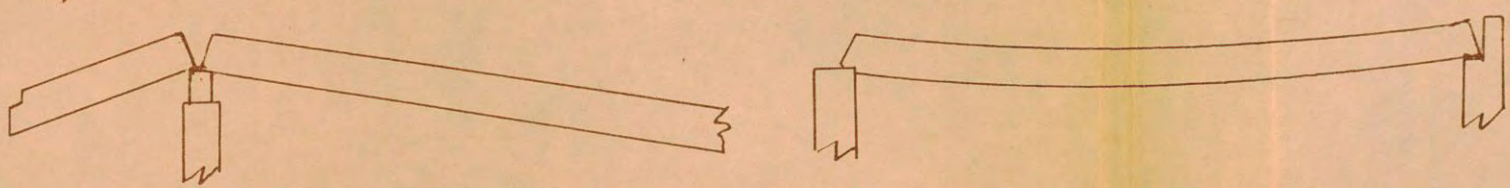


Stage 1-as originally constructed : only girder ribs cast.



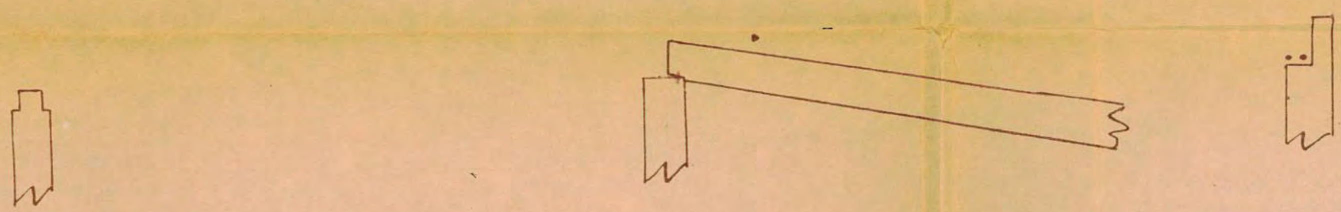
Stage 2.

Supporting salbollah props yielded.



Stage 3.

Girder failed due to Improper bond between Conc. & Steel. One end of girder in span 2 fell down into the river still having support at other end. The girder ribs were removed by breaking Conc. in Parts.



Stage 4.

Girder in span 1 fell down into river bed and was removed by breaking conc. in Parts.

FIG. NO. 4.2

It was thought of redesigning the superstructure of the bridge, assuming that the substructure of the bridge was sound, but later it was observed that the foundation could not support the dead load of the structure.

In the next winter when the water level of the river was below the level of the well-cap and some portion of the well itself was exposed, it was found that there were vertical cracks in the second pier and the cracks were more wide at the bottom than at the top of the pier (Fig.4.2). The crack extended upto the well cap ; but the well itself was free from such cracks.

4.3. Analysis of causes of failure :

1) The salballah props yielded when ribs were cast and the ribs sagged between the supports. Sufficiently strong props should have been used for centering.

2) Bond between concrete and steel reinforcements could not develop its full strength. The reinforcements in the girders were closely spaced. Tamping by rods was not sufficient to take concrete to the remotest corner of the deep girders. If vibrator were used to vibrate the concrete mass thoroughly it was possible that concrete would have travelled to the required depth and to the nook and corners of the girder.

3) The vertical cracks in the pier were probably due to either unequal settlement of the pier or stress concentration on the pier.

Since the cracks did not go on to the wells and the well did not also show any sign of unequal settlement, the question of settlement of the pier is ruled out. Therefore, only reason for the crack must be due to stress concentration resulting from i) falling of the girder ribs on the pier with impact ; but it is not likely as the girder ribs were dismantled slowly; or ii) due to heavy load put on the pier during testing. The test load might have been put with eccentricity and without good bearing plate under the applied load, and the concrete in the pier was probably poor in bearing strength.

CHAPTER - V

FAILURE OF TONGI BRIDGE

CHAPTER - V

FAILURE OF TONGI BRIDGE

5.1 Location and description:

The bridge is situated at Tongi on Dacca-Tangail Road across the river Kohar.

This is a R.C. two-lane highway bridge. The main span is rigid frame (plate 5.1). The two-way deck slab rests on two main longitudinal girders and a number of suitably spaced cross-girders. The deck slab is continuous as cantilevered sidwalk on both sides. The piers rest on wells in the river. The pier has a multiple hollow section with rounded ends(Fig.A in Appendix).

Drawings show that vertical reinforcements with spirals from the piers have been extended up to the full depth of the girders(14). So the column was fixed with the girders, thus making the central span a rigid frame with cantilevers. At both the articulations the anchor spans were bolted with the cantilevers.

The bridge was opened to traffic in 1954.

5,2 Crack Pattern (Plates 5.2 A - 5.2 D)

Sometime in the month of December,1966, it was detected that the main longitudinal girders, the cross-girders and even the slabs and cantilevered sidewalk have developed cracks. These cracks, at the time of detection, were in the form of hair cracks. A continuous vigilance was maintained and telltales were provided with dates in order to examine whether the cracks would widen.

In two months' time the cracks have extended through the tales and widened upto a width of ⅛" in places.

These cracks are widely distributed and irregular, but more prominent and congested in the zones of negative bending moments, that is, in the floor beams and longitudinal girders upto a distance of nearly 8' on both sides of the piers of the main span. These cracks are more or less vertical, but sometimes inclined to vertical and even horizontal at some places.

There are noticeable cracks in the floor beams and main girders in the anchor span near the supports.

5.3 Possibilities for the failure:

The investigation consists in the examination of the possibilities and effects of the following:

A) Settlement of any pier.

b) The bolted connection at the articulations, that is, the fixity of the anchor spans with the cantilevers.

c) The central span acted as a rigid frame, because the girders were fixed with the columns.

d) There has been overloading on the bridge ; this is due to rapid growth of industries in this area.

e) Temperature changes and shrinkage.

5.4 Analysis:

a) The design of the bridge was reviewed. The sections and reinforcements provided are sufficient for $H_{20} - S_{16} - 44$ loading. Reinforcements provided for diagonal tension are sufficient. Bond stresses at all sections are within working limits.

b) The possibilities of settlement of a pier was examined. Even though any pier of the main span might settle down one inch, the total stresses do not exceed the working limit of stress. If it settles two inches, the total moment becomes only 60% of the ultimate moment of resistance of that section. Moreover, there is no proof about the settlement of any pier.

c) For the main longitudinal girders being fixed with the columns, moments are induced at the column heads; but the column section and reinforcements are more than sufficient for the induced moment.

d) The bending moment induced in the columns and beams due to shrinkage and change in temperature is not appreciable. According to specificati ns, increased unit stresses are allowed when temperature and shrinkage effects are combined with other loadings ; so this effect is not considered for designs.

e) Over-loading may be one of the causes for failure of the bridge. The distribution of cracks may probably justify the effect of over-loading. Cracks in the central span at or near the mid-section extending from top towards the bottom signify possibility of negative moment at this section where positive moment should be the criterion. This is possible when the cantilevers are overloaded.

f) The strength of concrete was examined at a number of places of the structure by an impact type concrete hammer which gives the ultimate strength. The strength of concrete shows that concrete was not of inferior quality.

g) It cannot be verified without breaking the structure to pieces whether reinforcements were placed and cranked according to the drawings. So it is premature to say whether the failure was contributed also to lack of proper supervision or faulty construction, it needs further study.

C H A P T E R - VI

FAILURE OF OTHER R.C. BRIDGES

6.1 Bhangaghat Bridge

6.2 A bridge on Bhanga-Madaripur Road

6.3 Chokedar Bridge

6.4 Jamtala Bridge

6.5 Kamaldi Bridge

6.6 A khal Bridge

6.7 Sadhur khal Bridge

6.8 A bridge on Demra-Daudkandi Road

6.9 A bridge on Feni-Chittagong Road

6.10 A bridge in Jessore district

CHAPTER - VI

FAILURE OF OTHER R.C.BRIDGES

6.1. Bhangaghat Bridge :

This balanced cantilever bridge is situated at Bhangaghat in Faridpur on the Faridpur-Bhanga Road across the River Kumar.

The bridge was opened to traffic before the monsoons of 1964 set in ; but sometime in August it was observed that the clearance between the screenwall and the cross girder of the main spans widened to an extent of 4" towards the Faridpur end and 7" towards the Madaripur end. The wing walls were slightly deflected. As monsoons advanced the wing walls of both the abutments collapsed. When the wing walls fell down the retained earth escaped.

A record was maintained on the gaps between the screen wall and the cross-girder and a chart of observations from August to November was made. The chart shows that the gap increased in the rainy day and remained constant in the dry weather. For a period of four months, the gaps increased to an extent of $\frac{1}{2}$" on both the sides.

Chart of observation(15)

Date	Faridpur side	Madaripur side	Remarks
27/8/64	4"	7"	Dry weather
30/8/64	4½"	7¼"	Heavy shower
1/9/64	4¼"	7¼"	Light rain
2/9/64	4¼"	$7\frac{3}{8}$"	Heavy shower
5/9/64	$4\frac{3}{8}$"	7½"	Heavy shower
19/9/64	4½"	7½"	Dry weather
End of No.'64	Same	Same	Dry weather

Causes of failure:

1) The wing walls deflected and ultimately fell down (Plate 6.1B) due to the scouring at the foundation of the wing walls. These were resting on open foundations.

2) The abutments were resting on piles. The piles under the heel slab of the abutments were most probably not sufficient. The pressure on the heel exceeded the bearing power of the soil and the heel settled resulting in the backward inclination of the abutments thereby increasing the gap with the cross-girder.

Remedy:

In order to release the load on the heel, the abutments were converted into piers by adding two 20' shore spans on the two sides (plate 6.1.A). The overburden on the heel was removed and the pier did not tilt any further. Three subsequent monsoons have passed without any deterioration.

6.2. A bridge on Bhanga-Madaripur Road :

(**Failure** of abutment)

The bridge is across a small river like a khal overlying a vast low-lying catchment area. During winter, the khal gets almost dried up ; but in the rainy season, the flow increases abundantly. In August of 1964, it rained heavily and the discharge increased tremendously. As the opening of the bridge remained unchanged, the velocity of flowing water increased. The soil in the bed of the river was sandy clay and huge scouring took place at the bases of piers and abutments. The piles below the abutment were exposed to an extent of 8'. The piles were no longer able to carry the load and gave way causing the collapse of the abutment.(plate 6.2).

