About the Lecturer - IGS 1994

Dr. Nayak graduated in Civil Engineering from the University of Bombay in the year 1959. He secured his M.Tech. in Geotechnical Engineering from the Indian Institute of Technology, Bombay in the year 1963 and his Ph.D. from University of Winsconsin, USA in the year 1970. After obtaining his Ph.D. from USA, Dr. Nayak moved to University of Panama, Panama, South Central America as a Professor and returned to India in the year 1972 after working year and half with the University of Panama.

After returning to India, Dr. Nayak worked with M/s Tata Consulting Engineers (TCS), Bombay for 8 years from 1972 to 1980 as Head of Geotechnical Engineering Wing.

Dr. Nayak joined M/s Asia Foundation and Constructions Limited (AFCONS), Bombay, as Senior Engineer in the year 1980. Presently he is working as Executive Director - Projects with the same Organisation. As Executive Director - Projects, he is actively associated with execution of all types of works undertaken by AFCONS including geotechnical investigation, slope stabilisation, ground improvement, underpinning of micro piles, execution of diaphragm walls, bridges, tunnels, submarine pipelines, highways docks and harbours.

In AFCONS Dr. Nayak has been responsible for extensive development and utilisation of rammed stone columns, cement bentonite slurry walls and high pressure jet grouting.

Dr. Nayak has published many papers in India and abroad and he is the author of the book "Foundation Design Manual", which was first published in 1980 and fourth edition of this book is expected in 1995.

Dr. Nayak has been actively associated with Indian Geotechnical Society since 1977. He worked as Executive Committee Member of the Indian Geotechnical Society for about 8 years. He was Organising Secretary of IGS Conferences held at Bombay in the years 1980 and 1990. He has also been a Member of Technical Committee (TC-6) on Expansive Soil and also Technical Committee on Preservation of Old Monuments set up by International Society of Soil Mechanics and Foundation Engineering. As Secretary of IGS Bombay Chapter, Dr. Nayak has organised numerous Seminars, Workshop, Short Courses for the benefit of Engineering Community of Bombay.

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IGS - Lecture 1994

Innovative Developments in Indian Geotechnical Practices

by

N.V. Nayak*

Introduction

The IGS Lecture was first introduced in the year 1978 and the 1st speaker was the distinguished professor and geotechnical expert Dr. R.K. Katti. Since then, these lectures are arranged every year. The lecture for the year 1993, has been delivered only yesterday by Prof. T.S. Nagaraj. The Table 1 lists the sixteen speakers who have spoken on diverse topics of interest so far.

Contribution of construction industry in the development of innovative practices in geotechnical engineering applications is very significant. Space limitation will not permit even to highlight all such contributions made by various construction firms. Hence, the author would like to restrict himself to some of the contributions made by M/s. Asia Foundations and Constructions Ltd (AFCONS), wherein he is working as Executive Director – Projects. Here, again only very few contributions are highlighted from field of :

a) Tunnel Construction
b) Bridge Construction
c) Slope Stabilization Measures
d) Special Marine Works

* Executive Director – Projects, Asia Foundations and Constructions Ltd (AFCONS), Bombay, India

TABLE 1
Speakers and Topics of IGS Annual Lectures - 1978 to 1994.

IGS Lecture No. (1)	Lecture Of the year (2)	Speaker (3)	Organisation (4)	Topic (5)
1	1978	Prof.R.K.Katti	Professor, I.I.T. Bombay.	Search for Solutions to Problems in Black Cotton Soils.
2	1979	Mr.Y.K.Murthy.	Chairman, Central Water Commission, New Delhi.	Some Challenging Geotechnical Problems in River Valley Projects in India.
3	1980	Prof.Dinesh Mohan.	Director, CBRI, Roorkee.	A Close Look at Problems of Research and its Applications to Pile Foundations.
4	1981	Mr.K.R.Datye.	Consulting Engineer, Bombay.	Simpler Techniques for Ground Improvement.
5	1982	Mr.H.C.Verma.	Chairman & M.D., AIMIL, New Delhi.	Need, Potential and Might of Geotechnical Instrumentation in India.
6	1983	Prof.Shamsher Prakash	Director, CBRI, Roorkee.	Past and Future of Geotechnical Earthquake Engineering.
7	1984	Prof.Jagdish Narain.	Secretary, Association of Indian Universities, New Delhi	Reinforced Earth.
8	1985	Prof. T. Rammurthy.	Professor, IIT, Delhi.	Stability of Rock Mass.

TABLE 1 Cont.

(1)	(2)	(3)	(4)	(5)
9	1986	Dr.R.K. Bhandari.	Director, CBRI, Roorkee.	Slope Instability in The Fragile Himalaya and Strategy for Development.
10	1987	Mr.K.Madhavan	Member, Central Water Commission, New Delhi.	Critical Issues in Geotechnical Enginnering of Water Resource Projects.
11	1988	Prof.Gopal Ranjan	Professor, University of Roorkee.	Ground Treated with Granular Piles and Its Response Under Load.
12	1989	Prof.S.K.Gulhati.	Professor, IIT, Delhi.	Geotechnical Aspects of Indian Offshore Environment.
13	1990	Prof. A.Sridharan.	Professor, IISc., Bangalore.	Engineering Behaviour of Fine Grained Soils - a Fundamental Approach.
14	1991	Dr.C.D.Thatte	Chairman, Central Water Commission, New Delhi.	Earthquakes, Dam Design and Tehri Project.
15	1992	Dr.B.G.Rao.	Scientist, CBRI, Roorkee.	Behavioural Prediction & Performance of Structures on Improved Ground and Search for New Technology.
16	1993	Prof.T.S.Nagaraj.	Professor, IISc., Bangalore.	Effective Stresses in Soils.
*17	1994	Dr.N.V.Nayak	Director, Asia Foundations & Constructions Ltd., Bombay.	Innovative Developments in Indian Geotechnical Practices.

* 17th IGS Lecture on 27th December 1994 at Warangal.

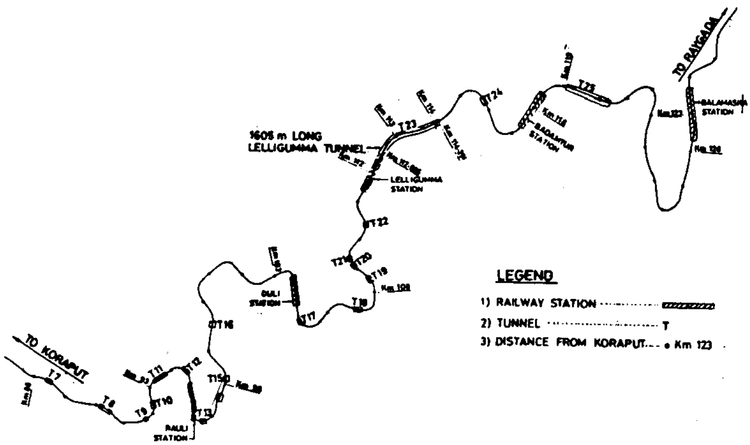


FIGURE 1 Location Plan of Lelligumma Tunnel on Rayagada-Koraput Railway Line Under Construction on South Eastern Railway

Developments in Tunnel Construction Practices

Stabilisation by Jet Grouting

South Eastern Railways (S.E.R) have undertaken the work of laying a 164 km single track, broad-gauge railway line, connecting Koraput and Rayagada in Orissa State. The railway alignment is passing through Eastern Ghat mountain ranges. There are 34 tunnels on this section. 1605 m long Tunnel No. 23, between km 112.686 and km 114.291, near Lelligumma (Fig. 1) is at a distance of approximately 48 km from Rayagada.

This tunnel has a diameter of 6.090 m in heading portion and overall height of 6.606 m and total cross sectional area of 41.57 m^2. The tunnel was provided with concrete lining of 260 mm thickness (Fig. 16). The work of construction of this tunnel was started in July 1988, by Premier Engineering Syndicate, from both the ends of tunnel. Complete tunnel work between Ch 00 – Ch 308 (308 m) and Ch 705 – Ch 1605 (900 m) and work of heading only, between Ch 308 – Ch 361 and Ch 669 – Ch 705, was completed by May 1991. Thus, in 1208 m of tunnel portion no problems were encountered (Fig. 2). Excavation was done by heading and benching, adopting conventional drilling/blasting. Temporary supports were provided by steel sections, which were later encased in concrete lining.

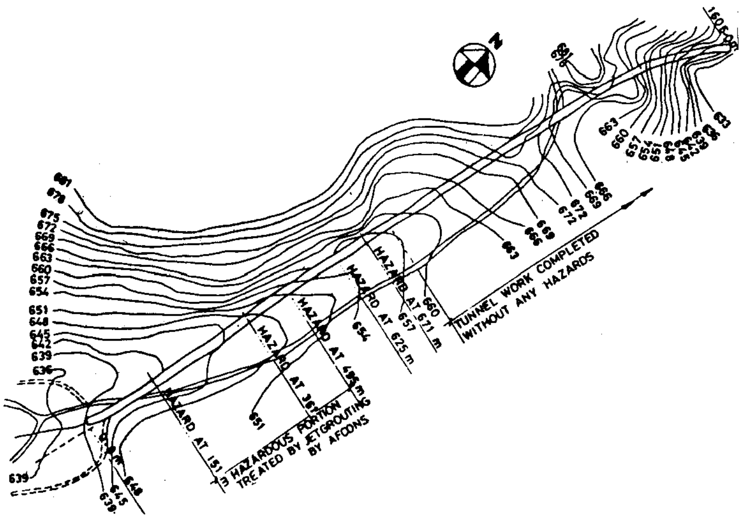


FIGURE 2 Locations of Hazards in Lelligumma Tunnel

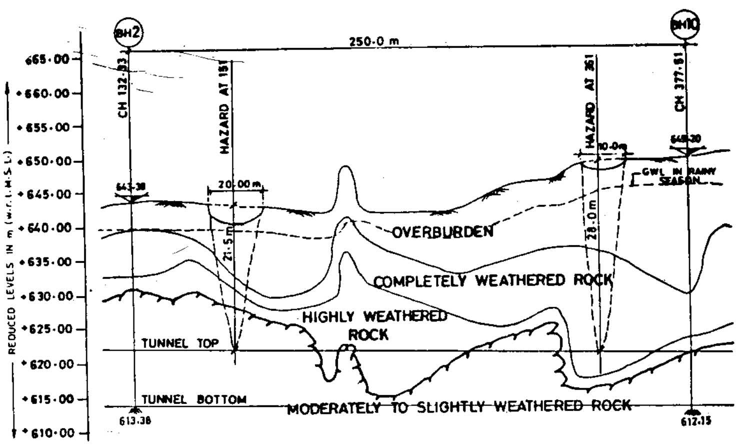


FIGURE 3 Longitudinal Section of Lelligumma Tunnel between Ch : 132 to 337 from Koraput End

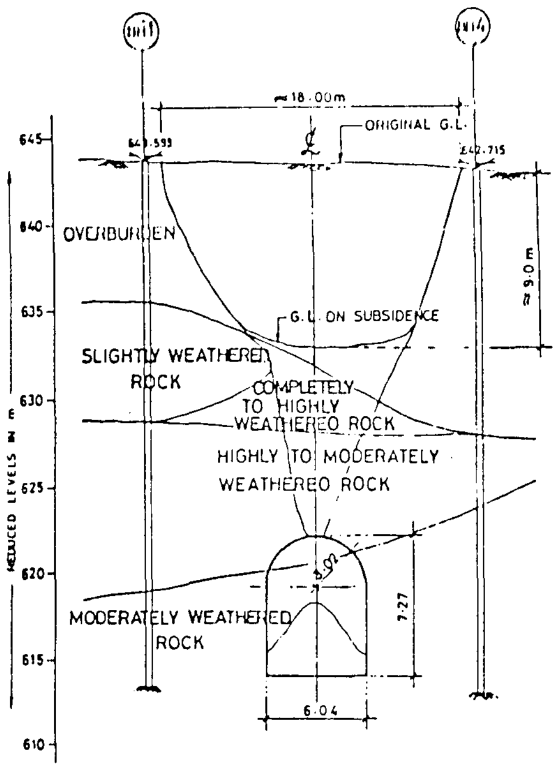


FIGURE 4 Cross Section of Lelligumma Tunnel at Ch : 151 from Koraput End

As work of excavation in benching portion commenced at Ch 361, the material lying above the crown, started collapsing into the tunnel and a chimney formation took place above the tunnel on 15th July, 1991. A pit of almost 10 m diameter at top and a depth of 6-8 m was formed above the crown of the tunnel(Fig. 3). There was heavy ingress of water into the tunnel from this pit. The tunnel was filled up with mud, boulders and soft rock which had fallen from the pit. Cement grouting was attempted to tackle the problem, but this did not prove successful.

At this stage, S.E. Railways awarded the work of construction of balance full 308 m of tunnel, +82 m benching and treatment of hazard at Ch 361, to another specialised agency viz. M/s. Asia Foundations and Constructions Ltd (AFCONS). Before the finalisation of awarding of this contract, another hazard took place on 10th October, 1991 at Ch 151, in the already excavated tunnel portion, where earlier no problems were encountered during the construction phase till October'91. This hazard punctured the tunnel at top, dislodged the rib supports erected. A pit of almost 9-12 m depth, 20 m in diameter at top and 2-3 m diameter at bottom (tunnel crown level) was formed(Fig. 4). The tunnel was filled up with almost 3500 cum of saturated soil, boulders and soft rock fallen from the chimney formed. Heavy ingress of water continued from this place into the tunnel.

Additional Investigation

To understand the collapse phenomenon in detail, AFCONS decided to carry out additional investigation by carrying out drilling of boreholes, collection of soil/rock samples and conducting packer permeability tests. As a part of geotechnical investigation, 25 boreholes varying in depth from 27 m to 47 m were drilled in period from 15.10.91 to 7.1.92, between Ch 112.814 and Ch 113.340.

These investigations brought into focus the hitherto unknown geological aspects, which had not been brought out by previously conducted soil investigations. These can be briefly summarised as follows

A. At many places, the material above the crown of the tunnel was predominantly soil overburden, consisting of hard/dense sandy clay/clayey sand, which used to get fully saturated with ground water during the monsoon.

B. The rock cover was formed of highly to moderately weathered rocks made up of

   a. Quartz silimanite schist, or
   b. Charnockite or
   c. Banded granitized quartz/silimanite gneiss.

C. Through the contact/joint planes between different rock units and along the shear planes, water was freely circulating, resulting into continuous weathering of rocks.

D. At Ch 151, the material overlying the crown of the tunnel, was almost entirely made up of completely and highly weathered rock. When the ground got saturated with water, with water table just 4 m below ground, these formations yielded, resulting into puncturing of the crown portion of tunnel and causing a mud flow carrying with it approximately 3500 cum of debris. (Fig. 5). The maximum ground subsidence at this location was approximately 9-12 m.

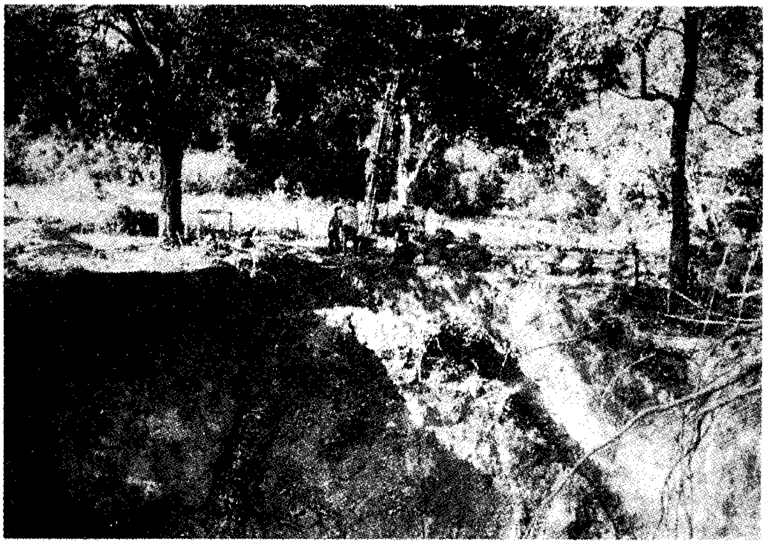


FIGURE 5a Details of Cavity at Ch. 151

FIGURE 5b Details of Cavity at Ch. 151

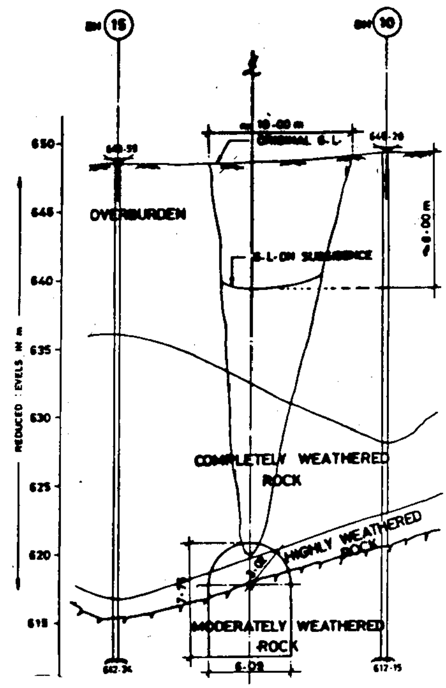


FIGURE 6 Cross Section of Lelligumma Tunnel at Ch : 361 from Koraput End

E. At Ch 361, the material overlying the crown of the tunnel, was completely weathered rock (Fig. 6).

F. At Ch 671, when the first controlled blast was taken, a heavy collapse accompanied by mudflow and ingress of water was experienced. This bent the steel ribs provided earlier and 10 to 15 m high cavity was developed above the crown of tunnel.

G. On the basis of geotechnical investigation conducted, it was predicted that similar hazard conditions were likely to be encountered at the following locations

Ch 350 to Ch 361	=	11 m length
Ch 470 to Ch 495	=	25 m length
Ch 565 to Ch 625	=	60 m length
Ch 651 to Ch 671	=	20 m length

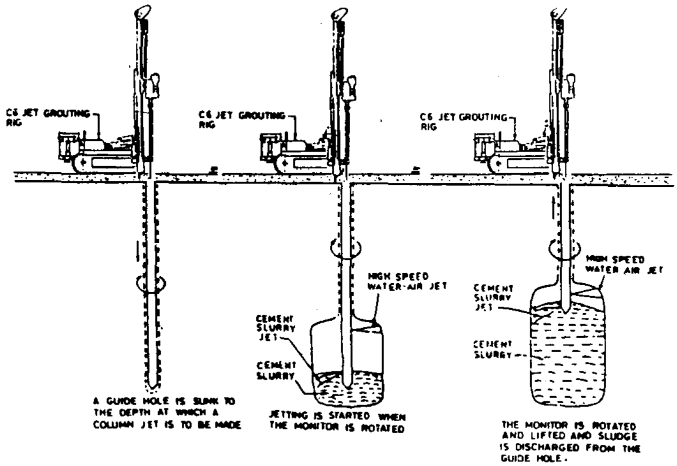


FIGURE 7 Schematic Details of Jet Grouting Procedure

The rockmass offered very unfavourable or very poor support conditions for tunnel.

H. Attempts to stabilise the ground by conventional drilling/grouting had not yielded satisfactory results. Hence, it was decided to use jet grouting procedure to stabilise rockmass.

Jet Grouting Methodology

The technique termed as jet grouting, was first developed in Japan in the late '70s and was introduced for the first time in India, by AFCONS in 1983.

It may be classified as 'permeation grouting', in which, the improvement of soils is carried out by reducing permeability and increasing strength by various injection techniques. By this technique, the grout fills the voids without any essential change to the original soil volume and structure. In 'jet grouting' the soil is mixed in - place, with a stabilising mixture under a very high nozzle pressure. By using this technique, it is possible to treat a wide range of soils and even weathered rocks by use of a simple cement grout mixed in place with soil particles under very high nozzle pressures of even upto 300 - 400 kg/cm^2. 'Jet grouting 1' implies the use of a 'single fluid (the grout) as a fracturing medium and stabilising agent, when single phase system is adopted.