6.3 Chokedar Bridge :

(Failure of Abutment)

This bridge is on the 6th mile of Bhanga-Madaripur Road in the district of Faridpur. The flood in August,1964, washed away the earth from below and sides of the abutment of the bridge. The abutment gave way and the bridge collapsed towards the Bhanga end. The wing wall is seen standing in position (Plate 6.3).

6.4 Jamtala Bridge: **(Failure of wing walls)**

The bridge is on a khal only eight miles from Faridpur town on Madaripur-Faridpur Road in the district of Faridpur. During monsoons the velocity of the water increased **greatly.** The wing walls resting on **shallow** foundations suffered scouring at the base. The wing wall was about to collapse. The photograph shows how the wing walls, tied with wire ropes, were prevented from falling down. (Plate 6.4).

6.5 Kamaldi Bridge:

(Failure due to insufficiency of water-way)

The bridge is on the thirteenth mile of Bhanga-Madaripur Road. The flood of August,1964, completely washed way the two approaches of the bridge. There was no scouring at the foundation and the bridge was standing in position. The photograph (Plate 6.5) shows that the water-way increased by washing away of the bridge approaches and the bridge was saved from imminent failure.

6.6 A khal Bridge:

(Failure of abutment and wing walls)

This is a single span bridge over a khal in the district of Barisal. During the flood, the bed of the khal was scoured by the rushing water with great velocity. Gradually the earth below the abutments and wing walls were washed away and the bridge collapsed (plate 6.6.)

6.7 Sadhur Khal Bridge:

(Failure due to excessive scouring)

The bridge is on the 12th mile of Bhanga-Madaripur Road across a khal called Sadhur Khal. During the flood the piles under the abutments were exposed (Plate 6.7) due to excessive scouring of the river bed. When the flood subsided completely, earth was packed under the abutment and on the river bed.

6.8 A Bridge on Demra-Daudkandi Road:

This is a one-span bridge of forty feet span length with masonry abutments. The masonry abutment cracked at a depth of a few feet below the girder(Plate 6.8). The failure was possibly due to that expansion bearings did not function. Mortar and brick bats made the girders fixed with the abutments. The girders expanded and pushed the abutment out thereby causing the horizontal cracks.

6.9 A Bridge on Feni-Chittagong Road:

(Settlement of a pier)

The bridge is on the fourth mile from Feni on Feni-Chittagong Road on a tidal khal in the district of Noakhali. The bridge has three simple spans. The abutments and piers are resting on piles. The pier towards Chittagong tilted and this was reflected in the girders and deck slab. The gap between the deck slabs increased. The settlement of the pier was due to insufficient number of piles under the piers. The foundation of the pier was dug open when the piles were exposed. Only half the number of piles that are shown in the drawing were actually provided.

6.10 A bridge in the Jessore district:

(Failure of abutment)

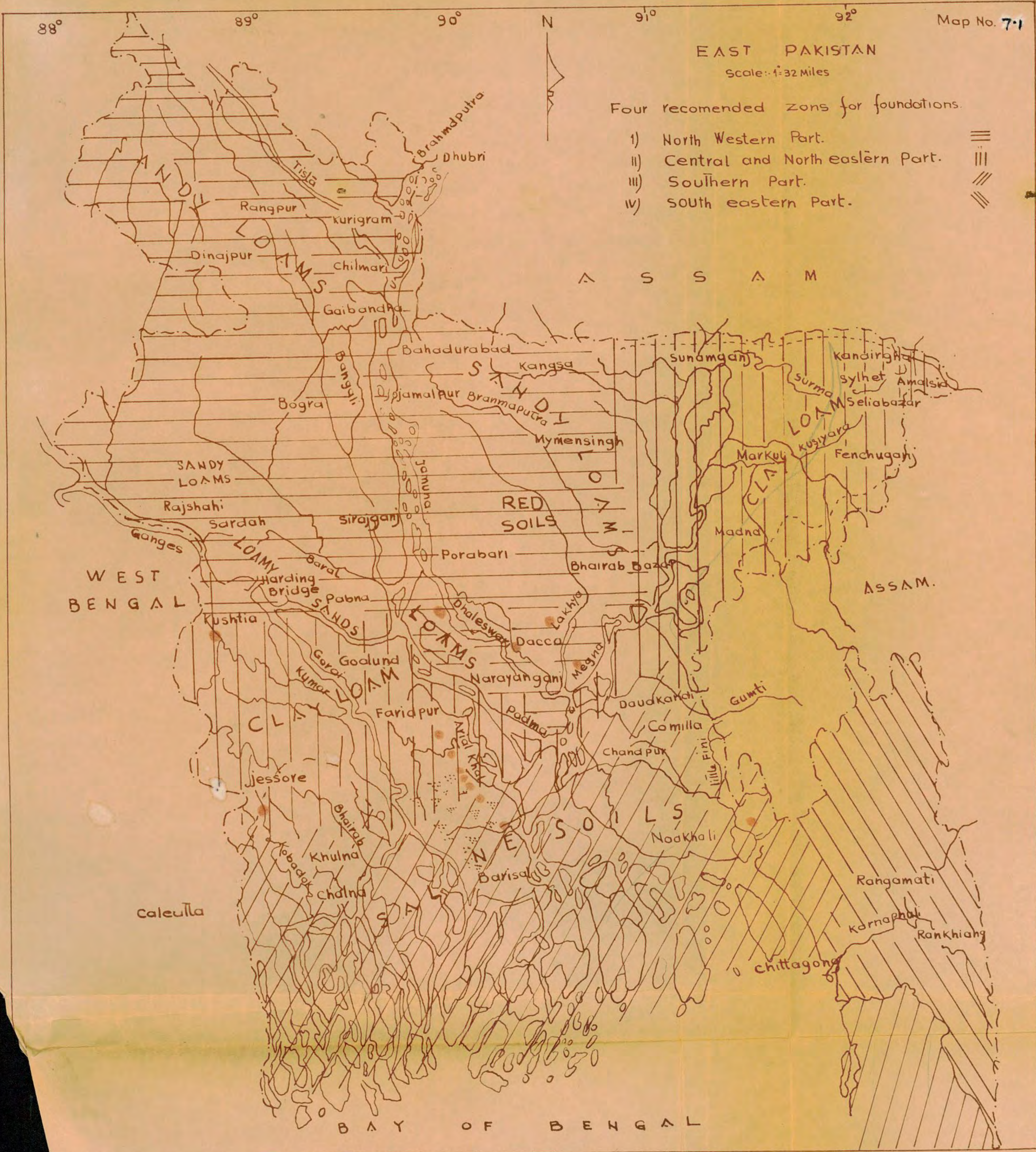
The abutment of this bridge inclined backward carrying the suspended span with it. This was detected by the increase in the gap between the adjacent railings at the articulation (Plate 6.10). The failure was probably due to insufficiency of piles under the heel of the abutment. The other probability is the use of cohesive soil in the back-fill. When a cantilever abutment is designed according to Rankine's formula, the back-filling material should be cohesionless soil, such as sand. As the soil behind the abutment was possibly a cohesive one which did not develop the assumed horizontal thrust against the abutment and the heel slab settled due to the load of the back-fill resulting in backward inclination.

CHAPTER - VII

DISCUSSIONS

7.1 Foundation failure

7.2 Structural failure



Map No. 7·1
EAST PAKISTAN
Scale:- 1"= 32 Miles
Four recomended zones for foundations.
I) North Western Part.
II) Central and North eastern Part.
III) Southern Part.
IV) South eastern Part.
88°
89°
90°
91°
92°
N
ASSAM
ASSAM.
WEST BENGAL
BAY OF BENGAL
SANDY LOAMS
RED SOILS
LOAMY SANDS
LOAMS
CLAY LOAM
SALINE SOILS
Dhubri
Rangpur
Kurigram
Dinajpur
Chilmari
Gaibandha
Bahadurabad
Bogra
Jamalpur
Mymensingh
Rajshahi
Sardah
Sirajganj
Porabari
Harding Bridge
Pabna
Kushtia
Goalund
Faridpur
Dacca
Narayanganj
Jessore
Khulna
Chalna
Barisal
Calcutta
Sunamganj
Sylhet
Kanairghat
Amalsid
Seliabazar
Markuli
Fenchuganj
Madna
Bhairab Bazar
Daudkandi
Comilla
Chandpur
Noakhali
Rangamati
Chittagong
Rankhiang
Tista
Brahmdputra
Bangali
Branmaputra
Jamuna
Kangsa
Ganges
Baral
Dhaleswari
Lakhya
Megna
Gorai
Kumar
Padma
Bhairab
Surma
Kusiyara
Gumti
Karnaphali

CHAPTER - VII

D I S C U S S I O N.

7.1 Foundation failure :

The Province of East Pakistan has a number of physiographic divisions (Chapter - I). In the Piedmont region, the soil is composed of alluvial deposit ; in the Barind area, there is lateritic highland. The Central Valley Flat is a deposit of fine silt; the Delta of Sundarbans has a soil of loosely packed alluvium; the south-eastern part of the province is composed of low-lying hills.

A particular type of bridge and its foundation should be decided from the properties of soil on which the bridge is to rest. Soils having more or less similar geological characteristics, particularly as to their mechanical behaviour, may be grouped into one zone, and according to this classification, the province may be divided into four zones (Map 7.1).

Zone I - The north-western part :

This zone stretches down below the northern boundary of East Pakistan to Padma on the south and also includes the districts of Mymensingh and Dacca. This zone covers the whole of the physiographic divisions like Northern Piedmont, Barind, Brahmaputra Valley Flat and the western portion of Surma Valley.

The soil is mostly alluvial deposit of sand and clay with exceptions of elevated tracts of lateritic soil in the districts of Dinajpur, Rajshahi and Bogra(8). Excepting Brahmaputra Valley, this zone is composed of compacted soil. In absence of any record about the bearing capacity of this soil, reference may be made to Dunham's "Foundations of Structures" where this type of soil is found to have a bearing capacity of 1-3 tons/sft. The Brahmaputra

Valley is liable to be flooded almost every year causing deposition of silt ; but this is limited within a narrow belt on both sides of the river.

The Nayarhat Bridge, the Mirpur Bridge and the Tongi Bridge (all major bridges that failed) are in this zone. As discussed earlier in Ch. III, IV and V, none of these failures is due to the failure of foundation media. Foundations with piles in the northern districts have given satisfactory results.

Simple spans and cantilever bridges may be safely decided upon in this zone and foundation will not be a major problem.

127

Zone II - The central and north-eastern part.

The central portion of the province, starting from below the Ganges and extending half-way to the Bay of Bengal, and the north-eastern portion of East Pakistan form this zone, It covers the districts of Kushtia, Jessore, Comilla, Sylhet and upper portions of the districts of Khulna and Faridpur. The district of Sylhet is bounded by hillocks of the Tripura State in the east which are composed of limestones and sandstones of past origin, but the presence of vast low-lying areas called Haors with characteristics of alluvial soil justifies the inclusion of this district in zone II. The upper portion of the Sundarban Delta, Meghna Flat and the eastern portion of Surma Valley form this zone.

The soil is mostly alluvial with exceptions in Meghna Flat where there is active deposition of silt carried by the river every year. Innumerable rivers and creeks discharge ultimately into the Bay of Bengal through this zone. The soil is not as consolidated as in zone I(8).

Failure of a major bridge over the river Mathabhanga and three other minor bridges have occurred in this zone. The failure of the major bridge is structural and that of the minor bridges is due to foundation failure. One of these minor bridges failed due to insufficiency of piles under the heel of the abutment, as a result of which, the abutment tilted backwards, The reason perhaps is the over-estimation of the frictional resistance of the soil. The other probability is that the back-filling material being cohesive soil did not develop the assumed horizontal thrust against the abutment. According to Rankine's formula, the back-fill is assumed to be cohesionless. If cohesive soil is used, the horizontal thrust exerted by the back-fill may be absent and the load of soil on the heel may press it down resulting in backward inclination of the abutment. The other two bridges failed due to scouring at the foundation. Provision of sufficient waterway could possibly avoid these failures.

Foundation for simple spans and cantilever bridges will not be an acute problem in this zone either, as slight settlement will not harm these structures. However, it may not be advisable to construct continuous and rigid frame bridges

On this type of soil unless results of proper soil investigation support it. Deep foundations are generally necessary and Reinforced Concrete bridges should have well foundations in the river bed. In case of pile foundations, when scouring takes place at the piles under piers in the river bed, the group action of piles is lost. If the piles are exposed to such an extent that these act as slender columns, failure due to bending or buckling may take place. In case of wells, the chance of failure by buckling is less even though the wells are exposed to an extent when piles are transformed to slender columns.

The soil of Faridpur is possibly the worst within this zone. Lighter bridges of steel and prestressed concrete would perhaps be advisable for this area.

Zone - III - The southern part.

The lower portions of the deltas of the Ganges and the Meghna form this zone. This includes the physiographic divisions like the lower Delta of Sundarbans, the Eastern Piedmont and the coastal plains. This zone covers the lower portions of the districts of Khulna and Faridpur, and the whole of the districts of Barisal and Noakhali and a narrow belt adjoining the Bay of Bengal in the district of Chittagong. The coast-line is very irregular because of the large number of rivers discharging into the Bay of Bengal through this area and forming innumerable deltas of different sizes.

The soil is loose alluvium and formation of new land is very active in this zone. Naturally, the soil has low bearing capacity. In certain portions of the districts of Faridpur and Barisal, the soil is of organic origin having characteristic colour and smell. Its behaviour under load is very uncertain and has not been investigated uptil now. Flood is of yearly occurrence and water-logging is a feature of this zone particularly in the marshy areas of Faridpur.

Failure of seven minor bridges in this zone has been studied and found to have been caused by scouring of loose soil from the river bed. Scouring appears to be a very common cause of bridge failure in this zone, because of the recently deposited loose sandy material in the river bed. Scouring may be appreciably checked by ensuring adequate water-way and carrying the foundation structures deeper. For this adequate study of the characteristics of the river and the catchment area should be made before the span length of the bridge is decided upon in order to ensure sufficiently low velocity in the river. The end supporting structure is also to be designed to function both as an abutment and a pier so that extra shore spans can be added in the future if necessary.

This zone is the worst so far as the problem of foundation is concerned. Continuous and rigid frame bridges are not to be selected unless proper protection for such structures is ensured, based on thorough investigation of the different aspects of the design of such structures. Foundation settlement may, in such cases, induce stresses in the structure resulting in failure. Massive structures are to be avoided as far as possible. If major bridges of long spans are must, bowstring girder bridges with deep foundations like pneumatic cassions may be considered. Before selecting a heavy structure, the kind and condition of the soil at the site must be thoroughly investigated so that the bearing capacity and anticipated settlement is known for the foundation medium with sufficient accuracy.

In the southern portion of this zone where consolidation of the soil is in progress, very light bridges like wooden bridges, if properly maintained, would be the best.

Zone IV - The south-eastern part.

The south-eastern hilly part including a portion of the districts of Chittagong and the whole of Chittagong Hill Tracts form this zone. A narrow belt adjoining the eastern boundary of the district of Sylhet also shows characteristics of soil similar to those of this zone. This region is covered with hillocks composed of sandstones and clay of past geologic origin. This zone is high in elevation and is free from flooding.

The bearing capacity of soil in this zone is the highest in the province and is upto 10 tons/sft (10). Any heavy structure may be constructed in this zone without apprehending settlement of any consequence. From economic point of view, advantage of the construction of continuous or rigid frame bridge should be taken here.

7.2 Structural failure.

Structural failure of four major bridges has been discussed in this thesis. These are the Mathabhanga River Bridge, the Nayarhat bridge, the Mirpur bridge and the Tongi bridge.

The design procedure and calculations were found alright ; the materials used were of good quality and showed strength above that assumed in the design. The reinforcing steel in the well-cap was placed without concrete covering ; the amount of reinforcement was inadequate for carrying the load from the superstructure. The failure was, therefore, due to the improper design of the well-cap and bad supervision.

The failure of Nayarhat bridge was due to incorrect placement of reinforcements and lack of proper bond between welded reinforcements and concrete. All the reinforcements were welded at one place which reduced the space between reinforcements for concrete to enter and develop bond.

The Mirpur bridge collapsed also probably due to lack of proper bond between steel and concrete. Concrete could not properly travel down the deep girders and ensure bond. This may be attributed to lack of good construction technique. Use of vibrator, high strength concrete, construction of deep girders in stages could have saved the catastrophe.

The Tongi bridge has no defect in design (as reviewed in the Appendix) and there is no evidence of settlement. The foundation has suffered no scouring and the quality of concrete is good as found by impact hammer tests. The failure may, therefore, be attributed to some other factor. Cracking at cantilever near the supports and propagation of cracks from the top towards the bottom near the midspan indicate that the failure of the bridge is probably due to overloading. The limited scope of this thesis did not permit to say whether the failure is also due to faults in the construction.

CHAPTER - VIII

CONCLUSIONS

CONCLUSION

The main conclusion that can be drawn from the studies and analyses presented in this thesis may be summarised as follows:

1. Collapse of foundations due to scouring or settlement is mainly responsible for the failure of bridges in East Pakistan. Structural failure, particularly due to faulty design is rare ; although strary cases of structural failure arising out of the use of inferior quality of materials, employment of unskilled labour and improper supervision are not altogether absent.

2. Adequate soil investigation in the field and in the laboratory must be made before the site of a bridge is selected and its design prepared.

3. Based on soil characteristics the province may be broadly divided into four zones (as presented in the Discussion). R.C. piles and open masonry wells may provide suitable and economical types of foundations for the bridges in zone - I (the north-western part) and in zone - II (the central and north-eastern part) respectively. Relatively lighter structure would be suitable for zone - III (the southern part) ; but for major structure, if it is required to be built, deep foundations like pneumatic cassion may be decided upon. All types of foundation may be suitable for the structures in zone - IV (the south-eastern hilly part).

4. Information regarding the maximum flood discharge and the nature and extent of catchment area must be obtained in order to decide upon the span of the bridge for providing sufficient waterway. If this information is not sufficiently available, the bridge should not be completed with wing walls. The wing walls may be constructed later when sufficiency of waterway is ensured. If waterway provided is not sufficient, extra shore spans may be added. The end support should be designed so that this can function as abutment as well as pier as and when required.

5. Supervision is to be strict in all stages of construction. Quality of materials should be controlled, proper compaction and consolidation of concrete must be ensured. Skilled labours are to be employed for correct placement of reinforcing steel according to the drawings.

APPENDIX

REVIEW OF THE DESIGN OF THE TONGI BRIDGE

1. Notations

2. Review of the design

N O T A T I O N S

a	=	depth of Whitney stress block
A_s	=	Area of reinforcements
A_g	=	Gross area (in^2)
α, β	=	Moment coefficients depending on the shape of beams
A_v	=	Area of web reinforcements
b	=	breadth of slab or beam
C_A, C_B	=	Moment coefficients in the A and B directions
C_1, C_2	=	Moment coefficients
d	=	effective depth of slab or beam
d'd	=	difference in depths at end and middle sections of a parabolic beam
D	=	Diameter of reinforcements
DL	=	Dead load
E	=	Width over which wheel load is distributed
E_c	=	Young's modulus of Concrete
E_s	=	Young's modulus of steel
e	=	eccentricity in the position of load
f_a, F_a	=	Stresses due to axial load
f_b, F_b	=	Stresses due to bending
f_s	=	allowable stress in steel
f_c	=	allowable stress in concrete
f_y	=	ultimate strength of steel
f_c'	=	crushing strength of concrete (28 day strength)
FEM	=	Fixed End Moments
H_A, H_D	=	Horizontal forces at hinges

H_r, H_f	=	Horizontal reaction due to rise and fall in temp.
I	=	Impact factor
I_{xx}	=	Moment of Inertia about x-x
LL	=	Live load
M, BM	=	Bending moment
M_{FBC}	=	Fixed End Moment at B of the member BC
M_u	=	Ultimate moment of resistance
Σo	=	Perimeter of rods
p	=	percentage of reinforcements
r	=	carry-over factor
R_1, R_2	=	Reactions
s	=	stiffness of a member
S	=	Span between girders
SF	=	Shear force
V	=	Total shear at a section
V'	=	Shear carried by web reinforcements
Z	=	Section modulus.

Design of the two-way slab(continuous on all directions)

Clear span = 15'-8" (between longitudinal girders)

" " = 15'-8" (between cross-girders) (Fig. A.)