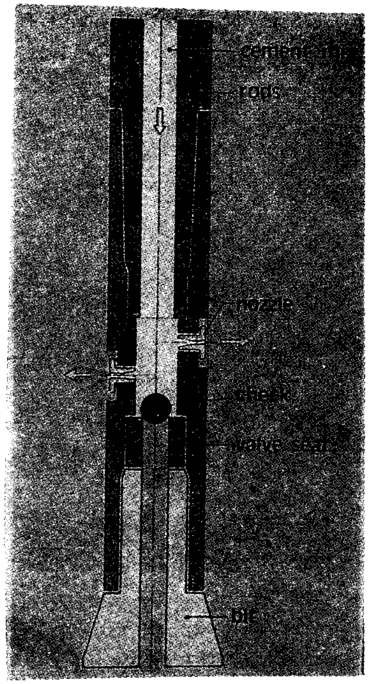


FIGURE 8 Details of Jetting Tool for Single Phase System of Jet Grouting

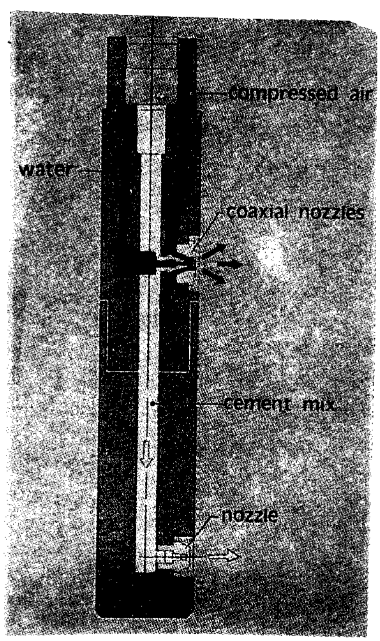


FIGURE 9 Details of Jetting Tool for Three Phase System of Jet Grouting

'Jet grouting 3' means the use of 3 fluids, air and water as the fracturing and washing media and grout as the stabilising agent, when three phase system is used.

In 'Jet grouting 3', soil is cut by cutting action of air-water jet, at pressure of 300 to 400 kg/cm^2 and removed by water flow and is simultaneously replaced by cement grouting (at 40 kg/cm^2 pressure). This system is known as the three phase (air, water and grout) procedure (Fig. 7).

In three phase jet grouting process, the monitor, which comprises a triple phase fluid drill pipe, conveying three process elements of air, water and cement grout, is lowered into predrilled hole.

There are two nozzles (Figs. 8 and 9) one above the other, separated by a spacing of 500 mm. The upper nozzle is of 4 mm diameter and the lower nozzle is of 7 mm diameter. Air and water are passed through the upper nozzle. This jet is called cutting jet and consists of a concentric arrangement, in which water is surrounded by air. Water is supplied through a central 1.8 mm diameter hole at a pressure of 300-400 kg/cm^2. The cutting action of jet, is made more effective, by enshrouding it within a collar of compressed air at 7 kg/cm^2 pressure. Cement grout is passed through the lower level nozzle, at a pressure of 40 kg/cm^2, in the void that has been formed, by cutting action of water jet. Generally, the cutting action of jet is effective, upto a distance of 1.5 m from the nozzle. The maximum size of jet grout column formed, is generally between 1.0 to 2.0 m, depending upon the type and consistency of strata. Fig. 8 gives the details of jetting tool for single phase system.

After the monitor has been inserted to the bottom of the predrilled hole, cutting by water jet and simultaneous grouting by cement grout is started. The spent air is lifted to surface via the borehole. Work is continued by rotating and raising the monitor evenly and continuously until the desired depth has been grouted. The main advantage of this method, is that grouting can be stopped at any defined level below the surface, without disturbing of the remaining overlying soil. After stopping the cutting action at desired level, the monitor is simply withdrawn to the top of hole, maintaining only a restricted grout flow to fill up the drilled guide hole.

In brief, the jet grouting technique aims at treating the unstable stratum, by replacement of weaker materials with stronger materials, to enable the same to bear the overburden on the top, by arch action transferring the loads to stable supporting media.

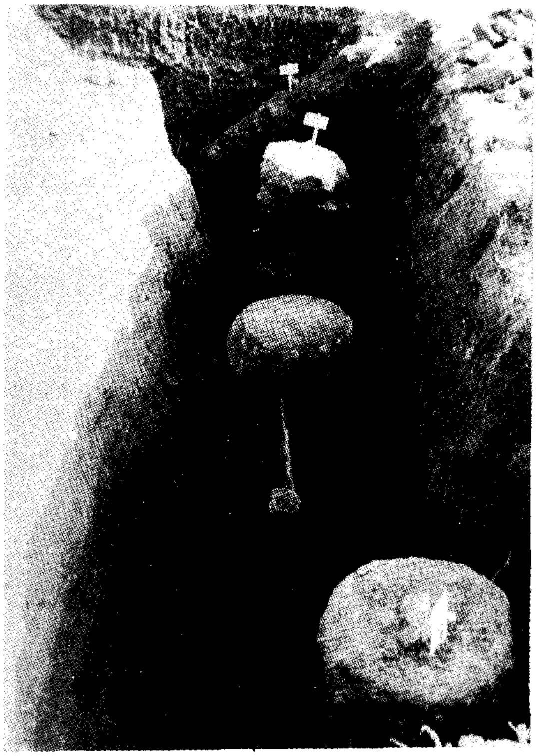


FIGURE 10 Close up view of Jet Grout Columns formed in Trial Study

Drilling Machinery

Drilling for Lelligumma tunnel jet grouting work, was carried out using an imported hydraulic drilling rig, called 'Casagrande C-6 rig'. This is a self contained, track mounted, hydraulic drilling rig, which can be easily manoeuvered in all types of ground conditions. It is powered by a 105 HP diesel engine and is able to develop high torque/speed ranges. It can drill boreholes upto 300 mm diameter.

At this site, 115 mm diameter boreholes were drilled, using special type drill rods, (63 mm OD/51 mm ID) with male female joints and tricone roller button bits, using a T-150 power swivel. The drilling was done at rate between 80 to 100 rpm. Bentonite slurry was used as a stabilising medium. This machine is basically used as a drilling machine, for carrying out drilling in soft formations. With this machine, it was possible to drill and grout 3 to 4 holes of maximum 40 m depth, in 3 shifts of 7½ hours each.

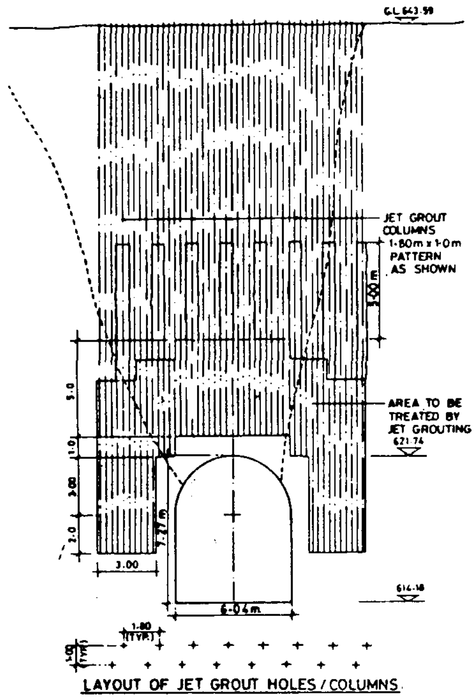


FIGURE 11 Initial Proposal of Jet Grouting from Top of Hill at Ch. 151

Jet Grouting at Lelligumma Tunnel

As mentioned earlier, soil investigation revealed presence of highly weathered rock, which needed to be consolidated to form an 'Umbrella Arch' of improved soil around the tunnel excavation face.

In order to study the effectiveness of this treatment and verify various treatment parameters, a field trial was conducted at site, using single phase system 'Jet Grout 1'. Four jet grout columns each of 4 to 5 m length, were formed using different water/cement ratios and withdrawal speed of jet grouting equipment Fig. 10 shows exposed jet grout columns formed in trial study.

Initially, it was proposed to carry out vertical jet grouting (Fig. 11) by forming a suitably supported platform at the bottom of cavity, and carrying out drilling by erecting rig on this platform. As, this platform formation was an extremely difficult proposition, an alternative methodology was devised.

Drilling rig was kept on top of hill and inclined drill holes upto required depth were formed (Fig. 12 and 13). Only partial depth of these holes was jet grouted, so that this portion could suitably form an arch over excavation face.

After studying the results of geotechnical investigation and field trial, it was decided to adopt a pattern of 60 cm × 60 cm for drilling the guide holes and a height of around 550 cm for formation of the grout column to support the load. Fig. 14 shows jetting action of equipment on ground at Ch 151. Fig. 15 shows a view of Lelligumma tunnel at Koraput face. Fig. 16 shows finished cross section of the tunnel.

Effectiveness of Jet Grouting

Vertical or inclined drilling and jet grouting have effectively stabilised the unstable strata in tunnelling work at Lelligumma. Areas which were anticipated to be hazardous, were properly treated in advance, by jet grouting and later on no problems were faced during the excavation of tunnel in these stretches. Whenever jet grouting treatment was given, excavation of tunnel face was carried out by using hand held jack hammers, allowing a 2 m clearance on either end of treated portion, where blasting was not carried out. By adopting jet grouting, out of 308 m of balance tunnel work, 153 m of tunnel has been successfully completed by AFCONS, by March 1994.

Tunnelling in Soft Ground Using Shield and Jet Grouting

Eastern Railways have constructed 17 km of first Underground railway in India, more popularly known as 'Calcutta Metro'. Metro Railway runs between Dum Dum Railway Station (Ch 0.0) km to Tollyganj Railway Station (Ch 16.25) km.

Trains of Metro Railway running underground, are brought to ground level workshop or car depot for routine maintenance every day. In order to provide, this way of passage below the existing 6 railway tracks of Eastern Railway, it was decided to construct an Underpass. The job required construction of Underpass beneath the 6 existing railway tracks (Fig. 17 and Fig. 18), without disturbing the traffic. Two Tunnels of 5.1 m inside diameter (one for upline and other for downline) were to be constructed for this purpose.

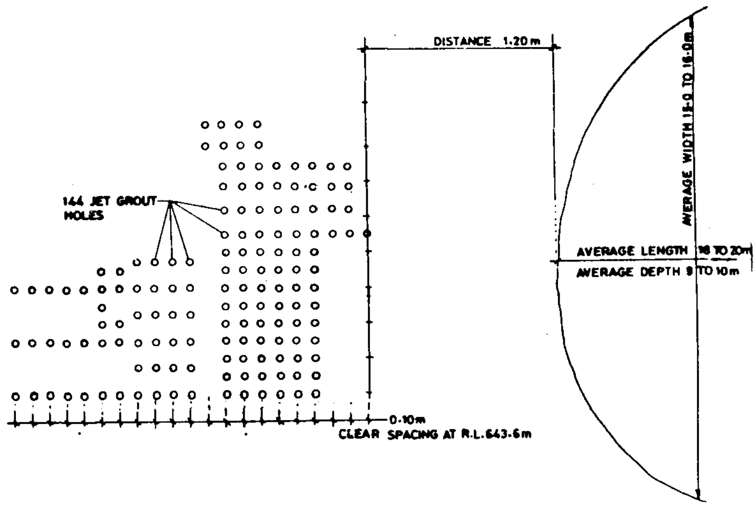


FIGURE 12a Layout of Jet Grout Holes in Plan at Ch. 151.00 m

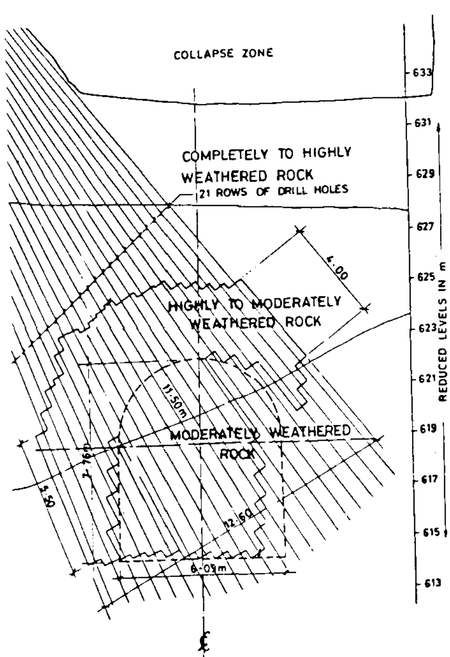


FIGURE 12b Details of Jet Grouting Treatment at Plan at Ch. 151 from Koraput End

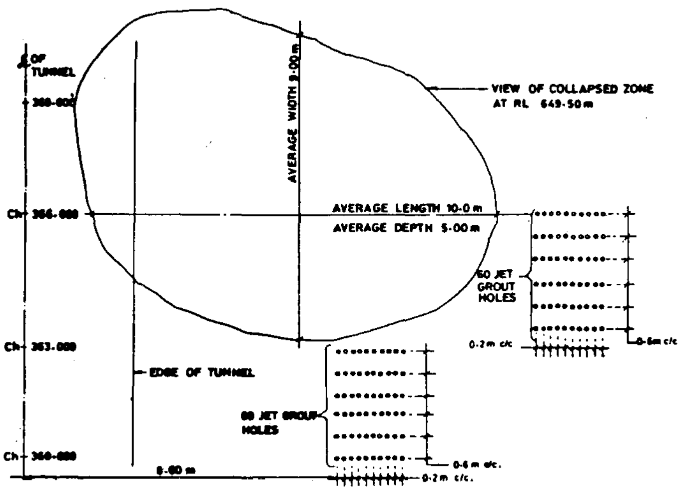


FIGURE 13a Layout of Jet Grout Holes in Plan at Ch. 361.00 m

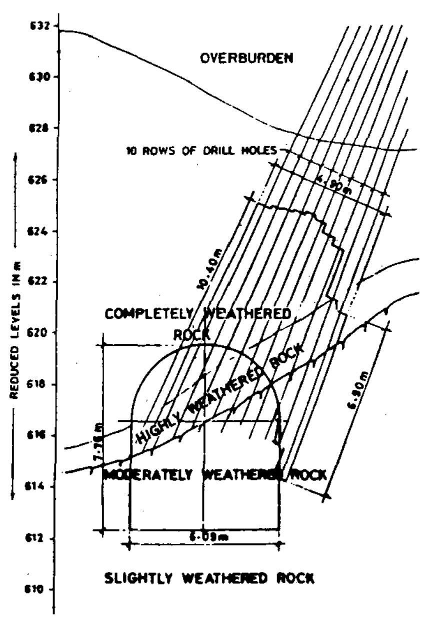


FIGURE 13b Details of Jet Grouting Treatment at Plan at Ch. 361 from Koraput End

FIGURE 14 Jetting Action of C–6 Ring on Ground at Ch. 151

FIGURE 15 A View of Lelligumma Tunnel at Koraput Face

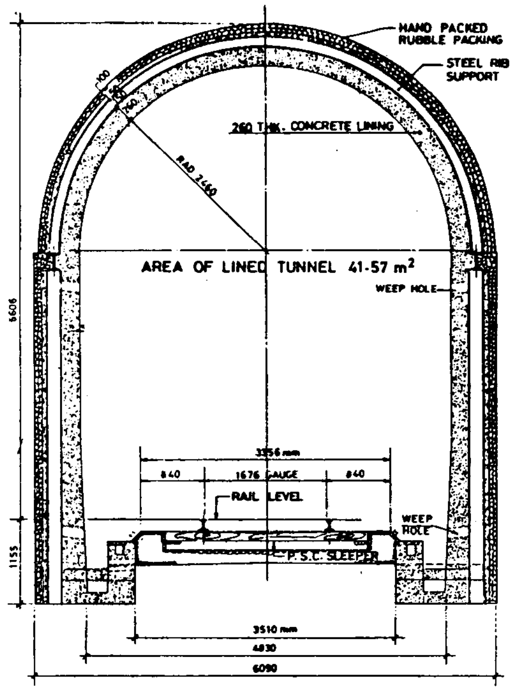


FIGURE 16 Cross Section of Lelligumma Tunnel

The Upline tunnel had a length of 128 m and the Downline tunnel had a length of 126 m (Fig. 18).

Methodology of Advancement in Soft Ground

In view of thickly populated area, space restrictions and soft soil conditions, it was decided to use a blade shield for supporting surrounding soil, manual excavation and placement of concrete lining for permanent support of this tunnel. This was the first time in India, a shield tunnelling unit, was indigenously manufactured and successfully operated by Indian engineers.

The shield unit used for this work was a cylindrical body with outside diameter of 5700 mm. It consisted number of trapezoidal shaped blades which were connected to the rear side frame of the shield with the help of hydraulic jacks (Fig. 19, 20, 21).

FIGURE 17 Plan of Calcutta Metro Showing Location of AI Underpass

Blades consisted of mainly two types viz.

a. Main blades and
b. Tail blades

The sequence of operation was as follows

a. First an entrance pit was excavated upto the required depth and shield unit was erected/assembled in the entrance pit.

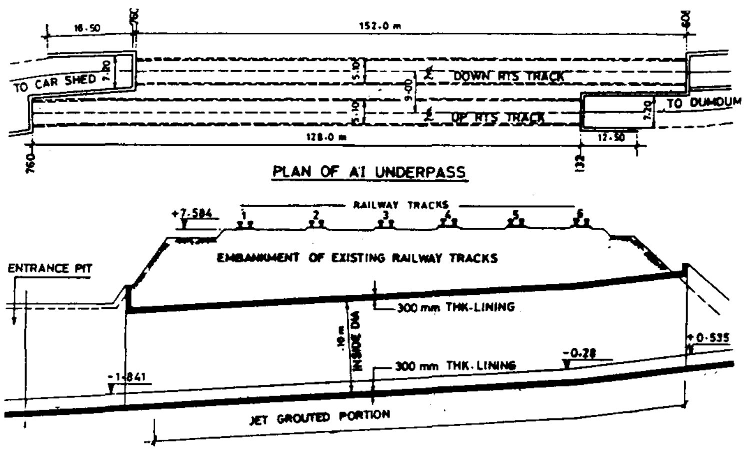


FIGURE 18 Longitudinal Section of AI - Underpass showing Treatment of Jet Grouting below Base of Tunnel prior to Construction

b. When the entire assembly was ready, it was touched to the face wall, with main blades in a position to enter in the face to be excavated.

c. An individual blade was pushed into the soil by operating the hydraulic jack.

d. By operating all the jacks, all the blades were pushed in the soil, upto a predetermined advancement of about 600 mm.

e. Then soil over the entire cross-section area of tunnel face, enclosed between the blades of shield unit was excavated upto the tip of blades.

f. All the hydraulic jacks were then closed simultaneously, so that the shield could advance forward bodily, over the distance in which soil had been excavated from the tunnel face and removed.

g. The process was repeated till the shield moved forward by 2 to 2.4 m. In case, soil in tunnel face was too soft or very loose, face boards were provided to check the soil flow into shield. The face boards were wooden planks, held in position with the help of hydraulic jacks fixed on shield frame. The excavation face was opened in a suitable sequence to carry out the excavation.

h. The reinforcement was then tied within the tail blade portion. A collapsible shutter of suitable size was erected within the tail blade portion. Concrete was placed with the help of concrete pump. Shutter

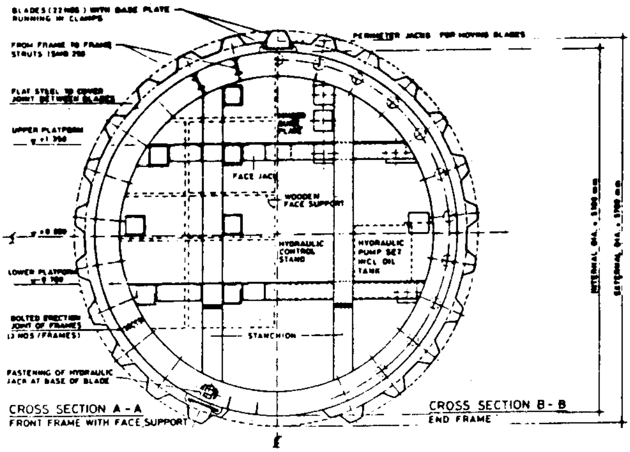


FIGURE 19 Cross Section of Blade Shield Unit at AI Underpass at Calcutta

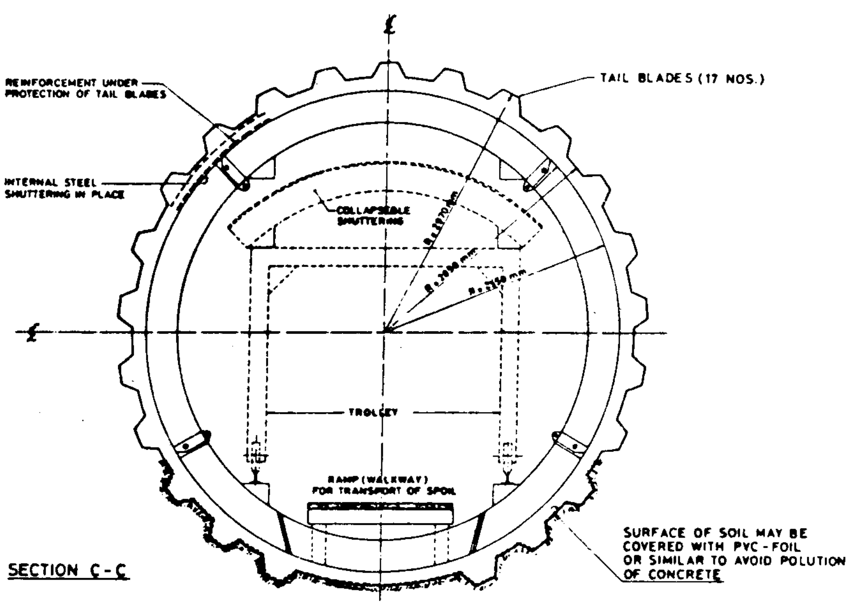


FIGURE 20 Cross Section of Blade Shield Unit at AI Underpass at Calcutta

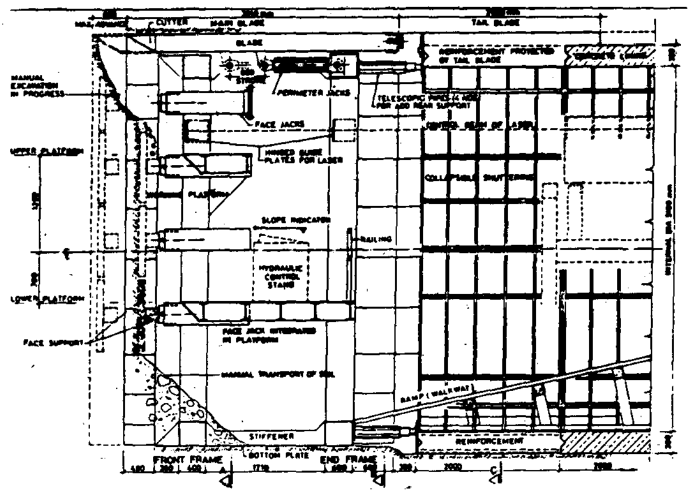


FIGURE 21 Longitudinal Section of Blade Shield Unit at AI Underpass at Calcutta

vibrators were used for vibrating the concrete, while it was being placed. After specified period, collapsible shuttering was removed and taken to next position. Once the concreting was over, the whole series of operations of excavation, shield shifting, concreting etc. were repeated and this cycle of operations was continued. Fig. 19 shows section of main blade of shield unit. Fig. 20 shows section of tail blade of shield unit. Fig. 21 shows Longitudinal section of blade shield unit. Fig. 22 shows sequence of operation of tunnel construction.

Problems Encountered

After construction of 30 m length of Upline tunnel, construction progress was hampered on account of very soft nature of strata. Due to soft clayey silt, the shield started sinking in soft ground underneath and could not be manoeuvered. At this stage, the shield nosc dived by 87 cm and it was very much in excess of specified tolerance limit of 50 cm (Fig. 23). Further construction of tunnel was then suspended.

It was decided, to use jet grouting technique to stabilise ground conditions, by strengthening the soft clayey silt beneath the base of tunnel, by mixing it with cement slurry introduced by jet grouting.

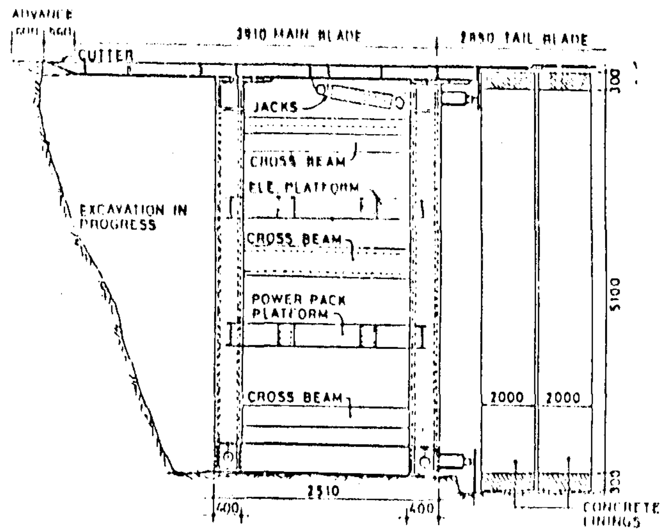


FIGURE 22 Railway Underpass near Dum-Dum Station Section AI by use of Blade Shield Tunnelling Excavation of Tunnel by using Tunnelling Shield

Remedial Measures

Initially, it was decided to stabilise the portion to be excavated, by jet grouting 'ahead of face', through the completed tunnel itself (Fig. 24). After doing a portion of 2 m by this methodology, this method was abandoned. This was on account of very slow progress, as, while jet grouting was being done, no other activity could be carried out inside the tunnel. Fig. 25 shows details of jet grout columns formed.

Alternatively, jet grouting was carried out from the outside of tunnel. For this purpose, jet grouting rig was kept on the railway embankment and inclined holes were drilled upto required depth. After completion of drilling, only 2 to 3 m portion of borehole, beneath the tunnel was jet grouted.

In plan, spacing between centres of adjacent drill holes was kept between 50 to 60 cms. Diameter of each jet grout column was of order of 1.5 m - 2.0 m. By orienting the drilling direction properly, the jet grout columns were so formed that, they were more or less touching each other and formed a composite envelope beneath the base of the tunnel. Jet grouting was done using single phase process i.e. water cement slurry only. Cement consumption varied between 300 - 400 kg/m of jet grout column. After

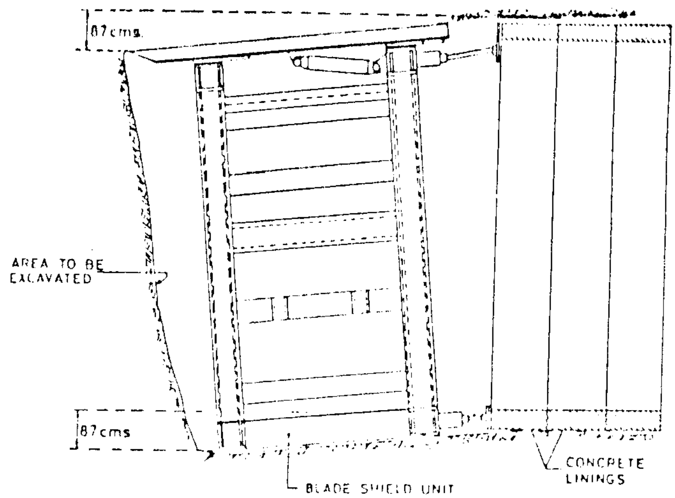


FIGURE 23 Nose Diving Tendency of Tunnelling Shield at AI Underpass, Calcutta

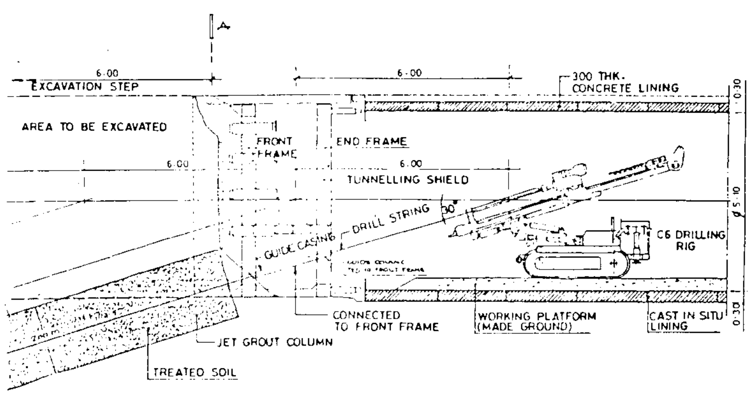


FIGURE 24 Longitudinal Section, Railway Underpass near Dum-Dum Station, Sub-Horizontal Treatment by Jet Grouting

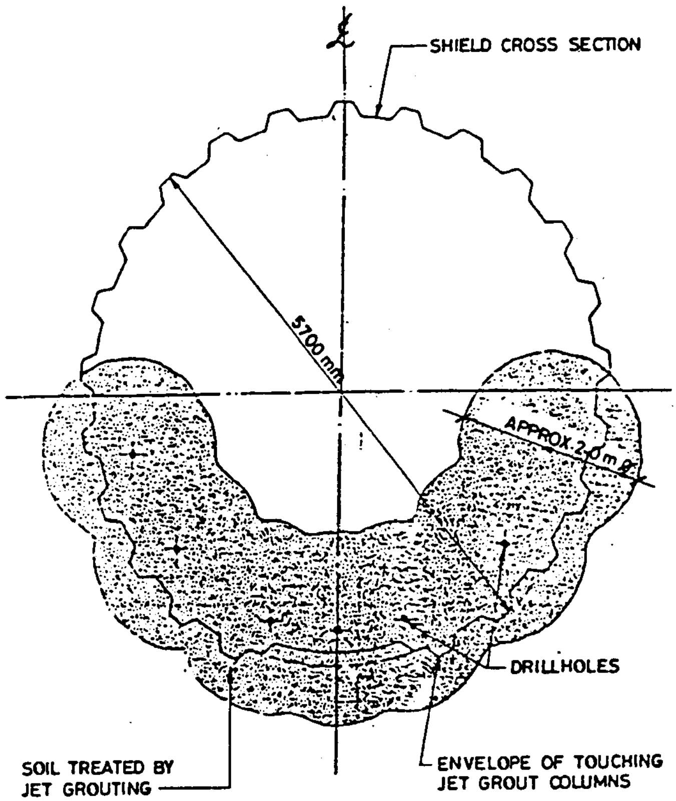


FIGURE 25 Cross Section of Sub-Horizontal Treatment by Jet Grouting at AI Underpass, Calcutta

treating a length of 8-10 m, excavation in that portion, was carried out using blade shield unit.

By providing jet grouting beneath the base, the nose diving tendency was totally restricted and tunnel shield could be manoeuvered in correct alignment. Fig. 26 shows schematic arrangement foı carrying out jet grouting. Fig. 27 shows jet grouting rig in operation at site. Fig. 28 shows excavation in progress in tunnel using blade shield unit. Fig. 29 shows view of partially completed tunnel indicating lining and excavation in progress.

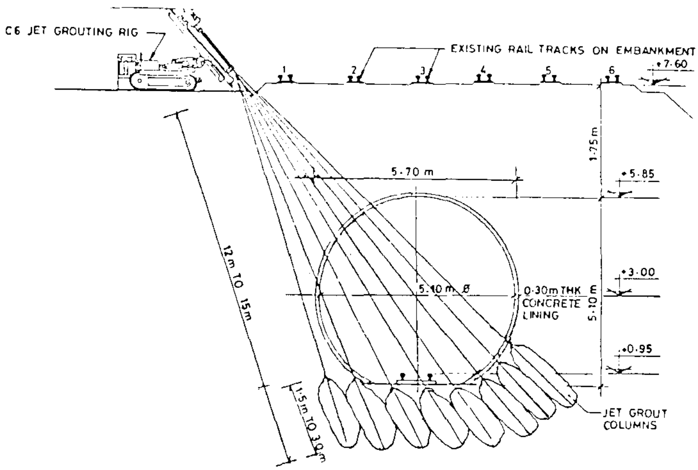


FIGURE 26 Cross Section of AI Underpass showing Treatment of Jet Grouting below Base of Tunnel prior to Construction

The work of jet grouting treatment, was completed in a period of 3 months. The work of construction of both the tunnels and other related supporting structures, was completed in a period of 24 months. These tunnels have been commissioned and are in operation since 1991.

Tunneling in Soft Soil to Rock Using Shield

Konkan Railway Corporation Limited (KRCL), has been entrusted with the responsibility of construction of a single track, broad gauge, railway line running through Konkan Region. This line upon completion, will connect Roha at northern end to Mangalore at southern end.

The work on this challenging task, was commenced in October'90 and is expected to be completed by October'95, in a time span of 5 years. This line when completed, will provide the much needed North South Route along West Coast, connecting Bombay with Kanyakumari. The terrain across this railway line alignment, features hill ranges emanating from Sahyadri mountains and reaching upto Arabian sea.

These hill ranges are interspaced by rivers and deep valleys. On account of this peculiar geographical feature, there are 75 tunnels with a

FIGURE 27 Jet Grouting Rig in Operation at Site of AI Underpass

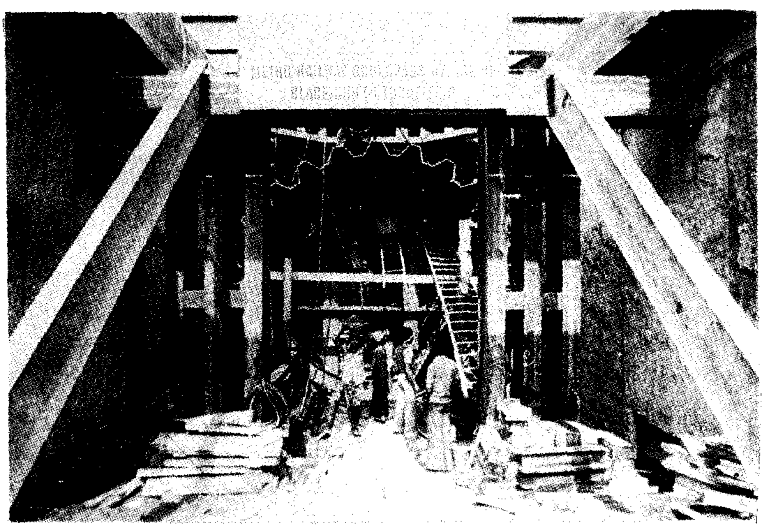


FIGURE 28 Excavation in progress using Blade Shield Unit

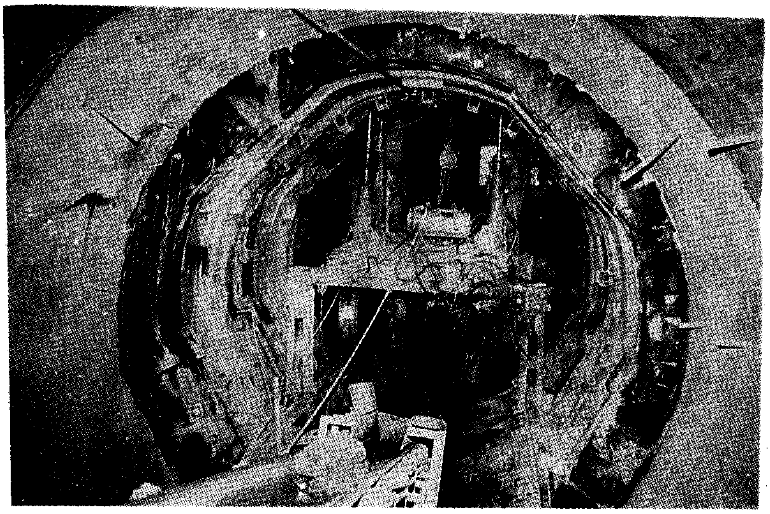


FIGURE 29 View of Partially completed Tunnel

total length of 78 kms. The longest tunnel is 6.5 km long. This tunnel being constructed at Karbude near Ratnagiri, will be the longest tunnel, in India on completion.

Location

AFCONS is carrying out the construction of Honavar tunnel (Tunnel 5 and 5A) (Fig. 30). This tunnel is situated near the town of Honavar, in Karwar district of Karnataka state and derives its name from the same.

As the tunnel is passing through completely to highly weathered laterite, 'soft strata', it was considered necessary to adopt shield tunneling technique.

This tunnel is circular in section with internal diameter of 7.2 m and external diameter of 7.8 m. Thus, internal cross-sectional area of tunnel is 40.715 m^2.

Construction Technique

A blade shield, comprising of 29 blades has been indigenously manufactured for this purpose. Each blade is connected to a 60 tonne hydraulic jack, with a travel of 100 cms (Fig. 31).

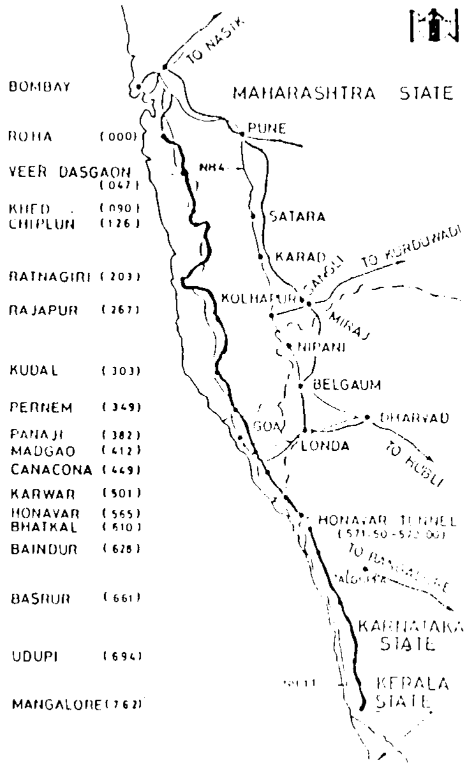


FIGURE 30 Location of Honavar Tunnel in Konkan Railway under Construction

The shield unit, provides protection to material lying over the crown from collapse and also makes the excavation process more safe. The nature of strata encountered along this tunnel varied from soft soil (flowing slurry in rainy seasons) to hard rock, and but for the shield, numerous accidents (fatal) would have occurred. Fig. 32 shows formation of cavities above crown of tunnel during construction.

Because of nature of strata encountered, excavation was carried out manually or using mechanical excavators or by controlled blasting in rock. Excavated material was removed by using dumpers. Fig. 33 shows removal of excavated material using dumper in Honavar tunnel.