Slab thickness provided = 6"

Dead load of slab = 75 lb/ft.

Moment calculations:

a) Dead load moment ;

i) Positive moments ; $C_A = C_B = 0.018$

$$BM = 0.018 \times 75 \times (15.67)^2 = 332 \text{ ft-lb.}$$

ii) Negative moments ; $C_A = C_B = 0.045$

$$BM = 0.045 \times 75 \times (15.67)^2 = 830 \text{ ft-lb.}$$

b) Live-load moment;

Impact, $I = \frac{50}{L+200} = \frac{50}{15.67+200} = 0.3$

For continuous spans over 7' - 0"

$E = 0.4\ S + 3.75 = 0.4 \times 15.67 + 3.75 = 10.05$

Moment,(both positive and negative)

$= \frac{1}{2} \times 0.2 \frac{P}{E} \times S = \frac{1}{2} \times 0.2 \times \frac{16,000}{10.05} \times 15.67 = 2,500$ ft-lb.

Including impact, $BM = 1.3 \times 2,500 = 3,400$ ft-lb.

Reinforcements :

Total moment (negative) = 3,400 + 830 = 4,230 ft-lb.

$$d = \sqrt{\frac{M}{R \times b}} = \sqrt{\frac{4230 \times 12}{236 \times 12}} = 4.25''$$

$$A_s = \frac{4,230 \times 12}{20,000 \times 0.87 \times 4.25} = 0.68 \text{ in.}^2$$

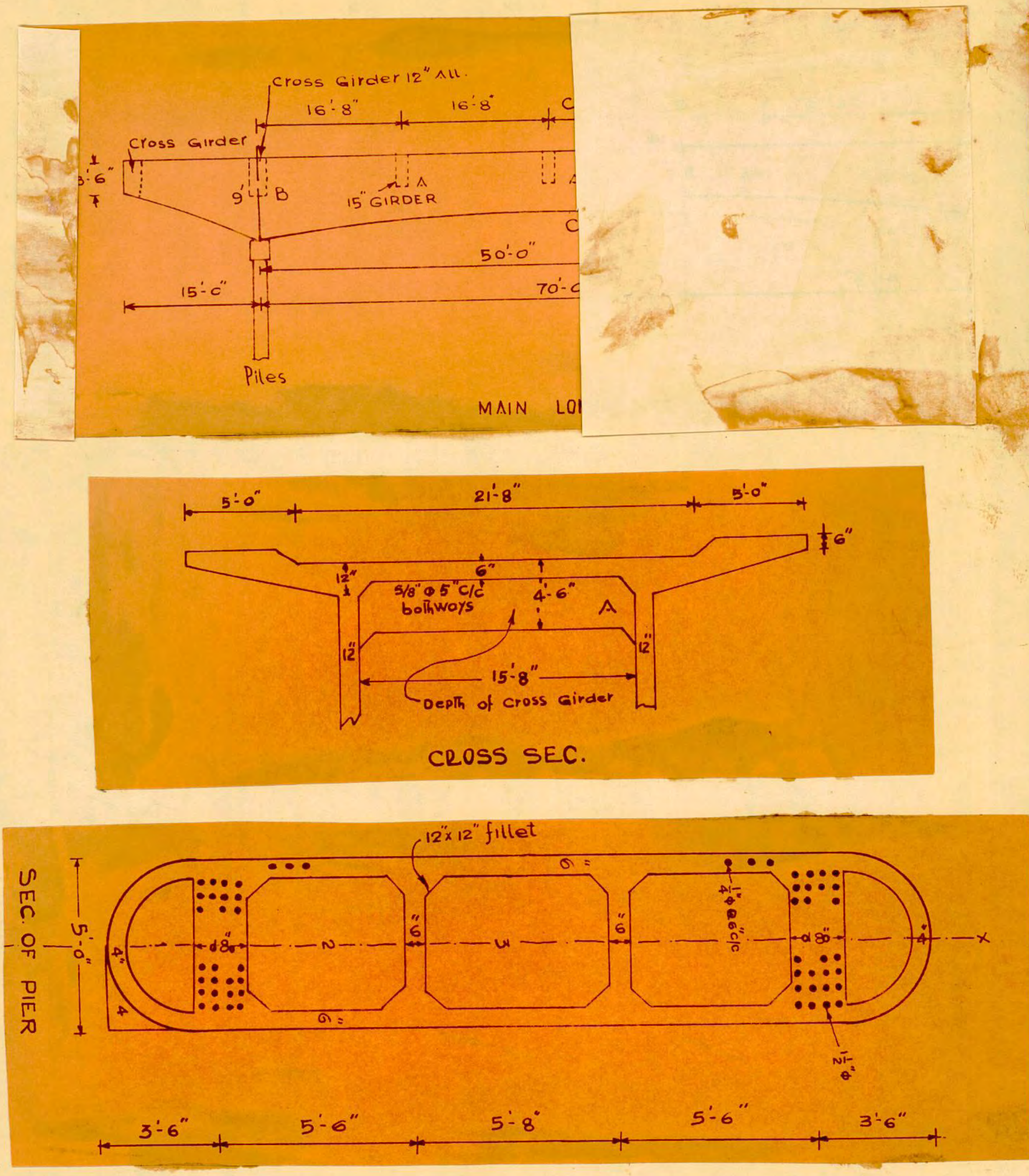

Cross Girder 12" All.
16'-8"
16'-8"
Cross Girder
3'-6"
9'
B
A
15" GIRDER
50'-0"
15'-0"
Piles
5'-0"
21'-8"
5'-0"
6"
12"
6"
5/8" Φ 5" C/C bothways
4'-6"
A
12"
12"
15'-8"
Depth of Cross Girder
CROSS SEC.
12"x12" fillet
SEC. OF PIER
5'-0"
4"
8"
6"
6"
8"
4
1 1/4" Φ @ 6" c/c
1 1/2" Φ
3'-6"
5'-6"
5'-8"
5'-6"
3'-6"

Fig. A

5 bars are used, spacing = $\frac{12}{0.68/0.31}$ = $5.5''$ c/c

Spacing provided is $5''$ c/c bothways.

MOMENT OF INERTIA OF THE COLUMN

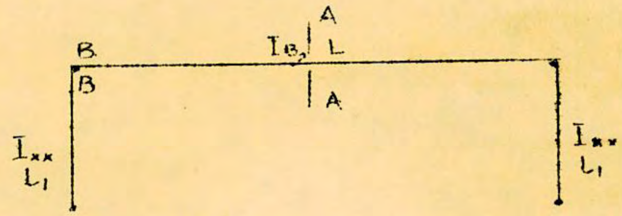
$$I_{xx} = \frac{23.67 \times 12 \times (5 \times 12)^3}{12} - 2 \times \tfrac{1}{2} \times \frac{3.14}{4}(26)^4$$

$$- 2 \times \frac{54 \times 48^3}{12} - 62 \times \frac{48^3}{12} - 4 \times \frac{1}{3} \times 30 \times 30 \times (\tfrac{3}{4} \times 30)^2$$

$$+ 12 \times \tfrac{1}{2} \times 12 \times 12 \times 20^2$$

$$= 26 \times 10^5 \text{ in.}^4$$

CARRY-OVER FACTOR AND DISTRIBUTION FACTORS



I_B = moment of inertia of the sections at the middle of both the beams (A - A)

$$= \frac{2 \times 18 \times 63^3}{12} = 7.5 \times 10^5 \text{ in.}^4$$

d = depth of section at A - A = $63''$

d'd = difference in depths of sections at B-B and A-A

= 120 - 63 = $57''$

$$d' = \frac{57}{63} = 0.9$$

Corresponding to these values (2),

Stiffness, $s = 11.8 \frac{I_B}{L}$ and $rs = 7.5 \frac{I_B}{L}$ *

whence $r = 0.65$

Relative stiffness of column $= \frac{3I_{xx}}{L_1} = 19{,}100$

Relative stiffness of beams combined

$$= 11.8 \frac{I_B}{L} = 11.8 \times \frac{750{,}000}{986} = 9{,}000$$

where L_I = height of column = 29 + 10/2 = 34 ft.

L = length of each beam = 82'-2" = 986"

Distribution factors

column	:	beams
$\frac{19{,}100}{28{,}100}$	:	$\frac{9{,}000}{28{,}100}$
2/3	:	1/3

Dead load reaction from anchor span at articulation

Calculation of dead weights : (Fig. A)

a) For one-foot strip of slab :

31'-8" x 6" x 1'-0" x 150 lbs. = 2.4 k/ft.

b) Rectangular portion of 15'-0" beams

weight of each beam/ft run

= 3'-6" x 15" x 1'-0" x 150 lb/ft. = 0.66 k/ft.

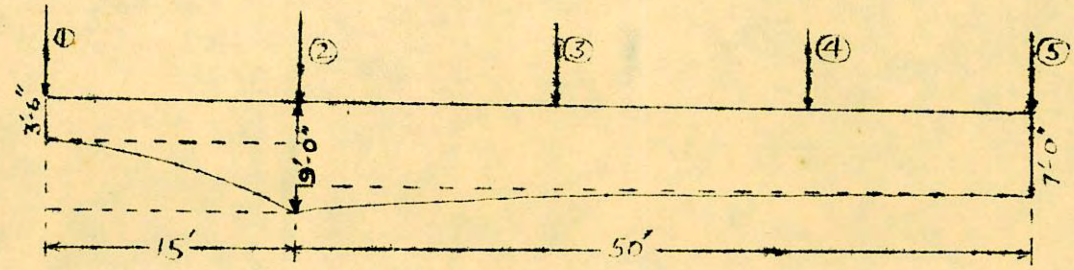
where breadth of each beam = 15"

* Fig. 15 - Page 139 - Cross and Morgan

c) Rectangular portion of 50'-0" beams

weight of each beam/ft run

= 7'-0" x 15" x 1'-0" x 150 lb/ft. = 1.37 k/ft.



d) Parabolic portion of 15'-0" beams :

Total weight for each beam

= 1/3 x 5'-6" x 15" x 15' x 150 lbs. = 5.2 k

e) Parabolic portion of 50'-0" beams :