This tunnel is fully lined with 300 mm thick concrete lining. Lining is being provided with 12 Nos. of precast RCC segments. Fig. 34 shows cross section of tunnel and liner segments. Each segment is 300 mm thick,

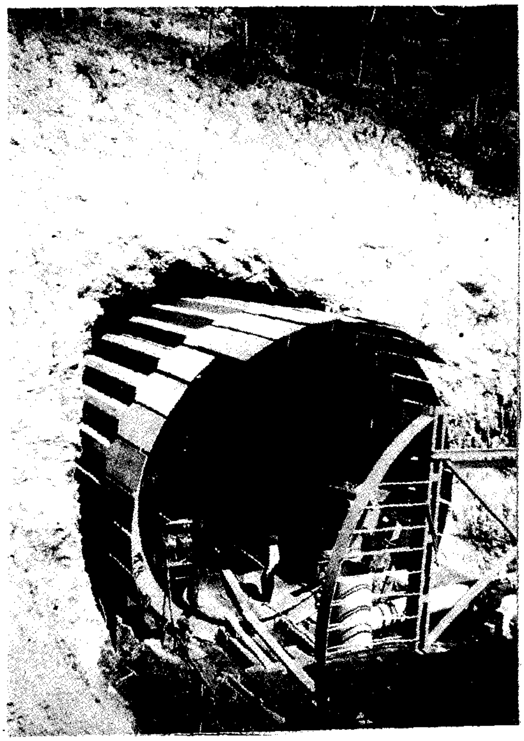


FIGURE 31 View of Blade Shield Unit at the enterence of Honavar Tunnel

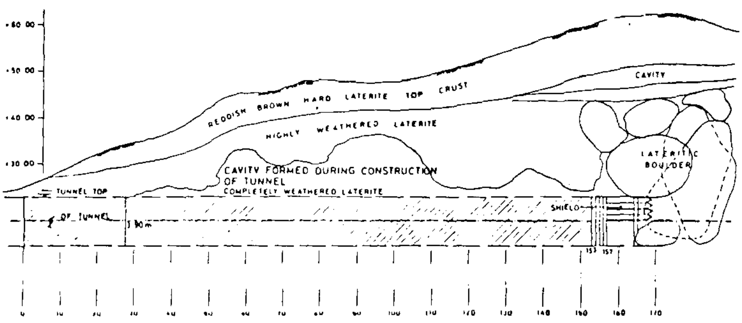


FIGURE 32 LongitudinalSection of Honavar Tunnel 5A showing Cavities encountered during Tunnel Construction

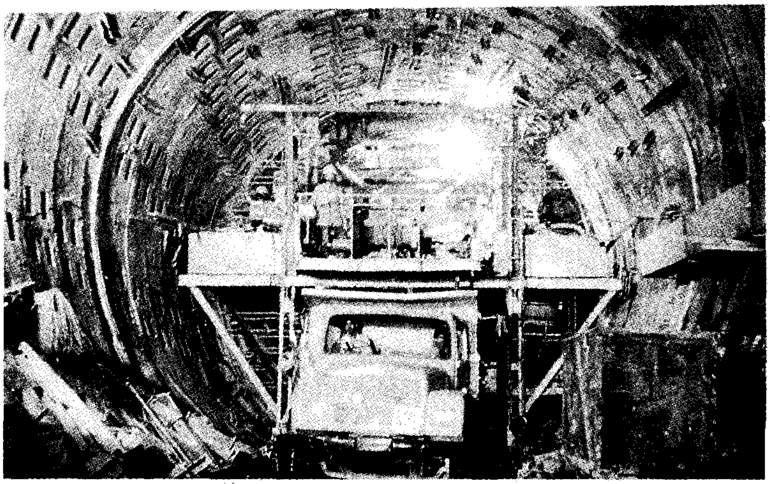


FIGURE 33 Removal of excavated material using Dumper in Honavar Tunnel

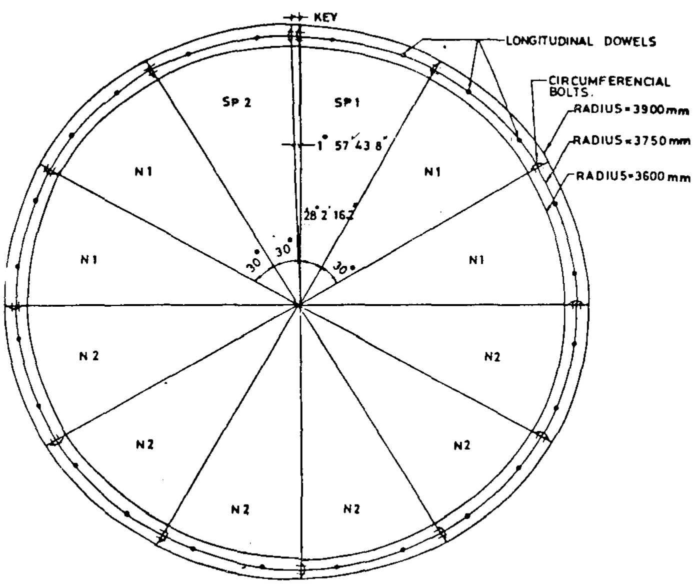


FIGURE 34 Cross Section of Honavar Tunnel showing arrangement of RCC Liner Elements

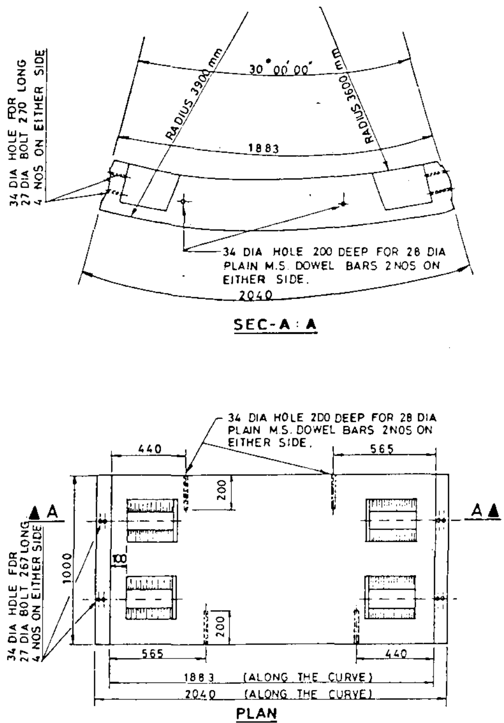


FIGURE 35 Details of RCC Liner Elements of Honavar Tunnel

1000 mm wide and 1885 mm long (inner curved length) and weighs about 1.5 tonnes. Fig. 35 shows some details of liner segments.

These are cast using accurate forms and M-50 grade concrete. Curing is done initially, for a period of 16-18 hours by steam curing and subsequently by wetting for 28 days. Fig. 36 shows reinforcement details of liner segments. Fig. 37 shows stacks of liner segments ready for erection.

Each segment weighs about 1.5 tonnes. Lining elements which are ready for erection, are brought on a trolley running on wheels, inside the tunnel upto desired place.

At appropriate place, these are lifted and erected by hydraulic manipulator/placer, which travels behind the shield. This hydraulic placer was designed and fabricated by AFCONS (Fig. 38).

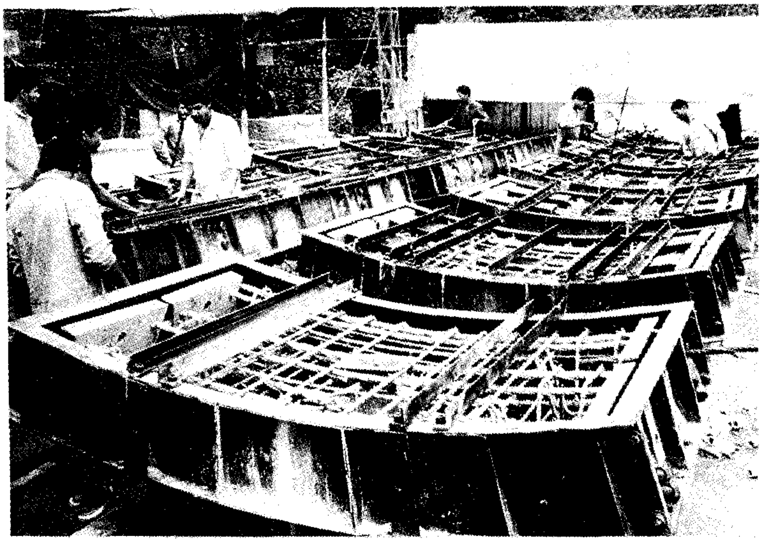


FIGURE 36 Details of Reinforcement in Liner Segment

FIGURE 37 Stacks of Liner Segments ready for Placement

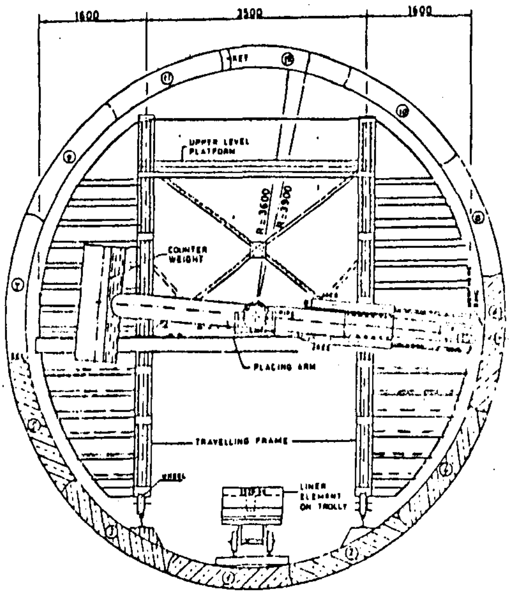


FIGURE 38 Cross Section of Placer for Erection of Liner Elements in Honavar Tunnel

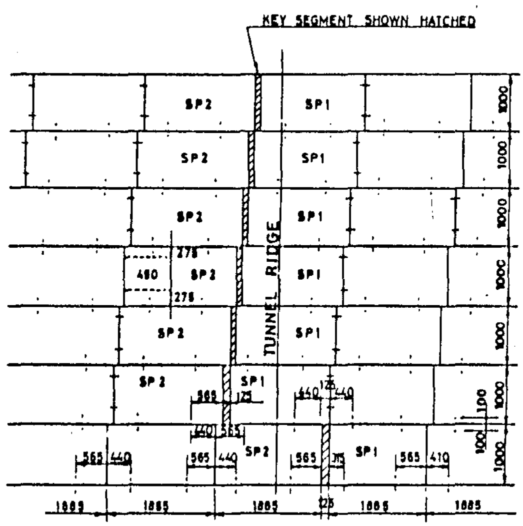


FIGURE 39 Typical Layout of SP1, SP2 type Liner Elements and Key Segments in Honavar Tunnel

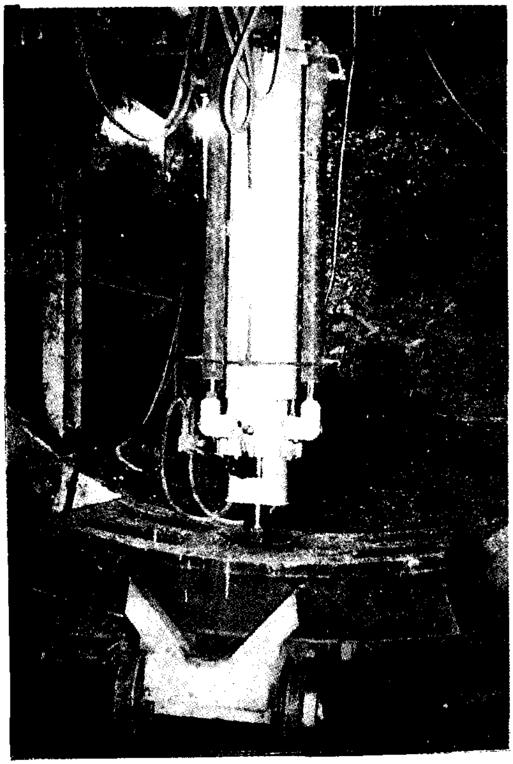


FIGURE 40 Transportation of Liner Element

Adjacent segments are connected by 4 bolts at each end. Longitudinally the segments, are connected by dowel bars. After erection of these concrete segments, surrounding soil is made impermeable by carrying out primary and secondary grouting, using cement slurry. Fig. 39 shows details of staggering of key elements in lining of Honavar tunnel. Fig. 40 shows transport of liner element. Fig. 41 shows erection of liner element using placer.

By 30th September'94, 160 m tunnel (Tunnel 5A) has been completed. Entire work is expected to be completed by March'95.

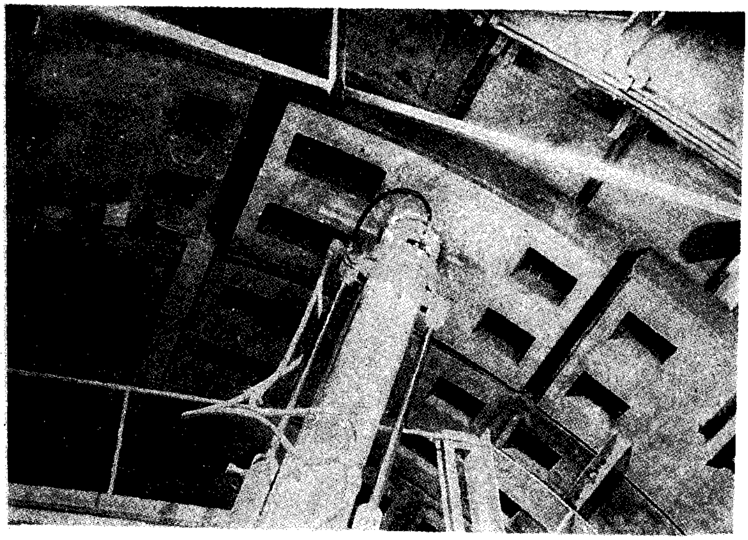


FIGURE 41 Erection of Liner Element using Placer

Developments in Bridge Construction Techniques

Construction of Thane Creek Railway Bridge

The city of Bombay has developed in a more or less 'ribbon pattern' in North-South direction. Government of Maharashtra, decided to develop the main land across Thane Creek in 1970s. For this purpose, City and Industrial Development Corporation (CIDCO) was formed in 1970. The main bottleneck in developments of New Bombay area, lying on the Eastern side of Thane Creek, was lack of communication links with island city of Bombay. To provide for better communication links, a 4 lane road bridge was constructed in 1967-72. As, it was decided to connect Bombay and New Bombay by a railway line, Metropolitan Transport Project (MTP) Railways, planned to extend the existing railway line at Mankhurd upto Belapur, by constructing a 17.96 km long new railway line, in 1985.

This necessitated construction of a 1872.5 m long bridge across Thane Creek. AFCONS was awarded this work in September 1986. AFCONS started the work in January 1987 and the construction of the bridge was completed by June 1991.

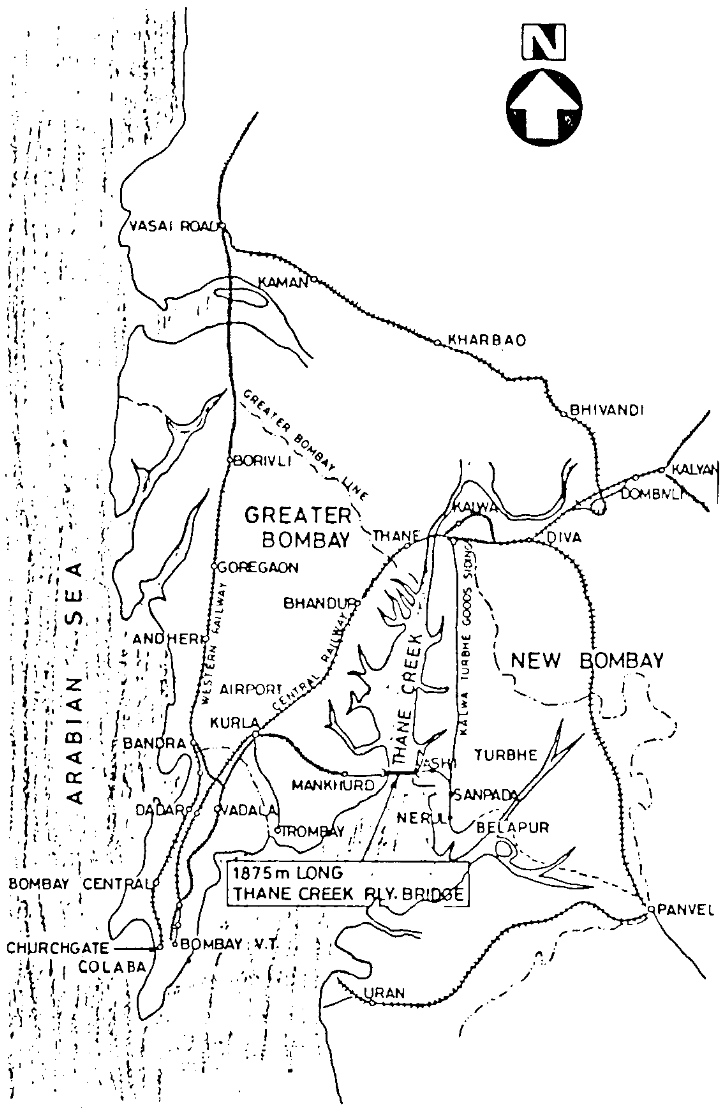


FIGURE 42 Location of Thane Creek Railway Bridge

Main Features

The construction of this 1872.5 m long bridge (Fig. 42), involved construction of approaches over soft marine clay on either side and construction of main bridge itself. The bridge has 35 spans, with each span having a length of 53.5 m. For providing foundations to 31 piers, well foundations were constructed using floating pneumatic caisson sinking method.

The work of ground improvement on Mankhurd side was done by AFCONS by constructing 300 mm diameter sand drains.

Ground Improvement

At Mankhurd end approach, the ground improvement was carried out by construction of sand piles. To prevent settlement of railway embankment, constructed on soft marine clay deposits, sand piles or sand drains were constructed in equilateral triangle pattern, at a centre to centre spacing of 2 m. Sand drains had a diameter of 250 mm and average depth of sand drains varied between 6.5 to 20.5 m. Sand drains were filled up with 40% crushed stone, 55% coarse sand and 5% commercial lime. At Vashi end of the abutment, where creek water extends for a considerable length behind the abutment, the marine clay was dredged out and good granular soil was filled up, upto the formation level.

Caisson Construction

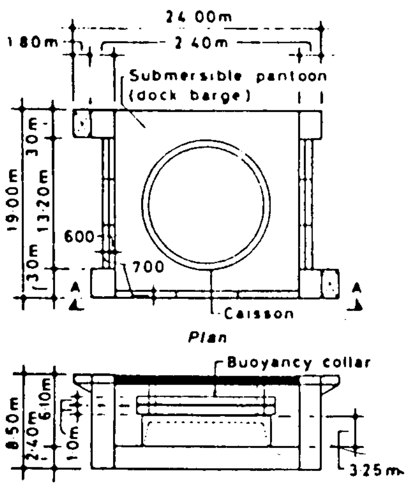
For construction of well foundations, instead of conventional method of well sinking, an innovative approach was adopted. This new approach is described briefly below (Fig. 43).

i. Two submersible barges of 24 m × 19.2 m × 8.5 m were fabricated at site. The submersible barge was provided with 14 Nos. of ballast tanks.

ii. A reinforced concrete pressure chamber of 11.m diameter and 3.55 m height and a circular well steining of 9.5 m diameter, 3.60 m height and 0.6 m wall thickness, were cast on the submersible barge near the jetty using concrete of M-40 grade.

   Two sets of buoyancy collars of 1.5 m × 1.0 m cross section were fitted with the well steining, to provide additional buoyancy and stability to the caisson in floating condition.

iii. The submersible barge with the fabricated caisson of 7.15 m height on it, was towed during high tide, to a location where approximately 8.5

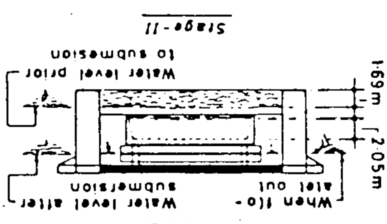


Section A-A

Stage - I

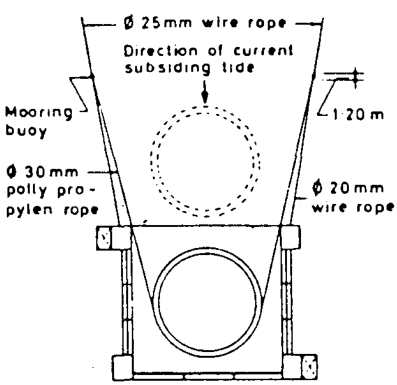
Stage I

Prefabricating lower part of the caisson on submersible pantoon up to a height of 6.10m with the help of land borne equipment. Two sets of buoyancy collars of 1.5 m x 1.0m are attached.



Stage II

Floating of caisson is achieved by submersing the pantoon, approximately 5.5-m deep from deck level, and providing compressed air in the working chamber to keep 2.05-m doup portion dry.

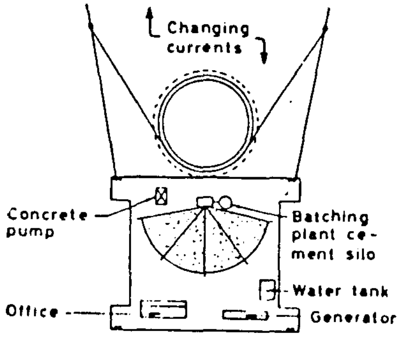


Stage III

Caisson in the floating position and is being towed.

Stage - III

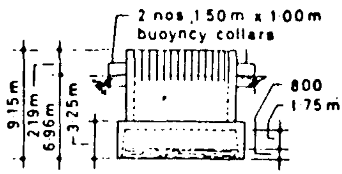
FIGURE 43 Construction Stages of Caisson Foundation



Stage IV

Stage IV

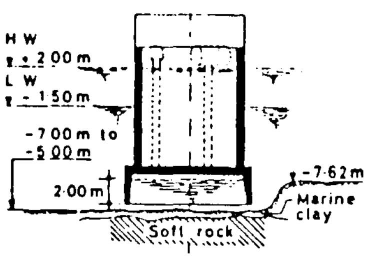
Raising of caisson 7-m per lift, keeping in mind that fresh concrete should not get in touch with the sea water within a period of 72 hours after concrating. Formwork is erected by floating crane. Concreting is done by pumping from floating batching plant.



Stage-V

Stage V

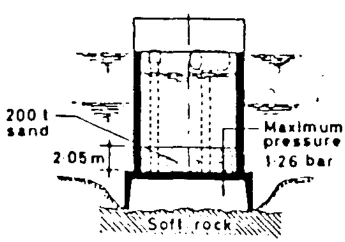
After pouring first lift, the caisson will have a freeboard of 2.19m.



Stage-VI

Stage VI

- Bed level at final location is lowered by dredging to – 9.00m.
- Caisson roughly located in final position.
- Air locks for personnel and for removal of excavated rock are installed.
- Caisson held in approximata position by four anchors, two for rising water and 2 for subsiding water. Anchor capacity = 20t (each).

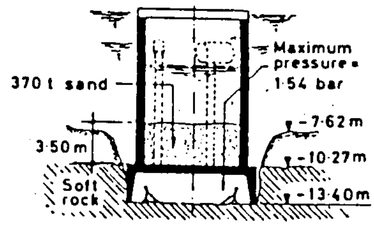


Stage-VII

Stage VII

- Caisson positioned accurately.
- Ballast of 200t provided (sand gravel quarry run).
- Marine clay formed into slurry by high pressura water jats and pumped out with 'Toyo' pump DB 50 B. This operation is performed by personnel working in the chamber undar a maximum pressura of 1.26 bar.
- Caisson gradually lowered on to the murrum/weathered rock formation.
- Level of the caisson and its position is checked, and if required adjusted.

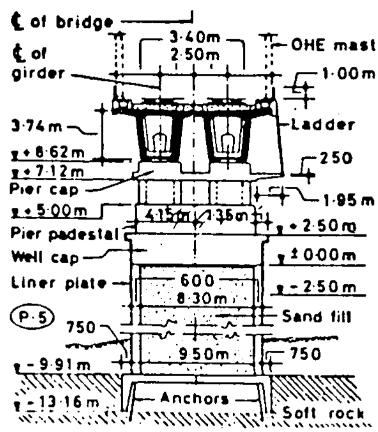
FIGURE 43 Contd.



Stage-VIII

Stage VIII

- Commencing excavation in murum/weathered rock by mannually operated pneumatic tools.
- Excavated material filled into a special bucket which is hoisted through the material shaft and lock.
- Suitable material is used to supplement the ballast fill.
- At any time ballast provides a minimum downward load of 70t in excess of the uplift force.
- When reaching the predetermined foundation level of - 13.40 m the ballast shall not be less than 370t.
- To facilitate handling of the caisson and the sinking operation ballast may be increased 10 percent at any intermediate stage.



Stage-IX

Stage IX

- When final foundation level is reached, checking plumb level and position of caisson and if required, adjusting the same.
- Drill and grout rock anchors.
- Filling working chamber with plain concrete of grade M25 by means of a pump under pressure to make sure that the chamber is filled to its ceiling.
- Removing the material/personnel from shaft and locks.
- Supplement internal fill with sand-gravel or quarry-run up to a height of 0.15 m under the well cap.
- Concreting well cap and constructing the pier.

FIGURE 43 Contd.

m water depth was available. With the help of pumps and valves, water was admitted in a predetermined sequence in the 14 tanks of submersible barge, to gradually sink it. As the barge sank by 8.1 m, the caisson would float by itself.

iv. Afterwards, with the help of 2 tugs, the floating caisson was pulled out of the barge and towed to its final position. Once, the caisson was taken away, the submersible barge was refloated by filling the ballast tanks with compressed air. As the barge came back to its original position, it was brought to shore.

v. After finalising the exact position of caisson, ballast was filled inside

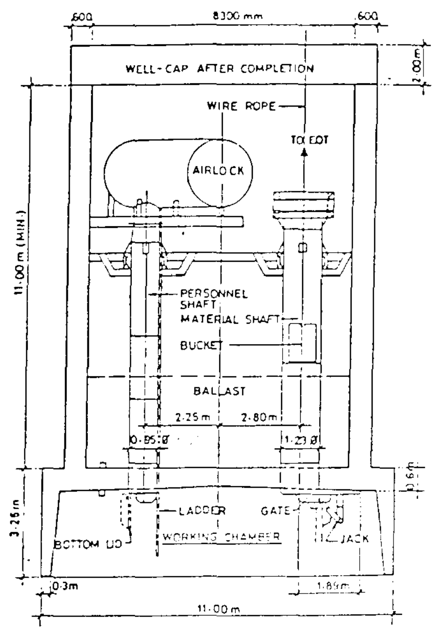


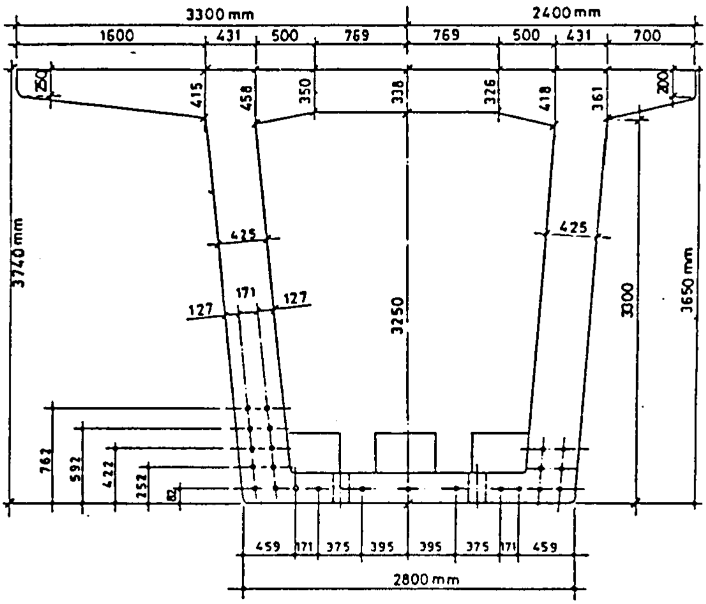
FIGURE 44 Details of Pneumatic Sinking Arrangement for Thane Creek • Railwat Bridge

the caisson and the caisson was sunk. The height of the steining was increased, by carrying out additional concreting in stages, in the floating condition of caisson. For this purpose, climbing form work was used. While raising the height of caisson, it was ensured that, fresh concrete did not come in contact with sea water for a period of 72 hours. In this manner, the height was subsequently increased, till the caisson rested on creek bed on marine clay.

vi. At this stage, man and material locks were attached (Fig. 44). Caisson was held in position by installing four anchors of 10 tonne capacity each and with the help of winch pontoons connected to caisson through hook provided in bulk head.

vii. Soft marine clay in the working chamber was pushed out by compressed

CROSS SECTION OF THE BOX GIRDER AT THE CENTRE OF THE SPAN



LENGTH OF BOX GIRDER	53.40	m
DEPTH	3.65	m
REINFORCEMENT STEEL	42.00	T
PRESTRESSING STEEL	16.00	T
VOLUME OF CONCRETE	300	m^3
WEIGHT OF GIRDER	800	T

FIGURE 45 Details of Prestressed Box Girder for Thane Creek Railway Bridge

air and firm marine clay was removed, by converting it into a slurry by high pressure water jets and pumped out with 50 DB Toyo Pump. Caisson was gradually lowered on the weathered rock formation. The work in the working chamber was done under a maximum pressure of 2.0 kg/cm^2 following systematically laid down safety procedures for under water working.

viii. Excavation in weathered rock was carried out, using hand held pneumatic drills/breakers. Excavated material was taken through the material shaft and lock, out of the caisson, using a special bucket.

ix. After completing the excavation upto the required foundation level, the rock stratum was inspected, visually in dry condition. The caisson was founded on the rock stratum, which had a safe bearing capacity of 200 t/m^2 with a factor of safety of six. The surface of rock was cleared off thoroughly, removing all the loose muck. Bottom plug concrete was done with M-20 grade concrete using concrete pumps, in absolutely dry conditions. After completing this, material/personnel shafts were removed and locks etc. removed from shaft. Balance ballast fill was poured in the well. Subsequently, top plug was concreted.

Fig. 44 shows details of pneumatic sinking arrangement.

Superstructure

The bridge has been provided with 2 numbers of precast prestressed box girders in each span, one for the down track and the other for the up track. Each box girder had a length of 53.4 m, depth of 3.65 m and width as 5.7 m at top and 2.8 m at bottom. Each girder had the weight of approximately 800 tonnes.

Casting of Girders

Each girder was concreted in a single continuous operation, over a period of 18-22 hours with M45 concrete (Fig. 45). Two batching plants and

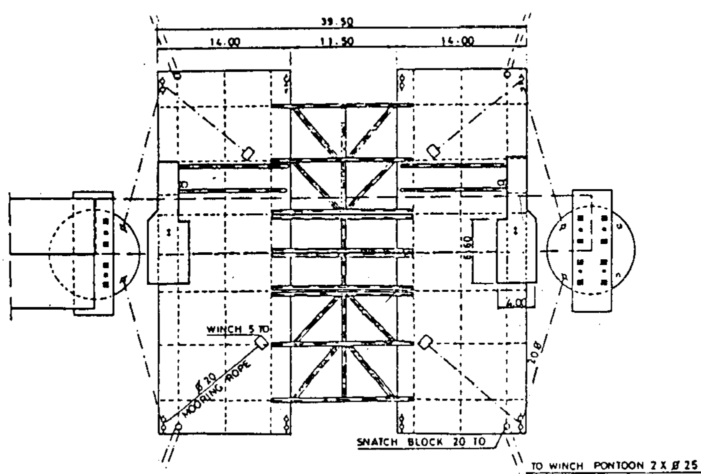


FIGURE 46(a) Plan of Floating Crane Equipment

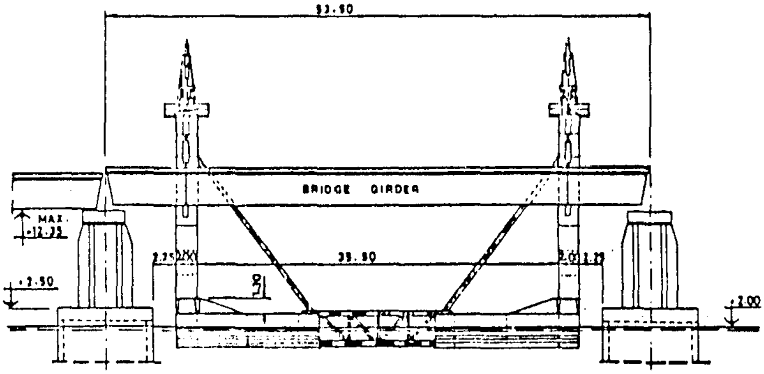


FIGURE 46(b) Elevation of Floating Crane Equipment

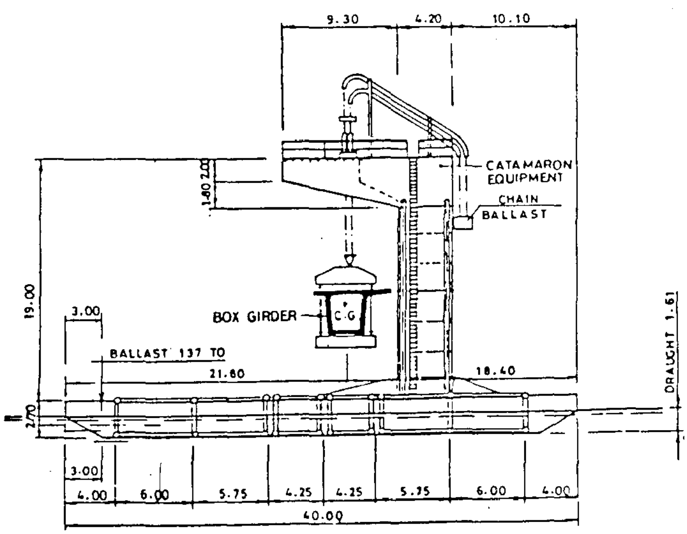


FIGURE 47 Lifting of Box Girder using Floating Crane

two concrete pumps were used for carrying out the concreting. The concrete sequence was worked out in such a way that, at no place, any cold joint could form in the concrete which was already placed.

Precast box girders were subsequently prestressed in three stages.

Launching of the Girders

The girder was brought from the casting bed to the location of launching jetty. For lifting of the girder, a special floating crane was manufactured (Fig. 46). The floating crane consisted of two barges of size 14 m × 40 m connected by lattice girders, making the latticed barges of size 39.5 m × 40 m. It was fitted with two 17.75 m high towers which had two cantilevers 9.2 m × 2.0 m. On the top of each tower, 4 hydraulic jacks of 125 tonnes capacity each were installed.

These jacks could lift/lower the girder at a rate of 300 mm/minute. This device is called as 'Catamaran'. This was the first time in India, that launching of 53.4 m long, 800 tonne girder was done using 'Catamaran' equipment. It was brought during the high tide near launching jetty. Temporary external prestressing of box girders was done to facilitate its lifting and placement.

In the next tide, the girder was lifted and 'Catamaran' was towed to the desired span location. Here, it was properly aligned with the help of four winches. The girder was lifted above the bearing level and Catamaran was taken in the span and positioned accurately. After aligning the girder, it was lowered on the bearings. Fig. 47 shows details of girder placement using floating crane.

As, this bridge is located in aggressive marine environment, special precautions were taken to ensure long-term durability of all the components.

The Bridge was inaugurated on 9th May 1992, by the President of India.

'Railway bridge across Thane Creek, 'Mankhurd Belapur Railway Project Bombay', was awarded the first prize for excellence, as "The most outstanding concrete structure in India - 1991" by American Concrete Institute. Maharashtra Chapter, on 8th August',92.

Indian Institution of Bridge Engineers awarded 'Railway Bridge Across Thane Creek', the first prize for excellence, of S.B. Joshi Memorial Award for the most outstanding Bridge (1992) on 12th February, 1994. Fig. 48 shows typical details of a complete pier. Fig. 49 shows casting of pressure chamber of caisson on submersible barge. Fig. 50 shows sinking of caisson. Fig. 51 shows raising height of steining of caisson. Fig. 52 shows details of girder casting. Fig. 53 shows girder being towed to desired location. Fig. 54 shows details of girder being placed on the pier location. Fig. 55 shows view of partially completed bridge. Fig. 56 shows view of completed bridge.