Total weight for each beam

= 1/3 x 2'-0" x 50' x 15" x 150 lbs = 6.3 k

f) Cross-girder No.1

Total weight = 3'-6" x 12" x 15'-8" x 150 lbs = 8.32 k

g) Cross-girder No.2

Total weight = 5'-6" x 12" x 15'-8" x 150 lbs = 13.0 k

h) Cross-girder No.3 and 4

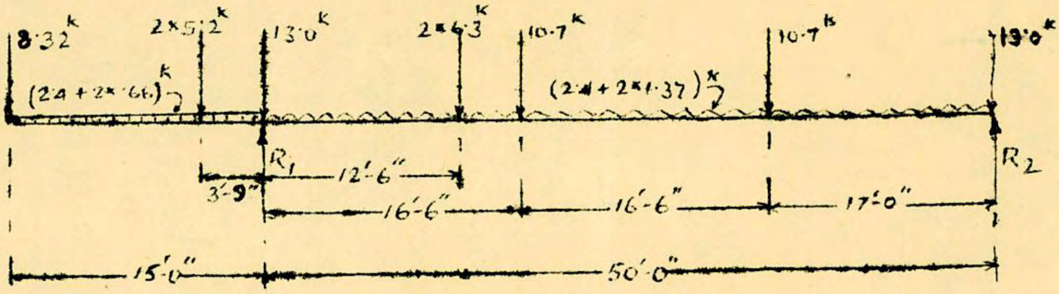
Total weight of each = 4'-6" x 12" x 15'- 8" x 150 lbs

= 10.7 k

i) Cross-girder No.5

Total weight = 5'-6" x 12" x 15'- 8" x 150 lbs

= 13.0 k



Taking moments about R_1,

$- 8.32 \times 15 - 2.4 \times 15 \times 15/2 - 2 \times 0.66 \times 15 \times 15/2 - 2 \times 5.2 \times 15/4$

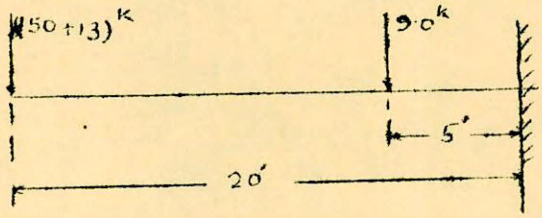
$- 2.4 \times 50 \times 50/2 - 2 \times 1.37 \times 50 \times 50/2 + 6.3 \times 2 \times 50/4 +$

$13 \times 50 + 10.7 \times 33 + 10.7 \times 16.5 - R_2 \times 50 = 0$

Or, $R_2 = 143.7$ k

Adding weight of railings, $R_2 = 150$ k

Dead load BM and SF at the fixed end of cantilever



Reaction from the anchor = 150 k

Reaction from the Cross-girder = 13 k

weight of the slab/ft run = 2.4 k/ft

weight of the rectangular portion of both the beam/ft run

= 2 x 7'-0" x 1'-0" x 18" x 150 lbs = 3.16 k/ft

Total weight of the parabolic portions of both the beams

= 3 x 20 x 1.5 x 150 x 1/3 x 2 lbs = 9.0 k

DL BM = 163 x 20 + 9 x 5 + 5.56 x 20 x 10 = 4,417 k-ft.

DL SF = 163 + 9 + 5.56 x 20 = 283.2 k

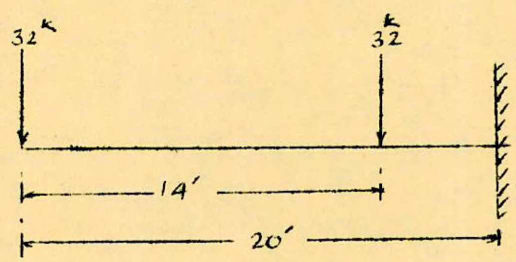
L.L. BM and SF when left cantilever loaded :

BM at fixed end of cantilever:

Two cases would be considered and the greater moment would be taken :

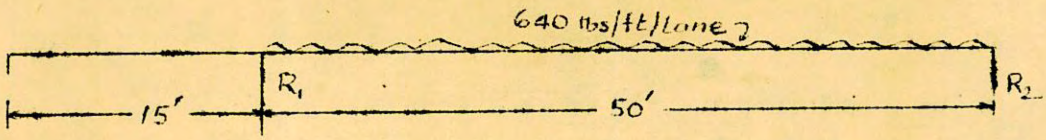
a) Truck loading on a span of 20'

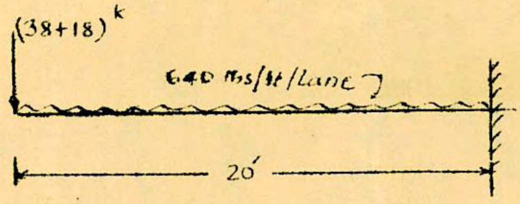
$$I = \frac{50}{L+200} = \frac{50}{20+200} = 0.227$$



BM = (32 x 20 + 32 x 6) x 2 x 1.227 = 2,050 k-ft

b) Loading on a span of 70'





$$I = \frac{50}{70+200} = 0.185$$

$$R_2 \times 50 = \frac{640 \times 50 \times 50}{2 \times 1{,}000} \times 2 \times 1.185$$

Or, $R_2 = 38$ k

$$BM = 38 \times 20 + 18 \times 2 \times 1.185 \times 20 + \frac{640 \times 20 \times 20}{1{,}000 \times 2} \times 2 \times 1.185$$

$= 1{,}919$ k-ft

Case (a) gives greater BM of 2,050 k-ft

LL SF = $(32+32) \times 2 \times 1.227 = 157.1$ k

So total SF at the fixed end of cantilever = $283.2 + 157.1$

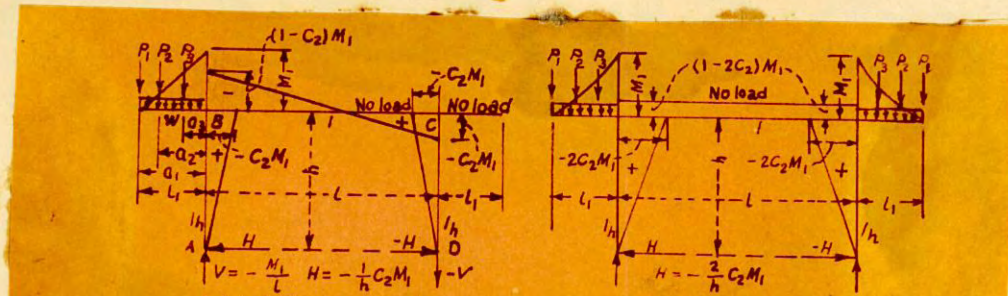
$= 440$ k

BM at column Head

In fig. B,

$$C_1 = \frac{\beta}{4\left[2\alpha + \beta + 2\left(\frac{h'}{h}\right)^3 \frac{I_h}{I_{hL}}\right]}$$

$C_2 = 6\,C_1$



(a) Left Cantilever Loaded (b) Both Cantilevers Loaded Symmetrically.

where α and β are constants depending on the shape of the horizontal beam(3).

$\alpha = 0.50$ *

$\beta = 0.68$ *

$$C_2 = 6\ C_1 = 6 \times \frac{.68}{4\left[2 \times .5 + .68 + 2\left(\frac{22}{34}\right)^3 \times \frac{2 \times 18 \times 6.3^3 \times 34}{12 \times 26 \times 10^5 \times 82.2}\right]}$$

$= 0.56$

Moment at column head = $- C_2\ M_1 = - 0.56 \times 2{,}050 = -1{,}150$ k-ft

Moment at the rigid end of the beams = $(1-C_2) \times M_1$

$= (1 - 0.56) \times 2{,}050$ = $-$ 900 k-ft

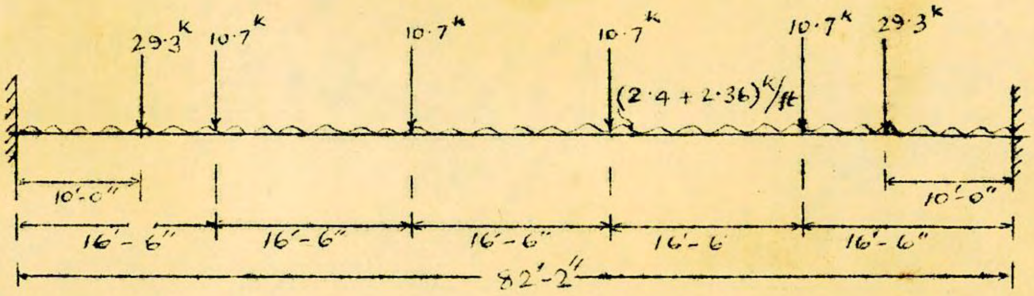
Bending moment at mid-span of beams:

BM = (1150 - 900)/2 = 125 k-ft (+)

* Table - 2, Page - 203, " Reinforced Concrete Bridges"

\- Taylor, Thomson and Smulski

FEM of the beams due to dead load :



Calculation of dead weights :

a) Weight of the rectangular portion of beams/ft run

= 2 x 5'-3" x 1'-0" x 1'-6" x 150 lbs = 2.36 k/ft

b) Total weight of the parabolic portions of both the beams:

= 2 x 4'-9" x 41'-1" x 1'-6" x 150/3 lbs = 29.3 k

With the help of coefficients for fixed end moments of parabolic beams from Cross-and Morgan (Fig. 15 - p 139)

1) Distributed load -

$$FEM = \left(0.0956 + \frac{0.0068 \times 4}{5} \right) wl^2$$

$$= 0.10104 \times 4.76 \times (82.2)^2 = 3,250 \text{ k-ft}$$

2) Cross-girders

$$FEM = 10.7 \times 82.2 \, (0.026 + 0.113 + 0.196 + 0.152)$$

$$= 430 \text{ k-ft}$$

3) Parabolic load

$$F.EM = 29.3 \times 82.2 \times (0.01 + 0.103) = 273 \text{ k-ft}$$

Total F.EM = 4,150 k-ft

Moment Distribution

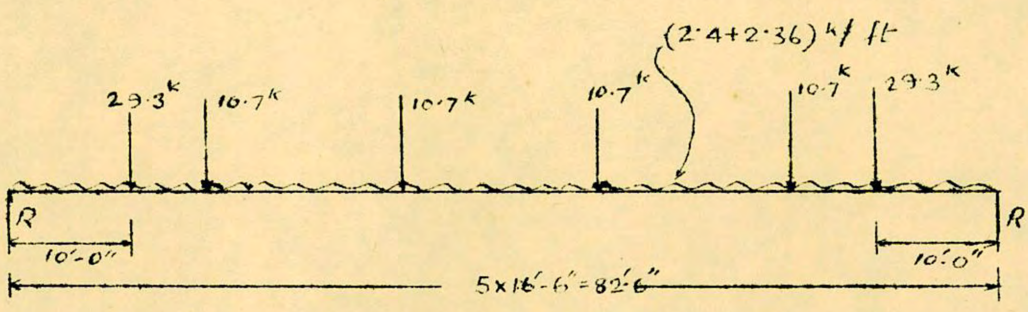
(Dead load Bending Moment)

Cantilever 0	1/3	Carry-over=0.65	1/3	0 Cantilever
- 4417	-4150		+ 4150	+ 4417
	- 89		+ 89	
	+ 58		- 58	
	- 19		+ 19	
	+ 12.4		- 12.4	
	- 4.1		+ 4.1	
	+ 2.6		- 2.6	
	- 0.9		+ 0.9	
-4417	-4190.0		+ 4190.0	+ 4417

-227 column head

+227 column head

Positive BM and SF due to dead load at mid-span of beam



R = 4.76 x 41.1 + 2 x 10.7 + 29.3 = 246.7 k

BM at mid-span = 246.7 x 41.1 - 29.3 x 31.1 - 10.7 x 24.6

- 10.7 x 8.1 - 4.76 x 41.1 x 41.1/2

= 4,872 k-ft

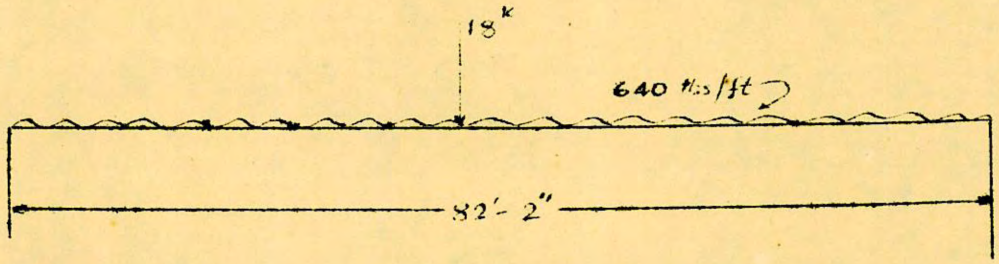
Total BM = 5,115 k-ft

Net positive moment at mid-span = 5,115 - 4,190 = 925 k-ft

Dead load SF at each end = 258 k

Fixed end moments due to live load in the horizontal beam

$$I = \frac{50}{82.2 + 200} = 0.177$$



FEM = $0.10104\ wl^2 + 0.175\ wl$

Including impact, FEM = $0.10104 \times 2 \times 1.177 \times 0.64(82.2)^2$

$+ 18 \times 2 \times 1.177 \times 0.175 \times 82.2 = 1{,}530$ k-ft

Moment Distribution

2/3	1/3	Carry-over = 0.65	1/3	2/3
Column	Beams		Beams	Column
	- 1530		+ 1530	
+1020	+ 510		- 510	-1020
	- 333		+ 333	
+ 222	+ 111		- 111	- 222
	- 72		+ 72	
+ 48	+ 24		- 24	- 48
	- 15.6		+ 15.6	
+ 10.4	+ 5.2		- 5.2	- 10.4
	- 3.9		+ 3.9	
+ 2.6	+ 1.3		- 1.3	- 2.6
+1303.0	- 1303.0		+1303.0	-1303.0

Positive moment at mid-span of the beam

$$\text{Free BM} = 18 \times 2 \times 1.177 \times 82.2/4 + \frac{640 \times 2 \times 1.77 \times (82.2)^2}{1,000 \times 8}$$

$$= 2,145 \text{ k-ft}$$

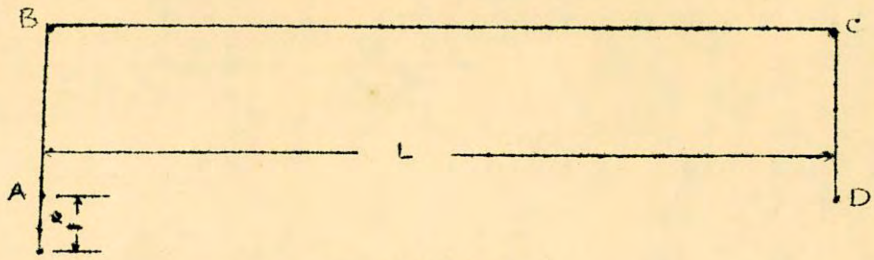
Net + ve BM = 2,145 - 1303 = 842 k-ft

L.L. SF at fixed end of the beam :

$$= \left(26 \times 2 + \frac{640 \times 2 \times 41.1}{1,000}\right) \times 1.177 = 120^{k}$$

So total SF at the end of the fixed beam = 258 + 120 = 378 k

BM due to sinking of left support by 1"



Stiffness of member BC = $\frac{4E\ I_A}{L} = \frac{11.8E\ I_C}{L}$

where I_A = average moment of inertia
I_C = moment of inertia of the section at middle
L = Length of the beam

So $I_A = \frac{11.8}{4} I_C$

$$M\ FBC = -\frac{6E\ I_A\delta}{L^2} = \frac{2x2x10^6 \times 6x\ \frac{11.8}{4} \times 18 \times 63^3}{1000 \times 986 \times 986 \times 12 \times 12} = \times 1 = 2,280\ \text{k-ft}$$

Moment Distribution

Column	Beam	Carry-over factor=0'65	Beam	Column
2/3	1/3		1/3	2/3
	+2280		+2280	
-1520	+ 760		- 760	-1520
	- 494		- 494	
+ 330	+ 164		+ 164	+ 330
	+ 106.4		+ 106.4	
- 71	- 35.4		- 35.4	- 71
	- 23.0		- 23.0	
+ 15.4	+ 7.6		+ 7.6	+ 15.4
	+ 5.0		+ 5.0	
- 3.4	- 1.6		- 1.6	- 3.4
-1250.0	+ 1250.0		+1250.0	-1250.0

Effect of temperature change and shrinkage

The range of temperature is assumed to be $\pm$ 30°F; shrinkage is equivalent to 15° fall in temperature.

$$H = 2\,C_2\,\alpha\,E\,t\,\frac{I}{h^2}$$

where H = horizontal thrust for change of temperature

C_2 = a constant

α = coefficient of thermal expansion per °F

t = change in temperature in degrees

I = moment of inertia of the horizontal beam (minimum for variable sections)

h = height of the column

Here $I = 7.5 \times 10^5$ in.^{4}

$\alpha E = 1{,}584$ lbs/ft.^{2}

$C_2 = 0.56$

$h = 34$ ft.

Horizontal thrusts at left hinge are :

Rise : $Hr = 2 \times .56 \times 1584 \times 30 \times \dfrac{7.5 \times 10^5}{12^4 \times 34^2} = -\,1{,}660$ lbs

Fall : $Hf = 2 \times .56 \times 1{,}584 \times 45 \times \dfrac{7.5 \times 10^5}{12^4 \times 34^2} = 2{,}490$ lbs

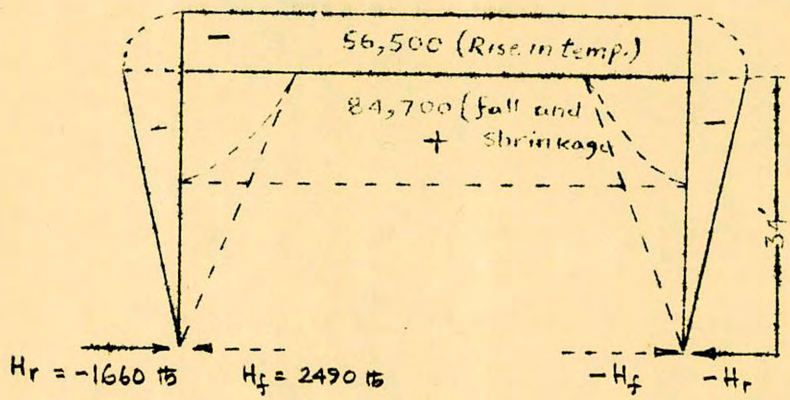
SUMMARY OF MOMENTS IN K-FT.

	Fixed end of left cant. (1)	Left end of fixed beam (2)	Head of left column (3)	Midpoint of beams (4)	Head of right col. (5)	Right end of fixed beams (6)	Fixed end of right cant. (7)
RIGID FRAME							
Dead load	- 4,417	- 4,190	- 227	+ 925	- 227	- 4,190	- 4,417
Live load Left cantilever loaded	- 2,050	- 900	- 1,150	+ 125	-1,150	+ 1,150	
Live load Main span loaded		- 1,303	+ 1,303	+ 842	+ 1,303	- 1,303	
For sinking of right support by 1"		- 1,250	+ 1,250		- 1,250	+ 1,250	
BALANCED CANTILEVER							
Dead load	- 4,417	- 4,417		+ 700		- 4,417	- 4,417
Live load Left cantilever loaded	- 2,050	- 2,050		- 1025			
Live load Main span loaded				+ 2,145			

Corresponding corner BM :

Rise : $M_B = M_C = - 1{,}660 \times 34 = - 56{,}500$ ft-lbs = -56.5 k-ft

Fall : $M_B = M_C = 2{,}490 \times 34 = 84{,}700$ ft-lbs = $+ 84.7$ k-ft

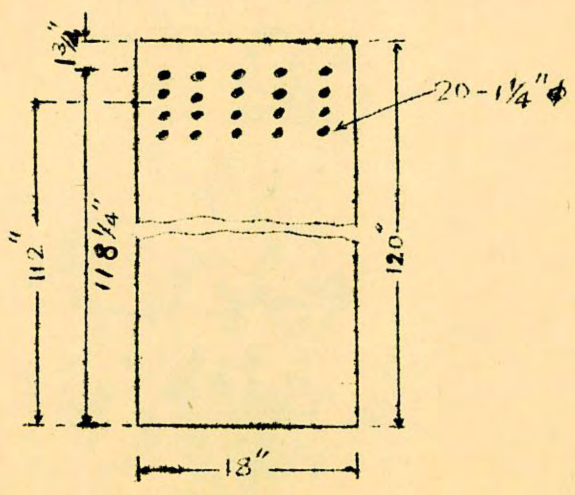


ANALYSIS OF SECTIONS :