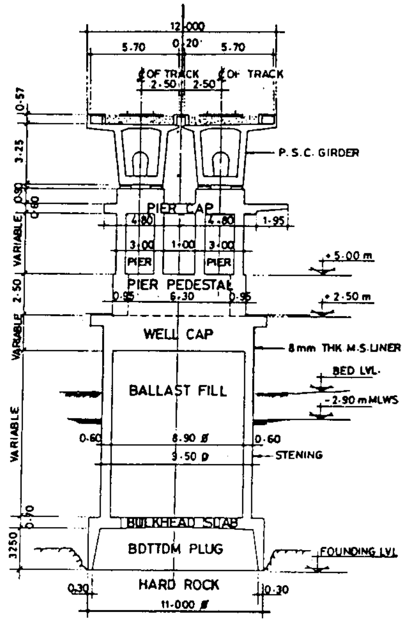


FIGURE 48 Typical Cross Section for Pier - Thane Creek Railway Bridge

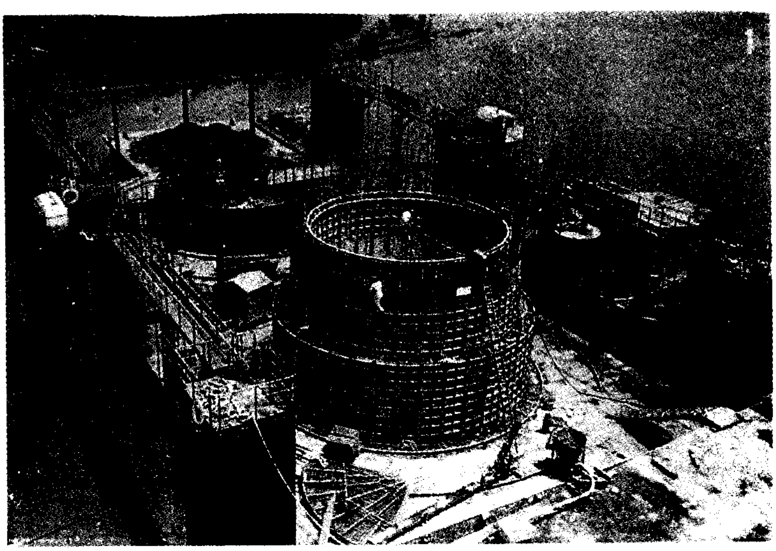


FIGURE 49 Casting of Pressure Chamber of Caisson on Submersible Barge

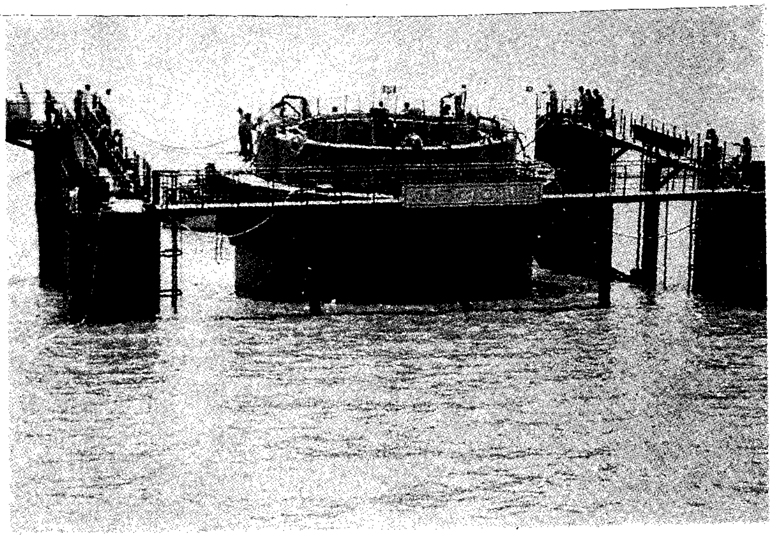


FIGURE 50 Sinking of Caisson

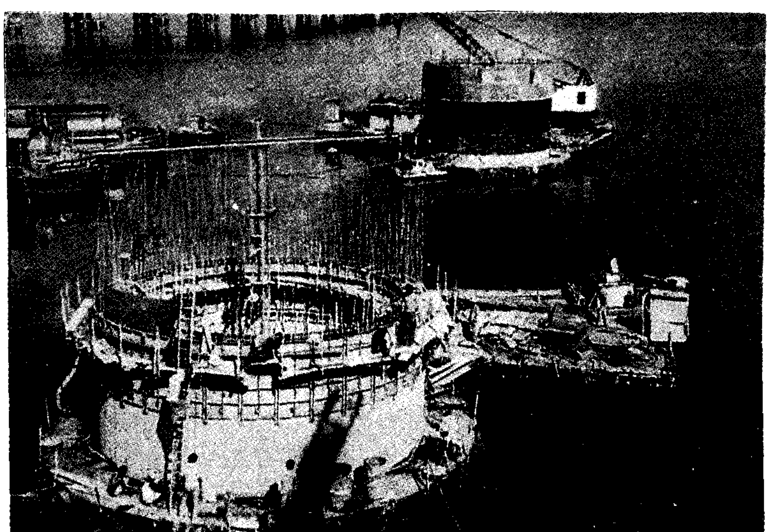


FIGURE 51 Raising Height of Steining of Caisson

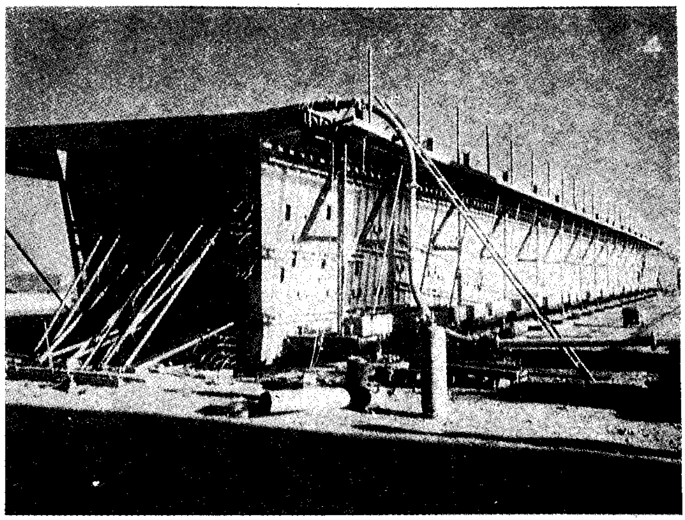


FIGURE 52 Casting of Box Girder

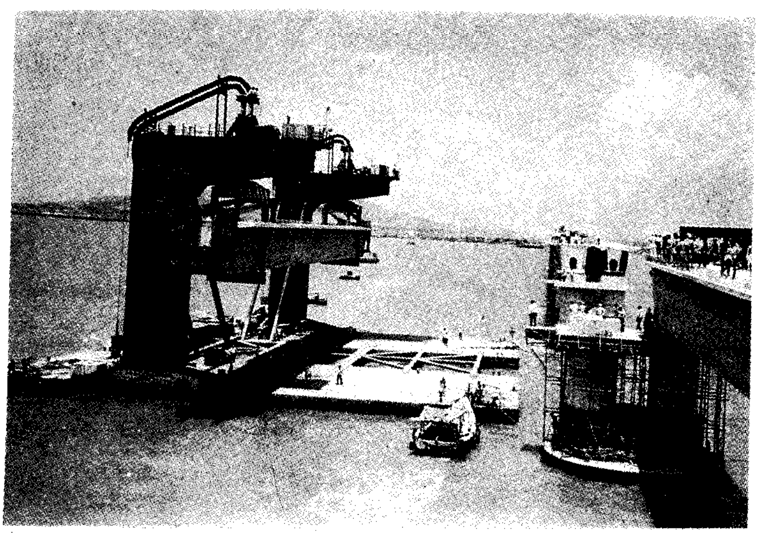


FIGURE 53 Towing of Girder to Location of Placement

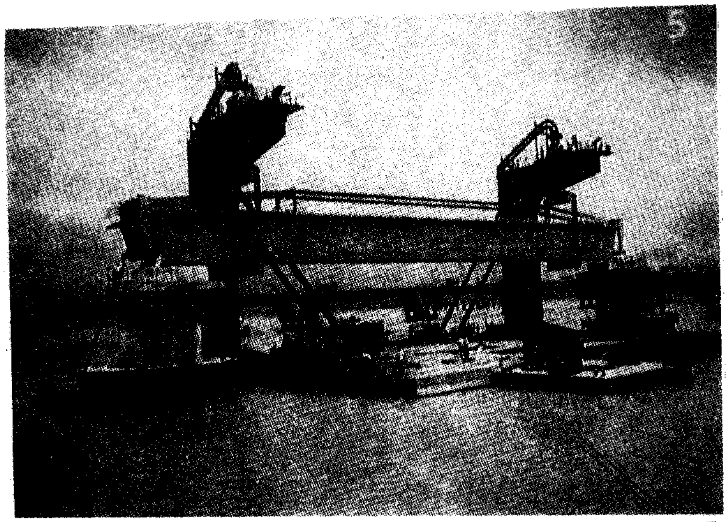


FIGURE 54 Placing of Girder at Pier Location

FIGURE 55 View of Partially Completed Bridge

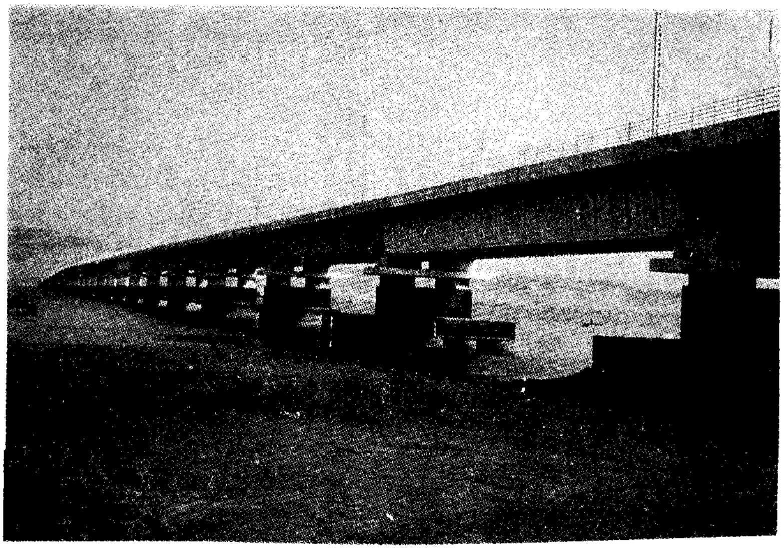


FIGURE 56 View of Completed Bridge

Construction Of Zuari Railway Bridge

The proposed Konkan Railway alignment crosses many small/medium/ big rivers in Maharashtra, Goa, Karnataka States. On the entire Konkan Railway route, 72 major bridges are being constructed. In Goa state, AFCONS is constructing three major bridges, two bridges on river Mandovi and one bridge on river Zuari.

Main Features

The construction of Zuari bridge involved construction of approaches over soft marine clay on either end and construction of main bridge. Fig. 57 shows part plan and elevation of Zuari bridge where caissons have been provided for foundation.

The work of ground improvement was done by AFCONS by constructing 300 mm diameter sand drains. Table 2 shows some of the important salient features of Zuari railway bridge. Fig. 58 shows longitudinal section of soil profile at Zuari bridge location. Fig. 59 shows section of typical pier on caisson W1 Fig. 60 shows section of typical pier on caisson W2.

TABLE 2
Salient Features of Zuari Railway Bridge

i)	Total length of bridge	987.7 m	
ii)	No. of spans	25	
iii)	Lengths of spans	124.2 m 53.5 m 22.8 m 22.15m	2 Nos. 7 Nos. 16 Nos. 2 Nos.
iv)	Type of substructure	One abutment and 13 piers A1, P1 to P13 : Pile foundations Two piers P14, P18 : Caissons W2 type Three piers P15, P16, P17 : Caisson on Pile foundations. One pier and one abutment P24, A2 : Open foundation	
v)	Piled foundations for a typical pier - P19	Pile diameter No.of Piles. Length of Piles	1.5 m 8 38.3 m
vi)	Length of piles	Minimum Maximum	38.0 m 47.0 m
vii)	Socketing in rock(weathered) for piles	Minimum Maximum	6.0 m 20.0 m
vii)	Well foundations	Caisson W1 Outer diameter Inner diameter Caisson W2 Outer diameter Inner diameter	3 Nos. 10.76m 9.56m 2 No. 8.0 m 6.8 m
ix)	Acceptance criteria for founding of caisson	Weathered rock with safe bearing capacity of 75-100 t/m^2 with factor of safety of 3	

Caisson construction

Like for Thane Creek railway bridge, for railway bridges on Zuari river and Mandovi channels, caissons were sunk using compressed air. But excavation of soil was carried out using water-jetting and Toyo pump. Water jetting was adopted to loosen the soil and then soil slurry was pumped out using Toyo pump. Excavation in weathered rock was carried out by using jack hammers and for removal of gravel, boulders etc. from caisson bucketing was adopted. For some of the caissons, these operations were guided on the basis of observation of C.C.T.V. (Closed Circuit Television).

Initially, it was felt that for some of the caissons, founding level may exceed 30 m below bed level and people working under compressed air

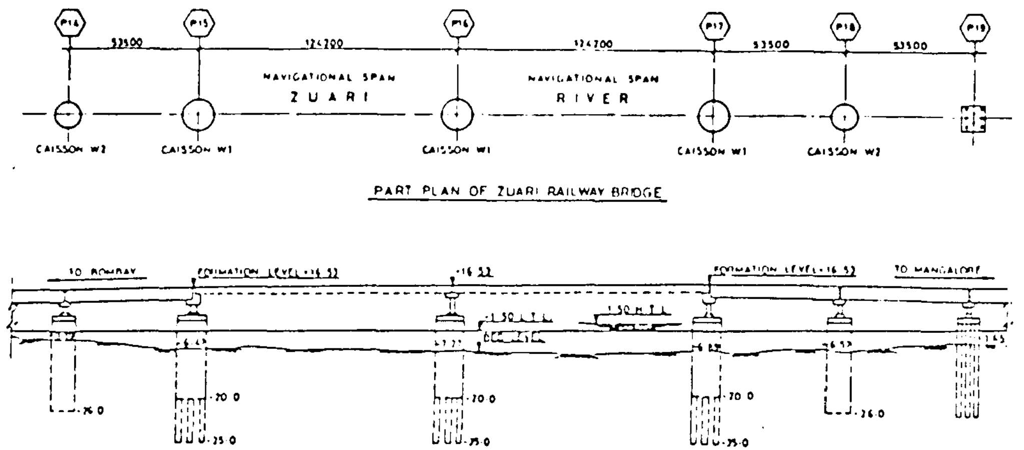


FIGURE 57 Part Elevation of Zuari Railway Bridge Showing Piers Founded on W1, W2 Type Caissons

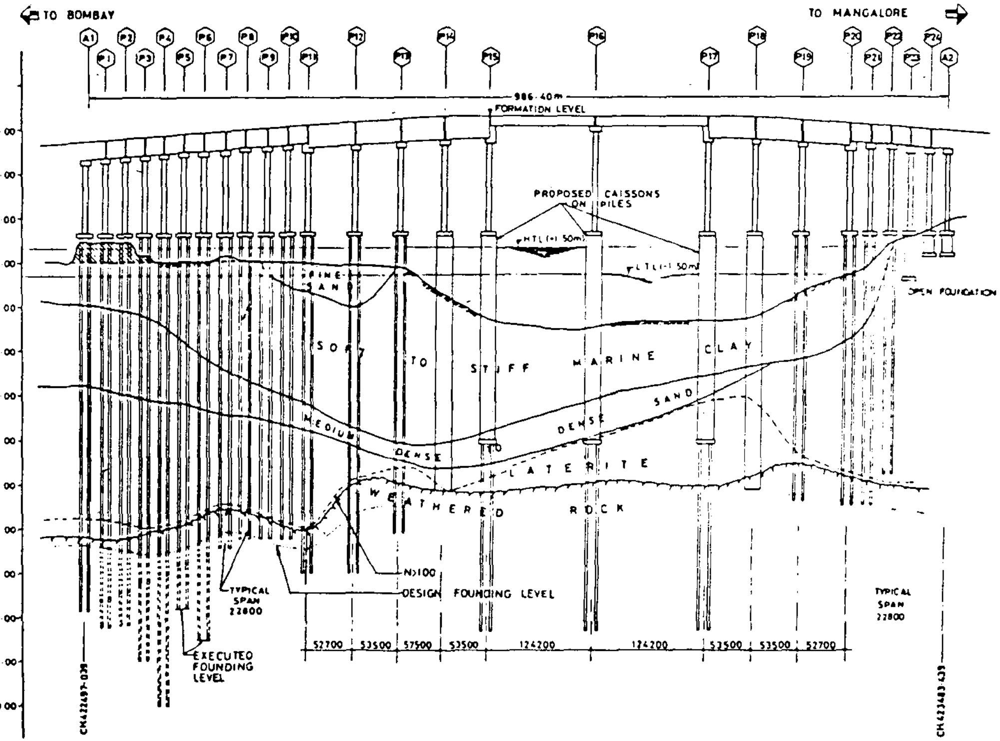


FIGURE 58 Sub-Surface Soil Profile at Zuari Bridge Alignment

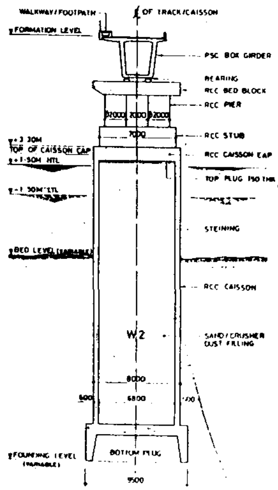


FIGURE 59 Typical Cross Section of W1 Caisson for P15, P16 and P17 at Zuari Bridge

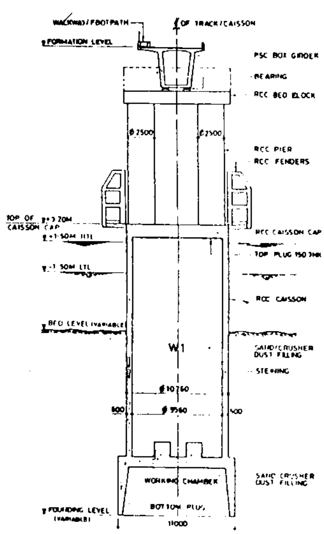


FIGURE 60 Typical Cross Section of W2 Caisson for P14, P18 at Zuari Bridge

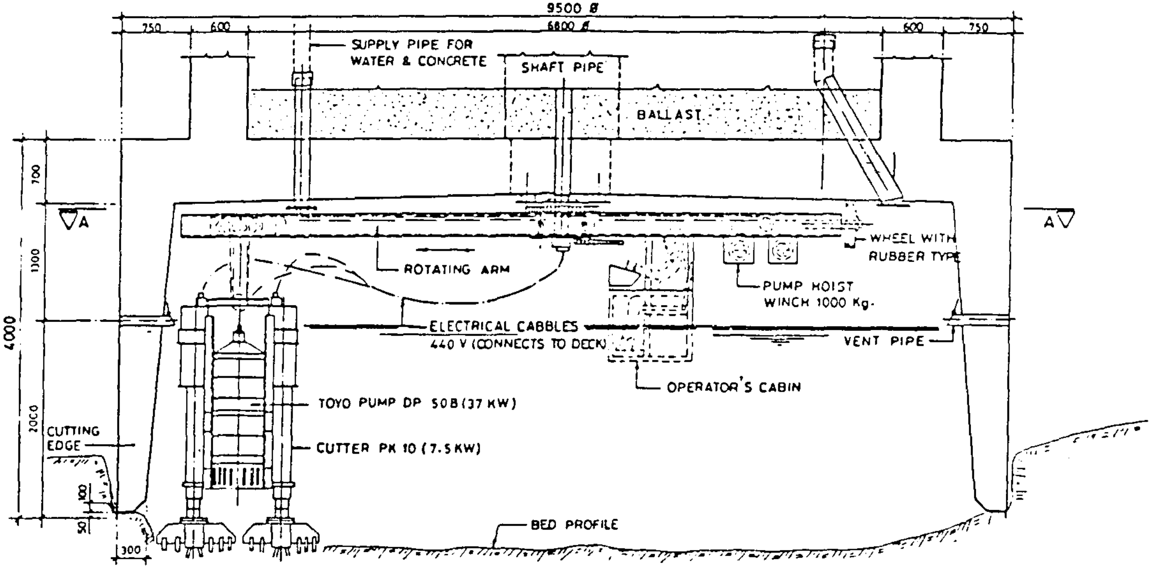


FIGURE 61 Cross Section of Working Chamber for W2 Type Caisson at P14, P18 on Zuari Railway Bridge

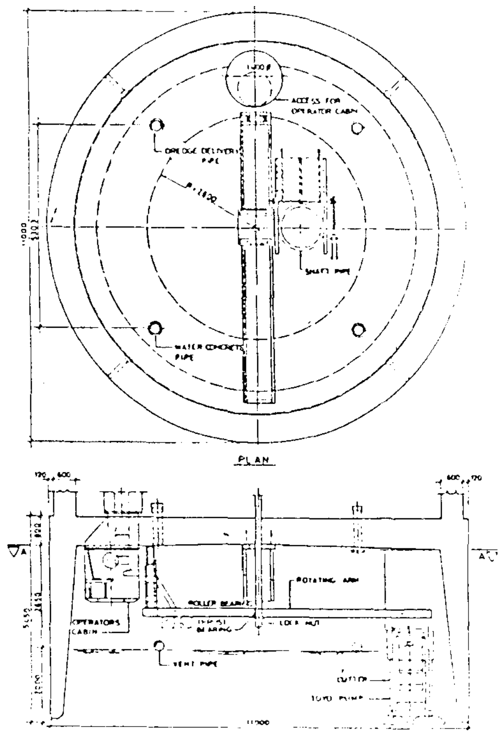


FIGURE 62 Details of W1 Type Caisson for P15, P16, P17 at Zuari

would have to work at pressure greater than 3 kg/cm^2. Working under such pressure would greatly reduce working hours. Hence, for such situations, it was initially planned that, Toyo pump operations will be carried out with operator working in special chamber subject to atmospheric pressure. But ultimately, such situation did not arise as either caissons were founded at shallower depths (because of bearing strata encountered at shallower depths) or because some of such caissons were made to rest on piled foundations (Fig. 67). Fig. 61 shows cross section of working chamber for W2 type caisson. Fig. 62 shows details of W1 type caisson. Fig. 63 shows close up view of Toyo pump and sand cutters installed in the working chamber. Fig. 64 shows close up view of precast working chamber before launching. Fig. 65 shows progressive construction of caisson. Fig. 66 shows removal of excavated laterite material with the help of bucket. Fig. 67 shows caisson resting on piled foundations.