A) Fixed end of left cantilever:

a) Moment:

Total negative moment = 6,467 k-ft



In the section provided,

$$d = 120 - 1\tfrac{3}{4} - 5 \times 1\tfrac{1}{4} = 112 \text{ in.}$$

$$A_s = 20 \times 1.27 = 25.4 \text{ in.}^2$$

Allowable moment, $M = A_s f_s jd = \dfrac{2 \times 25.4 \times 20{,}000 \times 0.87 \times 112}{12{,}000}$

$$= 8{,}000 \text{ k-ft}$$

Ultimate moment of resistance :

$$\text{Actual } p = \frac{25.4}{18 \times 120} = 0.0121$$

$$p \max = 0.75 \times .85 \times .85 \; \frac{3{,}000}{40{,}000} \times \frac{87{,}000}{87{,}000 + 40{,}000} = 0.0278$$

So failure by yielding is assured.

$$a = \frac{A_s f_y}{.85 f_c' \times b} = \frac{25.4 \times 40{,}000}{.85 \times 3{,}000 \times 18} = 21.3 \text{ in.}$$

$$M_u = A_s f_y (d - a/2) = 2 \times \frac{25.4 \times 40{,}000}{12{,}000}(112 - \frac{21.3}{2})$$

$$= 16{,}500 \text{ k-ft}$$

b) Diagonal tension:

Total shear = 440 k

$$v = \frac{V}{bd} = \frac{440 \times 1000}{18 \times 112} = 218 \text{ psi}$$

Allowable v without web reinforcement = $1.1\sqrt{f_c'} = 55$ psi

So stirrups are to be provided.

$A_v = \dfrac{V'S}{f_v d}$ where V' = shear carried by web reinforcement

S = spacing of stirrups = 6"

f_v = tensile stress in web reinforcement

$$= \frac{(218 - 55) \times 18 \times 112 \times 6}{20,000 \times 112} = 0.88 \text{ in.}^2$$

Two-legged stirrups of ½" dia has area

$$= 2 \times 2 \times \frac{3.14}{4} \times \frac{1}{4} = 0.8 \text{ in.}^2$$

As two-legged **stirrups** of ½" dia @ 6" c/c have been provided, the section remains slightly weak in diagonal tension.

c) Bond:

ACI Code (1963) specifies

Bond stress not to exceed $\frac{3.4}{D}\sqrt{f_c'} = \frac{3.4 \times \sqrt{3,000}}{1.25}$

$= 148$ psi. nor 350 psi for top bars.

$$u = \frac{V}{\Sigma ojd} = \frac{440,000}{2 \times 3.14 \times 1.25 \times 20 \times .88 \times 112} = 28.4 \text{ psi}$$

o.k.

B) Left end of fixed beam :

a) Moment

Total negative moment = 6,393 k-ft

This is less than the moment at the fixed end of the cantilever, but the same reinforcements and section are used. So it is o.k.

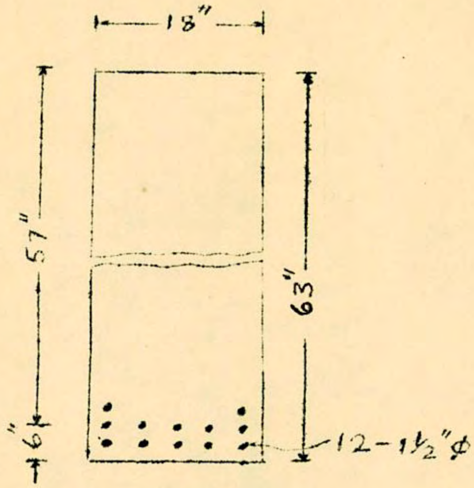
b) Total shear = 378^k

This is less than the shear at the fixed end of the cantilever, so it is o.k.

c) Total shear = 378^k

This is less than the shear at the fixed end of the cantilever, so bond stress is o.k.

C) Mid-section of the fixed beams:



a) Moment-

Total positive moment = 2,017 k-ft.

In the section provided,

$d = 57''$

$A_s = 12 \times \frac{3.14}{4} \times (3/2)^2 = 21.2 \text{ in.}^2$

$f_y = 40,000$ psi

$f_c' = 3,000$ psi

Allowable moment, $M = A_s f_s jd = \frac{2 \times 21.2 \times 20,000 \times .87 \times 57}{12,000}$

$= 3,560$ k-ft

Ultimate moment of resistance:

Actual steel ratio, $p = \frac{21.2}{18 \times 63} = 0.0187$

$$P_{max} = 0.75 \times .85 \times k_1 \frac{f_c'}{f_y} \times \frac{87,000}{87,000 + f_y} = 0.028$$

So failure by yielding is assured.

$$a = \frac{A_s f_y}{.85 f_c' b} = \frac{21.2 \times 40,000}{.85 \times 3,000 \times 18} = 18.5 \text{ in.}$$

$$M_u = A_s f_y (d - a/2) = \frac{2 \times 21.2 \times 40,000}{12,000} (57 - 18.5/2)$$

$$= 6,800 \text{ k-ft.}$$

PHOTOGRAPHIC PLATES

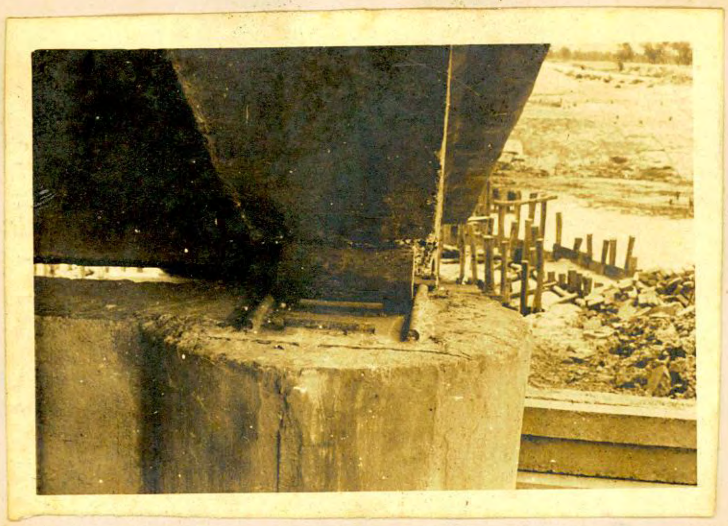


Plate 2.3 Mathabhanga Bridge
(Showing rollers in displaced position)

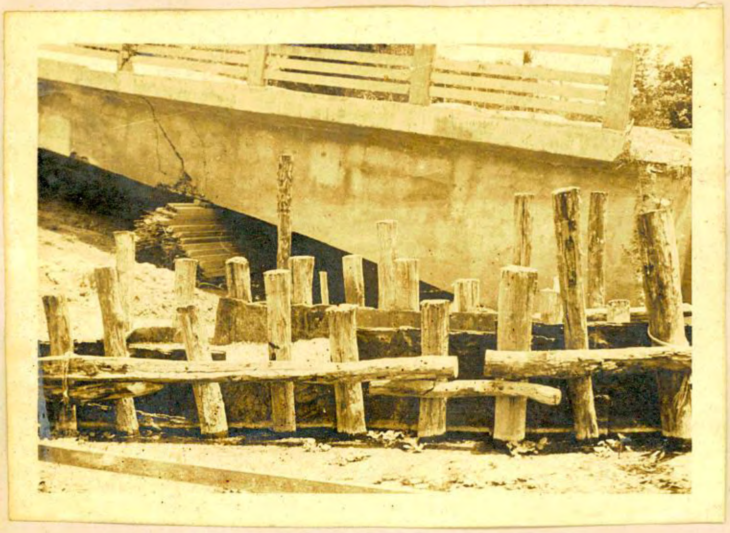


Plate 2.4 Mathabhanga Bridge
(The ring embankment for investigation)

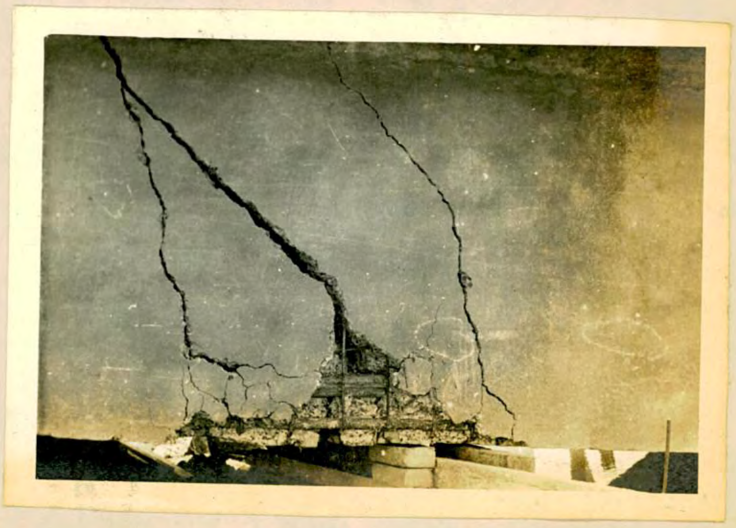


Plate 2.5 Mathabhanga Bridge
(Cracks in the girder due to impact on falling)

Plate 5.1 Tongi Bridge
(The general view showing the spans,girders,etc)

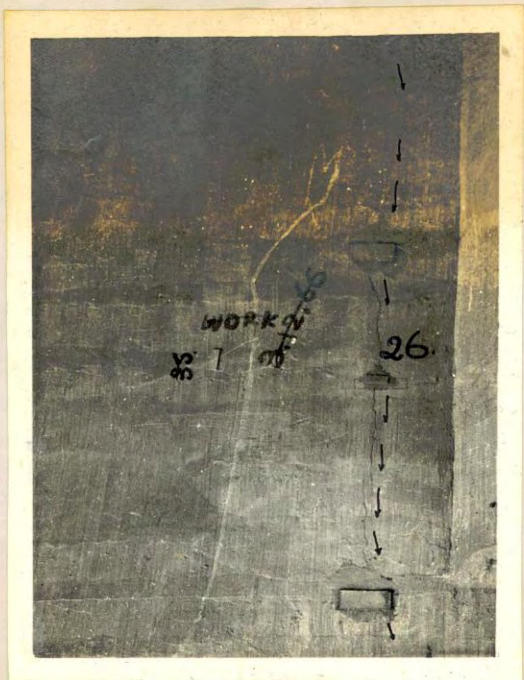


Plate 5.2 A Tongi Bridge
(Vertical crack in the down-stream main girder near the pier)