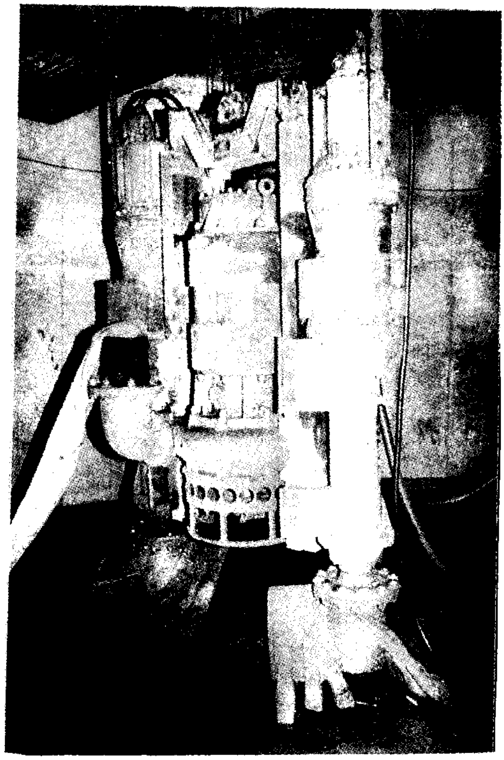


FIGURE 63 Close-up View of Installed Sand Cutters and Toyo Pump

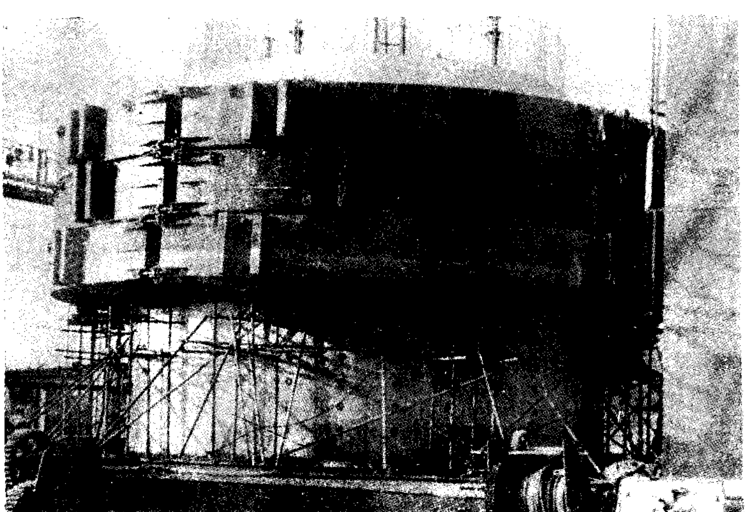


FIGURE 64 Close-up View of Precast Working Chamber before Launching

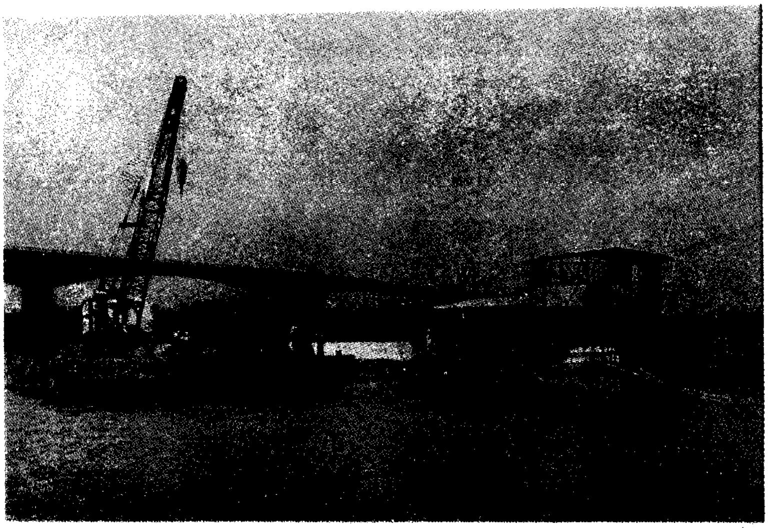


FIGURE 65 Construction of Caisson at P18 in Progress

FIGURE 66 Removal of Excavated Laterite material with help of Bucket

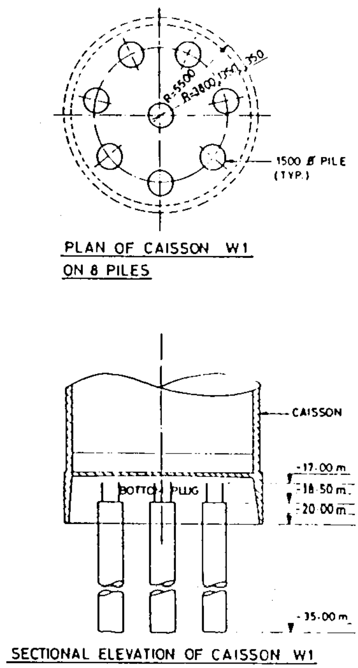


FIGURE 67 Details of Caisson W1 for Pier P15, P16, P17 of Zuari Railway Bridge

Innovations in Slope Stabilisation Works

Earthslide at Watawala in Sri Lanka (1992)

This earthslide has taken place along a 100 m stretch, on the railway line connecting Colombo to Badulla in Sri Lanka, at a distance of 161.3 km from Colombo.

At this place, the railway line is passing at an elevation of 978 m above MSL, through gently slopping hilly terrain (Fig. 68). This area receives heaviest rainfall in Sri Lanka, approximately on an average 5000 mm/year. At this particular site, earthslides have been taking place since 1956. Major earthslides had taken place in 1981, 1985, 1988 and 1991.

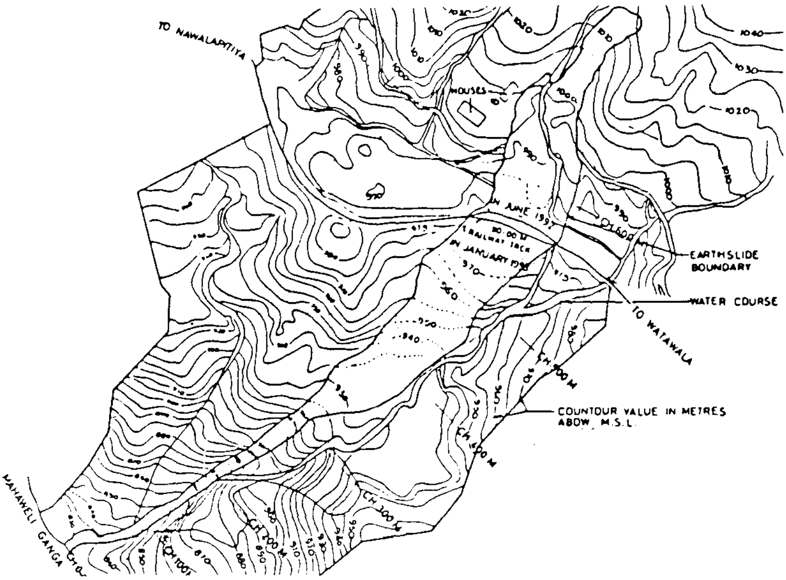


FIGURE 68 Contour Plan at the location of Watawala Earthslide

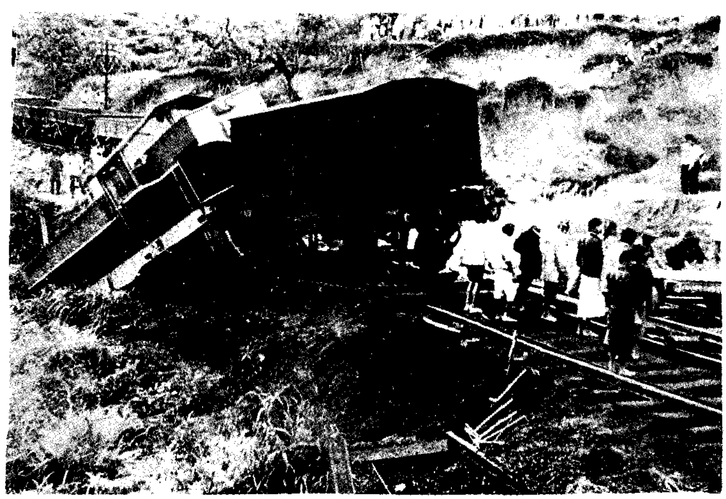


FIGURE 69 Derailment of Goods Train at Watawala on 3rd June 1992 due to Earth-Slide

On 3rd June, 1992, following a heavy rainfall, the earthslide was once again triggered. This time the engine of a passing goods train was derailed(Fig. 69) on account of failure of track due to slide. Following this slide, detailed investigations were undertaken to study the mechanism of earthslide.

The area affected by earthslide, was 80-90 m wide on an average and extended about 250 m over the up hill slope and about 150 m over the downhill slope of the track.

It was observed that, average vertical subsidence at railway track level was 2.5 to 3.0 m and following heavy rains, lateral surface movements took place at a much faster rate and on some days, movements of order of 1 m/day were observed.

AFCONS was invited to drill 6 numbers of deeper boreholes in the proper earthslide affected area. AFCONS completed the field work between 29th January'93 and 7th March'93. The findings of this investigation are briefly summarised below

Testing and Analysis

Undisturbed samples collected, were tested in laboratory primarily for classification and estimation of shear strength parameters. Observed geomechanical properties of colluvium material in general, are presented in Table 3.

A large number of in-situ tests were carried out on boundary shear surface, wherever evidence of slickenside was available. These tests conducted in the month of March/April-93 indicated

$$c'_p \text{ (peak)} = 0.02 \text{ to } 0.03 \text{ kg/cm}^2$$

$$\phi'_p \text{ (peak)} = 27^\circ \text{ to } 28^\circ$$

Slope Stability Analysis

As these shear strength parameters are initial strength values (peak), they get considerably reduced (to residual strength), when sliding surface undergoes large amount of relative displacement. Colluvium soils, are generally stiff to hard and in undisturbed condition, individual samples have relatively high shear strengths. However, creep or sliding process during slope development, reduces the shear strength along movement surfaces, to residual or near residual levels. Gray has reported residual strength values as c'_r = zero and $\phi'_r = 14^\circ - 18^\circ$ for a slope in colluvium, where c'_p = zero and $\phi'_p = 27^\circ$ were found out from laboratory testing. In-situ block shear strength tests conducted on slickensided surfaces suggested values to be

TABLE 3
Geomechanical Properties of Colluvium

Soil Description	yellowish brown sandy silt/silty sand
Grainsize distribution	
Gravel	2%-15%
Sand	37%-72%
Silt	8%-48%
Clay	2%-20%
Consistency limits	
Liquid limit	43% - 57%
Plastic limit	28% - 45%
Plasticity index	12% - 18%
Classification SM - SC Ml	
Specific gravity	2.58 - 2.71
Bulk density 1.56 - 1.75 t/cum	
Natural moisture content	30% - 36%
Natural dry density	1.25 - 1.45 t/cum
Cohesion (peak)	0.05 - 0.20 kg/cm²
Angle of shearing resistance (peak)	25° - 32°

c'_r = zero and $\phi'_r = 18°$ and these were accordingly adopted for analysis.

The entire slope, was divided into number of sections of varying length between 100 – 120 m each. Slope stability calculations were performed for each section using Bishop's simplified method. In these calculations slip circles were so selected, that they passed along the predetermined probable slip surface. A large number of trials were conducted to determine the critical circle giving lowest factor of safety. In the region near the crown of the slide, where artesian conditions existed, ground water table was considered above ground. In the remaining slope portion, ground water table was considered at the ground surface, as the worst condition which can take place in the monsoon.

The results of these computations indicated, slope to be having higher than 1.0 as factor of safety during dry season and factor of safety falling close to 1.0 or less than 1.0 during monsoon, thereby causing slides during heavy monsoon.

Causative Factors

(a) The overburden soil is made up of colluvium and at many places, it is in a loose to medium dense state with very low N values of standard penetration test.

(b) The overburden is underlain by completely and further highly weathered rock of granitic gneiss and charnockitic gneiss types.

(c) The extent of weathering is considerable, upto depths varying between 30 – 50 m. (Fig. 70)

(d) The earthslide effected area is being fed with water from two perched underground water basins at higher elevations, through two aquifers converging in earthslide region.

(e) Artesian conditions were observed near the head region of slide, indicating existence of some barrier/other factors leading to poor subsurface drainage conditions.

(f) At many places along the boundary shear surface, slickensided surfaces indicating very large relative movement were observed, even at substantial depth. This indicated that it would be reasonable to assume that residual shear strength parameters would be actually acting along slip surface.

(g) From number of instrumentation readings, borehole records and other observations, a well defined slip surface could be constructed along the slope.

On the basis of the ground profile, after the slide, back analysis was carried out for the slope on the verge of failure and it was observed that ϕ was varying between 23° to 31° in different sections of the slope, when c' was assumed as zero.

Before permanent remedial measures could be undertaken, the monsoon of 1993 season started. Once again, the earthslide had been triggered in the last week of May/first week of June 1993. Fig. 71 indicates in plan, the extent of 1992 earthslide and details of retriggered earthslide in 1993 monsoon. Fig. 72 indicates location of probable slip surface.

Remedial Measures

The stability of a slope, depends on the equilibrium of the forces and static moments acting on it. The magnitudes of these forces, may change

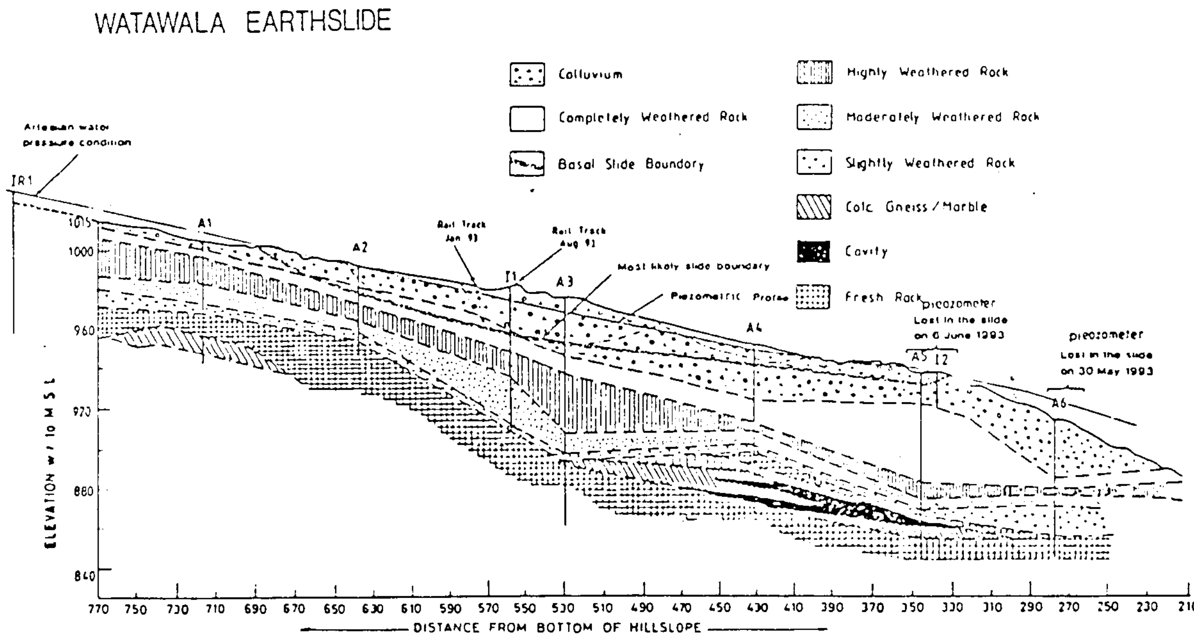


FIGURE 70 Subslope Profile at the Site of the Earthslide Established through Drilling

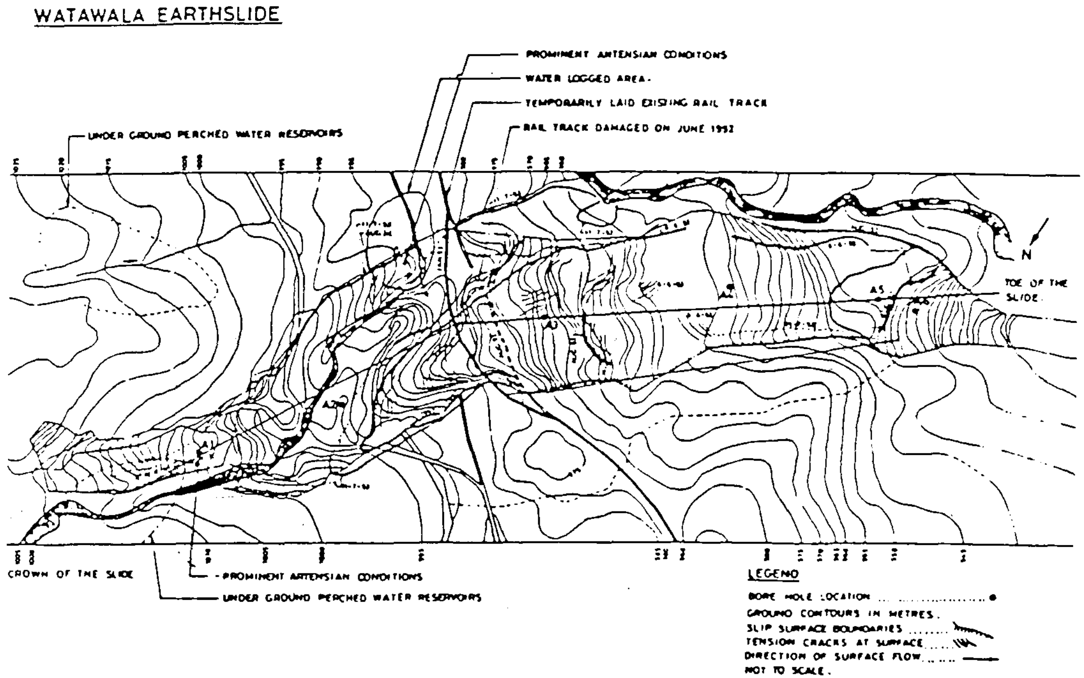


FIGURE 71 Plan of Watawala Earthslide showing damaged Railway Track and Bore Hole Location

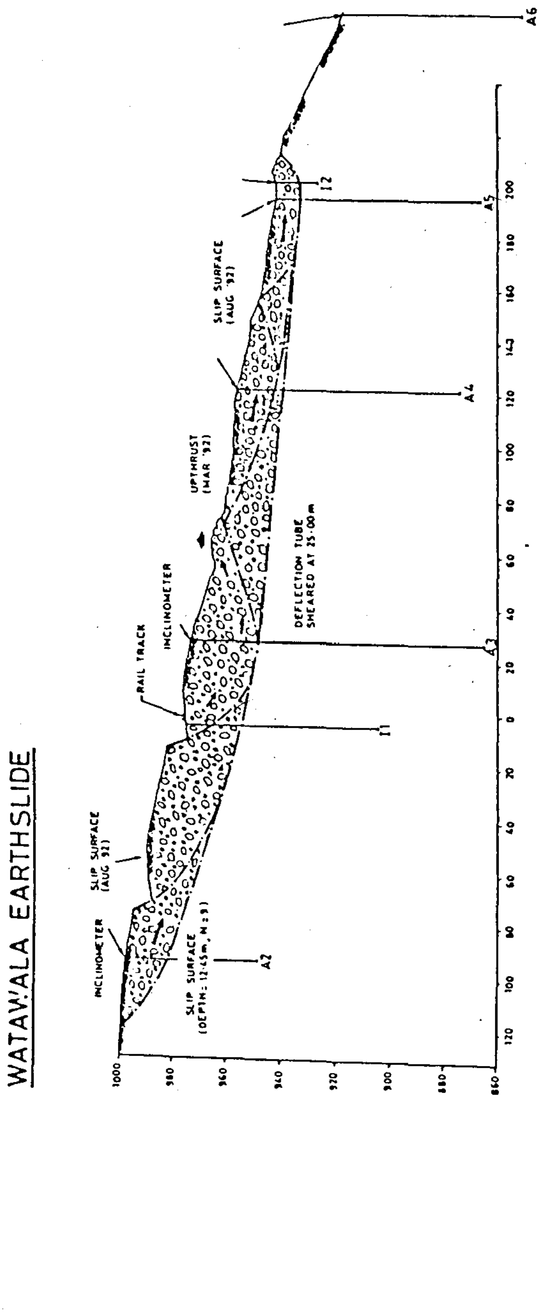


FIGURE 72 Location of Probable Slipsurface for 1992 Earthslide at Watawala

under the influence of various processes taking place on the slope or in it. As seen in the causative factors of the slide, following processes have been found responsible for activating this slide viz.

a) Increasing hydrostatic or hydrodynamic pressure in the sliding body and the underlying formations, due to rise in the water table, on account of rainfall, and continuous infiltration of groundwater into the earthslide region through two converging aquifers originating at higher elevations.

b) Reduction of shear strength at actual or potential sliding surfaces by wetting of the groundmass and softening of kaolinitic clay pockets.

c) Reduction of weight of artificial structures in the lower part of the slide by erosion.

d) Reduced resistance to sliding, in front of the slide, due to failure of artificial measures like toe constructed in the lower part of slide.

The disturbance of the equilibrium, resulting in the sliding, may be due to one or more of these processes. Hence, remedial measures taken to control the earthslide, should produce effects, opposite to those of processes mentioned above. Thus, having decided the primary objectives to be fulfilled by remedial measures, following remedial measures are suggested to control the earthslide. These have been divided into two groups eg.

i) Measures which reduce the active forces.

ii) Measures which increase resistance to sliding.

Measures Which Reduce the Active Forces

a) Reduction of weight of the soil/rock, in the upper part of soil, by removing the soil/boulders etc.

b) Reduction of hydrostatic and hydrodynamic pressure, on ground water in the sliding body and its substratum by

- draining the surrounding ground or covering it up by impermeable geotextiles to prevent water getting into the sliding body;

  As laying of geotextile, over such a vast area in crown, will be very time consuming and expensive proposal, only draining the surrounding groundmass in the head region of earthslide is proposed.