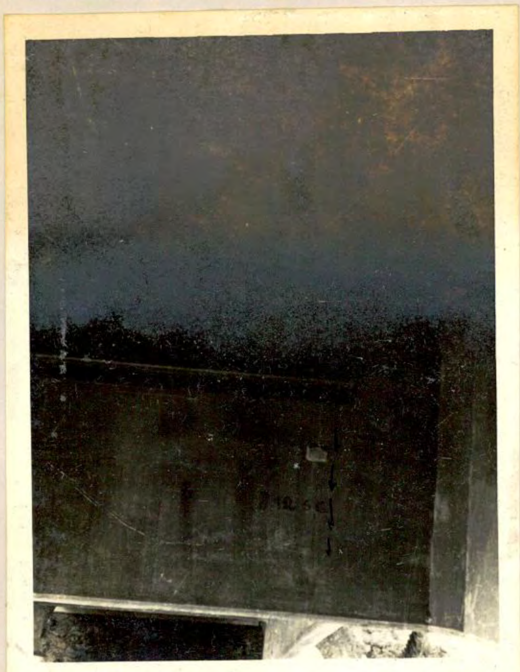


Plate 5.2 B Tongi Bridge
(Crack in the cross-girder at articulation, showing the bolts that connected two opposite cross-girders at articulation)

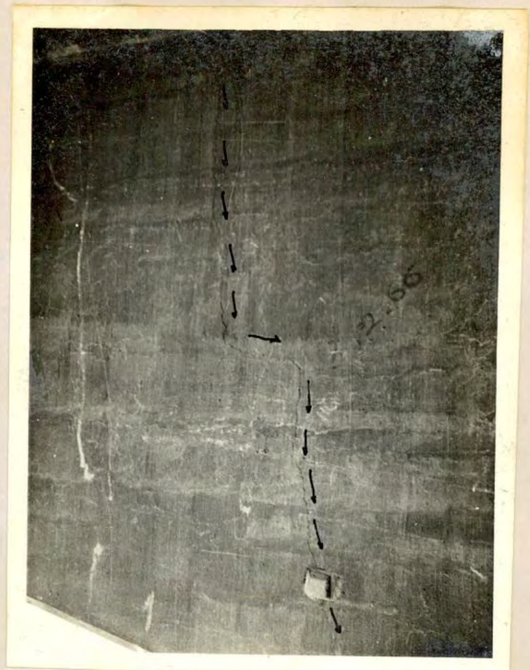


Plate 5.2 C Tongi Bridge
(Crack in the upstream main girder; the wide crack is vertical, then horizontal and vertical again at the bottom)

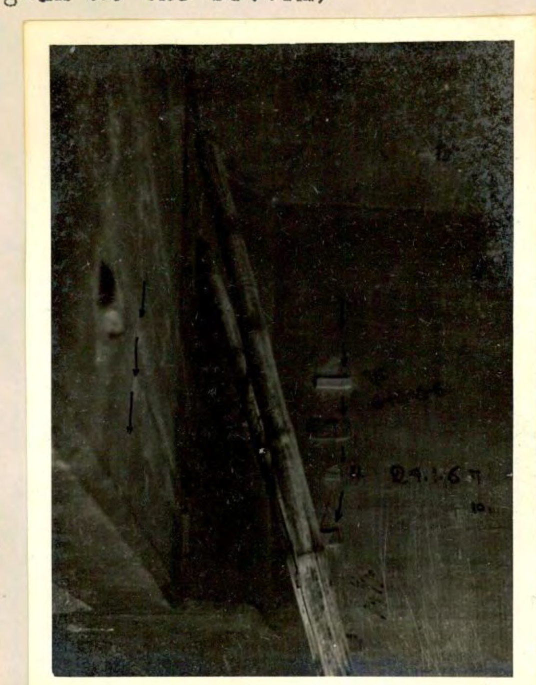


Plate 5.2 D Tongi Bridge
(Cracks in the main girder in the negative zone and in the cross-girder at the pier)

Plate 6.1. A Bhangaghat Bridge
(Extra shore-span under construction)

Plate 6.1. B Failure of the wing-wall

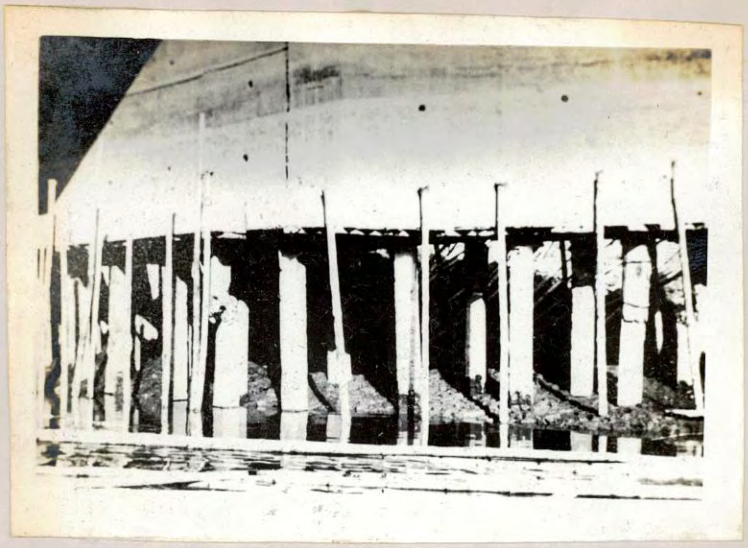


Plate 6.2 Failure by scouring and subsequent breaking of piles under the abutment

Plate 6.3 Failure of abutment
(Abutment collapsed, but wing-wall remained in position)

Plate 6.4 Failure of wing wall

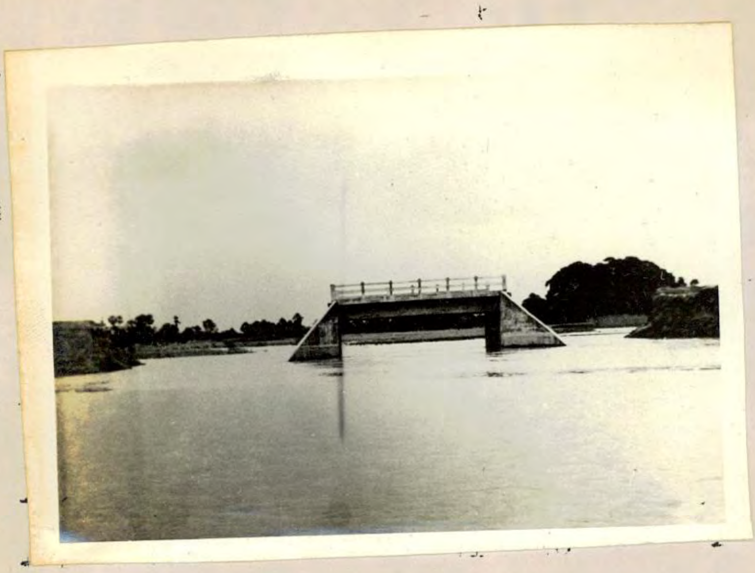


Plate 6.5 Failure due to insufficiency of water-way
(The approaches were washed away)

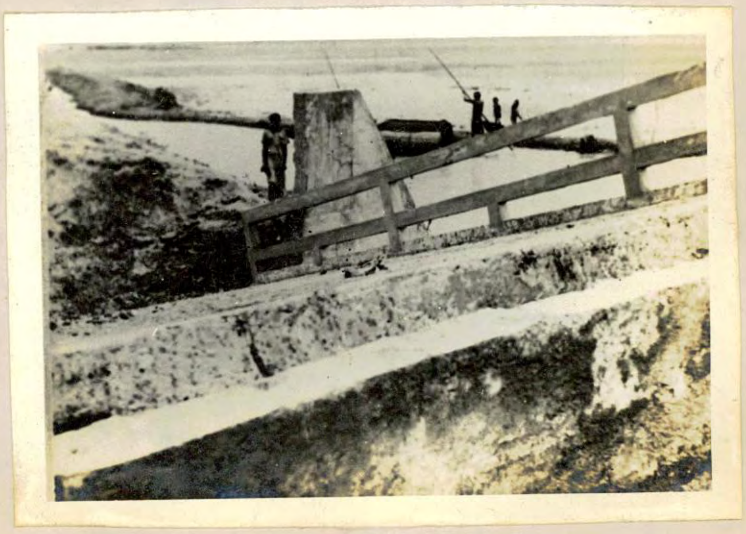


Plate 6.6 Failure of abutment

Plate 6.7 Failure due to excessive scouring at the foundation.

Plate 6.8 Failure due to non-functioning of the expansion bearings (Horizontal cracks in the masonry abutment)

Plate 6.10 Failure of abutment reflected at articulation

REFERENCE

R E F E R E N C E

1.	Eusufzai, M.H.S.H.K..	Flood Control in East Pakistan- a master's thesis submitted to the Texas A & M College, USA.
2.	Cross, Hardy and Morgan, N.D.	Continuous Frames of Reinforced Concrete - John Wiley & Sons Inc.
3.	Taylor, Thompson & Smulski	Reinforced Concrete Bridges, 1963.
4.	Czechy, D.	Foundation Failure.
5.	East Pakistan Water and Power Development Authority	Annual Report on Flood in East Pakistan for 1963 - EPWAPDA Water Supply Paper - 182.
6.	- do -	Technical Report on Flood in East Pakistan ; EPWAPDA Water Supply Paper - 7.

7. East Pakistan Water & Power Development Authority — Gauge and discharge Observations for Ganges River at Hardinge Bridge, Paksey, 1933-58; EPWAPDA Water Supply Paper - 2.

8. Chowdhury, M. I. — Morphology of Bengal Basin- a master's thesis submitted to the University of Camb.

9. Abbas A.T., B.M. — Utilisation and development of the Deltaic Area of East Pakistan. A report for the Regional Symposium on Flood Control, in Bangkok, July, 1963.

10. The American Association for State Highway Officials. — Standard Specifications for Highway Bridges, 1957.

11-15. Directorate of Roads and Highways (Design Division) — Design, drawing and report of Mathabhanga River Bridge

Nayarhat Bridge

Mirpur Bridge

Tongi Bridge

Bhangaghat Bridge