This can be done by drilling 100 mm diameter 30-40 m long horizontal drain holes in the slope. Perforated pipes may be installed in these holes to collect water.

- making impermeable screens in surface of the slide to prevent the percolation of surface runoff into the slide;

  This may be done by excavating trenches 2-3 m deep, 1.5 m wide at base, Lining these with impervious geotextiles and filling these trenches with permeable materials like gravel/cobbles. By forming a systematic net work of such trenches, surface runoff can be diverted away from slide area.

- draining the sliding body, with the aim of reducing and diverting hydrodynamic pressures in it.

  This may be done by a number of ways eg.

  1. By drilling horizontal drain holes, penetrating the earthslide affected area, from the stable area on both the sides. In this approach, the number of drill holes to be drilled becomes quite large and lengths to which horizontal drill holes can be drilled is limited to 50 - 60 m. Besides, there is likelihood of these drains getting damaged, if earthslide again takes place.

  2. By installing deep rammed stone columns. This may be done, at the proper rail track level. By forming a network of closely spaced stone columns, hydrostatic pressure in groundmass below rail track will be lowered, as these stone-columns filled with granular material, will act as drainage path and water collected in these stone columns, will come up to ground surface and shall be transferred away through surface drains.

  3. By constructing horizontal and inclined drain holes, at greater depths through the sides of large diameter caissons constructed in the proper earthslide area. This may be done by sinking a 6 - 8 m diameter caisson, upto a depth of 25 - 30 m in stable formation first. Drilling horizontal drain holes of 100 mm diameter and 30 - 40 m length in proper orientation, at regular interval, along depth of caisson through side walls of caisson. Transfer of water collecting in caisson through large diameter pipes installed in the lower portion of caisson. This will facilitate draining of slope upto a greater depth, say 12-15 m below ground surface, in the proper earthslide area. Thus, by preventing water from rising into the sliding body upto ground

surface, hydrostatic pressure acting on the sliding surface shall be considerably reduced.

Measures Which Increase Resistance to Sliding

a) Augmenting resistance to shear, at actual or potential sliding surfaces, by

- increasing the shear strength of the earthmass, by reducing its water content. This shall be achieved by dewatering or draining the slide zone as mentioned earlier.

- Replacing natural weaker material with material of greater shear strength. This shall be achieved by filling the trench drains with materials like, gravel/cobbles which are free draining in nature and which are stronger than colluvium. By formation of granular compacted piles or rammed stone columns, material up to considerable depth shall get replaced with compacted gravel. This will improve the drainage characteristics of groundmass at rail track level. Besides around these stone columns, diameter will also get improved substantially.

- Reinforcing the bond between the sliding body and the underlying formation. This may be achieved by constructing rows of large diameter R.C.C. piles, socketed into weathered rock below the railway track. In addition, the stability of formation below railway track and overall sliding slope, may be improved by construction of suitably spaced prestressed rock anchors.

b) Increasing the weight of earthmass in the lower part of the slide, or preventing reduction there of by

Preventing undermining the toe region of the slide and promotion accumulation of stronger materials at the toe of the slide. This may be accomplished by constructing a strong rock toe by placing layers of well compacted rockfill material near the lower end of the slide.

c) Increasing the weight of artificial structures on the lower part of the slide by erecting structures specifically for landslide control. This may be achieved by constructing 2 – 3 m high gabion walls in the toe region. Gabions may be constructed by forming boulderfilled crates, formed of noncorrosive zinc coated steel wires, arranged in regular geometrical shapes.

d) Abutment works : construction of structures providing buttressing against

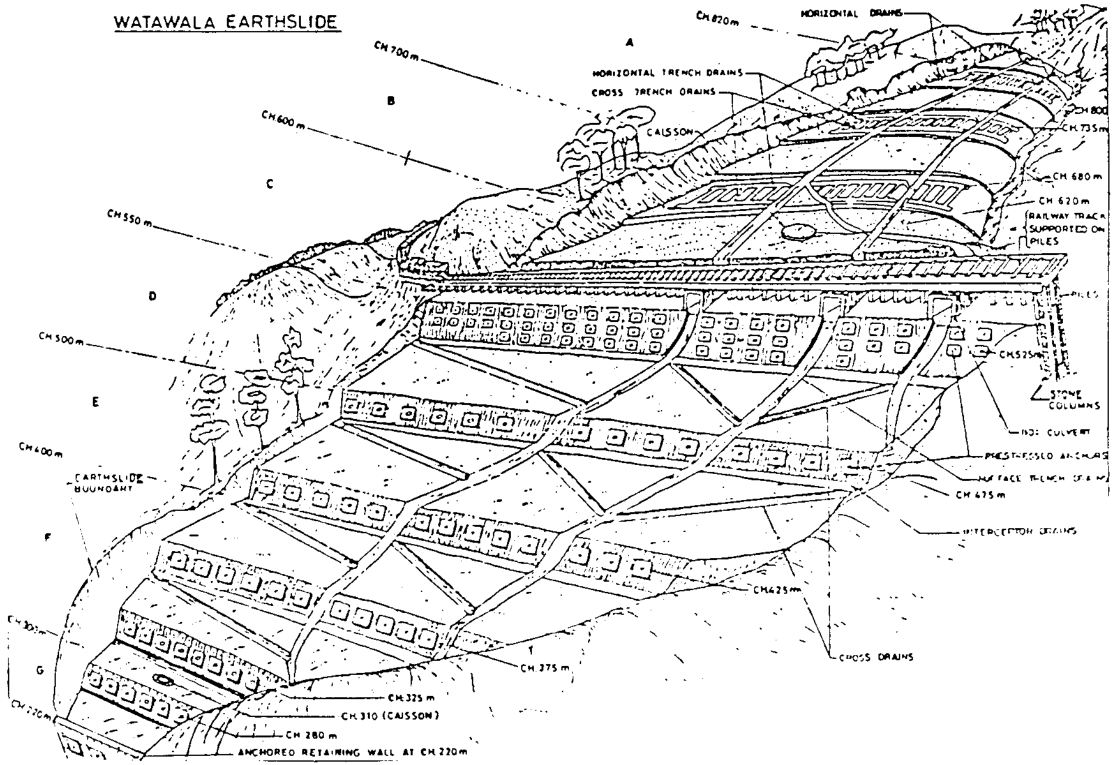


FIGURE 73 A Conceptual Scheme of Permanent Control Measure for Stabilisation of Hillslope at Watawala

sliding. This may be accomplished by constructing a 6-8 m high RCC retaining wall anchored into rock, at lowest portion of the slide.

e) Artificial coverage of the slide, with or without bonding to the stable base. This may be accomplished by planting local varieties of deep rooted trees, bamboos, eucalyptus trees, at regular intervals in a mixed pattern along slope. The growth of trees requires a large time frame, but once fully grown, root systems of these trees help in stabilising the slope and as well help in preventing the surface erosion of the soil.

A conceptual sketch of proposed remedial measures at this site is indicated in Fig. 73.

The hill slope is going to be stabilised on basis of these measures soon.

Special Marine Construction

Construction of Oil Berth Using Caissons At Vishakhapatnam

406.8 m long oil berth was constructed in Vishakhapatnam Port. Fig. 74 shows oil berth and complete layout of system in Visakhapatnam Port. Fig. 75 shows elevation and plan of oil berth.

This oil berth is made up of 5 Mooring Dolphins, 2 Berthing Dolphins and 1 Central Unloading Platform. This oil berth is a part of entire oil conveying system to 3.0 M TPA HPCL Refinery at Vishakhapatnam.

For construction of these units following types of caissons were used.

	Nos.	Caissons/Unit	Caissons
Mooring Dolphin	5	4	20
Berthing Dolphin	2	6	12
Central Unloading Platform	1	8	8
		Total	40

Construction method

The Top Level of caissons was +4.0 m and these were to be founded

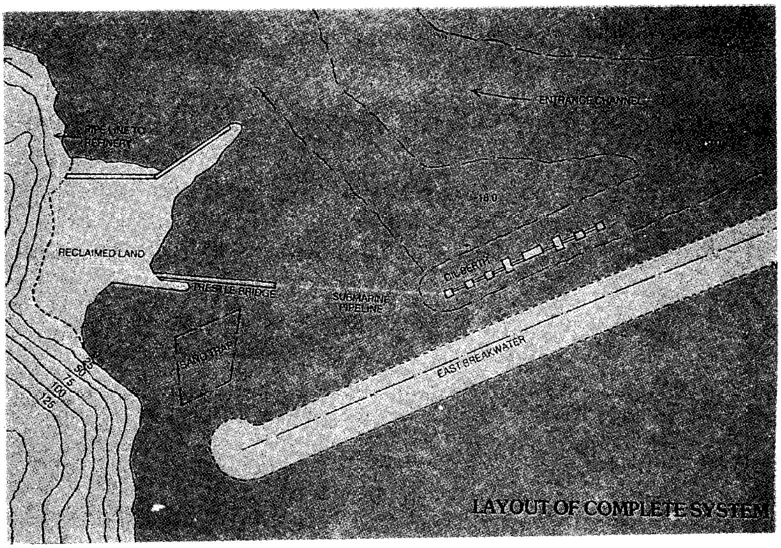


FIGURE 74 Layout of Oil Berth at Vishakhapatnam Port

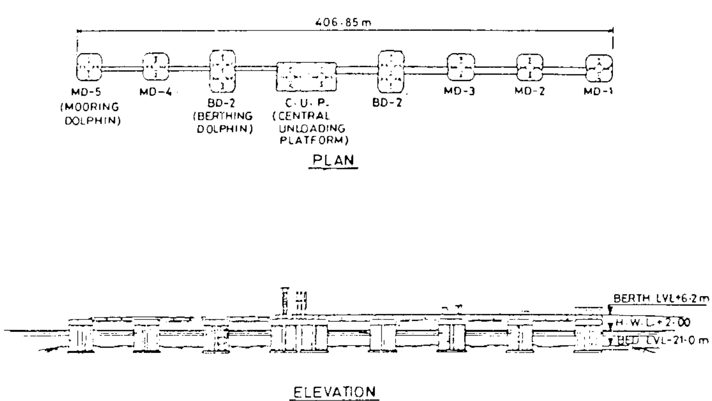


FIGURE 75 Details of Oil Berth at Vishakhapatnam Port

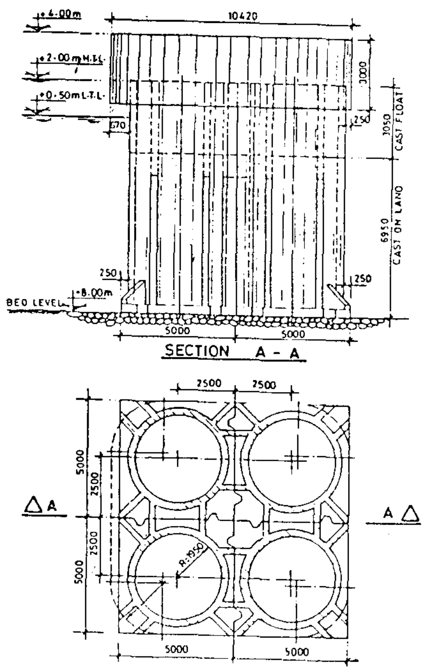


FIGURE 76 Plan of Single Caisson for Oil Berth at Vishakhapatnam

at an average bed level of -21.0 m. The heights of completed caissons varied from 25 m to 30 m. Methodology of construction is briefly described below

i. The parts of 4.5 m diameter, 6.95 m high caissons known as crib unit, were cast in casting yard on reclaimed land adjacent to shore. Fig. 76 shows plan and elevation of single caisson. Fig. 77 shows plan and elevation of combined unit of caisson for Mooring Dolphin. Fig. 78 shows concreting of crib unit in casting yard.

ii. This single caisson unit was shifted to slipway using hydraulic jacks. Fig. 79 shows shifting of crib with hydraulic jacks.

iii. Caisson unit was launched in sea with use of jacks, cranes. Fig. 80 shows completed unit ready on platform for launching. Fig. 81 shows

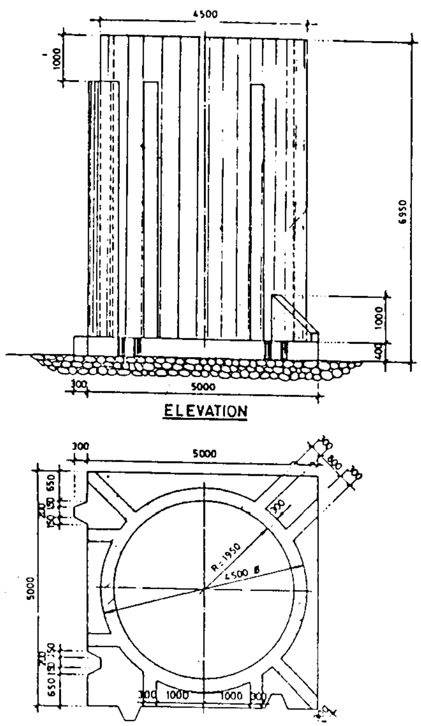


FIGURE 77 Plan of Combined Unit for Mooring Dolphin of Oil Berth at Vishakhapatnam Port

launching of crib unit. Fig. 82 shows crib unit ready for placing in position.

iv. Height of caisson was increased from 6.95 m to 12.00 m in floating condition near slipway. Fig. 83 shows joining and increasing steining height of caisson in floating condition.

v. Two units which were put in water were brought together, aligned and bolted with top bracket.

vi. Special type of rubber seals were fixed on sides of adjacent caissons, water between two projection of was pumped out, so that units came closer and rubber seals got compressed. Outside hydraulic pressure keeps both the caissons joined together. The base slab in these pockets was concreted rigidly and further height of caisson was increased. Fig. 84

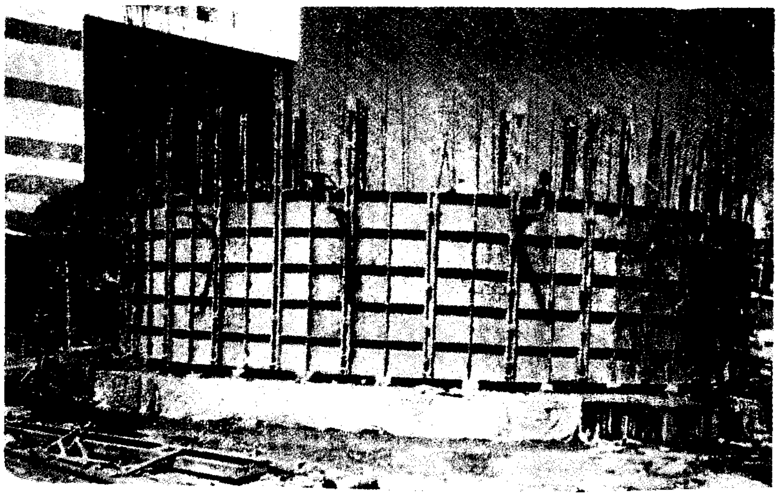


FIGURE 78 Concreting of Crib Unit in Casting Yard

shows details of joining of two caissons with help of rubber seals.

vii. At the desired location of caisson, the bed was prepared by carrying out dredging upto (-)21.0 m below HSL. The bed was leveled manually in dry condition using Diving Bell. Dredging was carried out using DP 50B Toyo pump attached with 2 numbers PK 10 Type cutters. With this equipment dredging upto 21 m depth could be carried out at rate of about 150 cum/hour. Dredged material was collected in barges and towed away to dumping yard 2 km away.

viii. Caisson units thus joined together, were brought to its final position and sunk by pumping water inside the caisson. After the entire unit rested firmly on founding bed, additional deck slabs and other structural components were cast.

Finally, for additional stability, ballast was filled inside the caisson to certain extent. Fig. 85 shows caissons in final position.

ix. Upon completion of construction of all 40 caissons, the superstructure was completed and installation of 36" diameter pipeline, unloading arm etc. were completed. Fig. 86 shows pipeline installation work in progress on completed oil berth.

The work of construction of 40 caissons, carrying out necessary dredging

FIGURE 79 Shifting of Crib Unit with Hydraulic Jacks

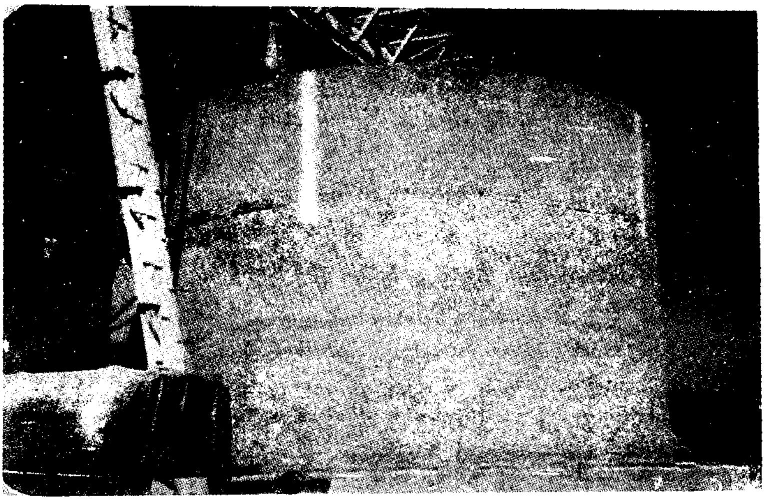


FIGURE 80 Completed Crib Unit ready for Launching

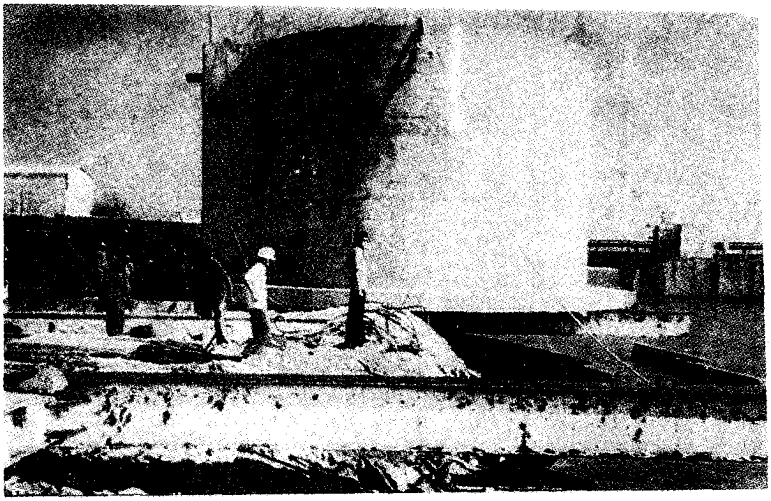


FIGURE 81 Crib Unit ready on Platform for Launching

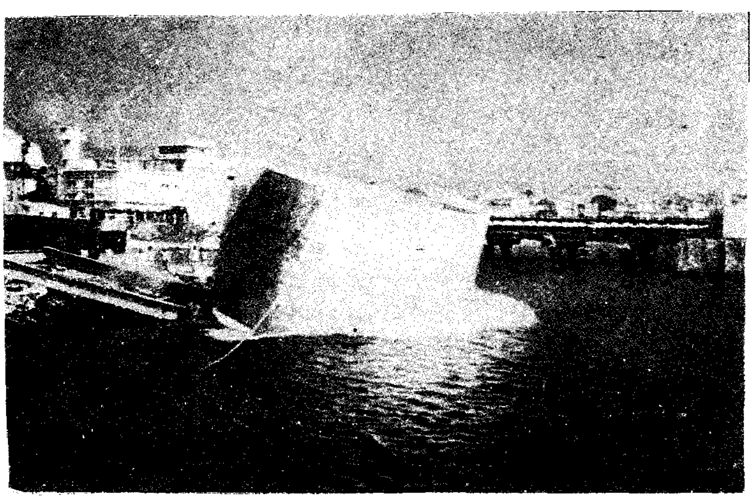


FIGURE 82 Launching of Crib Unit

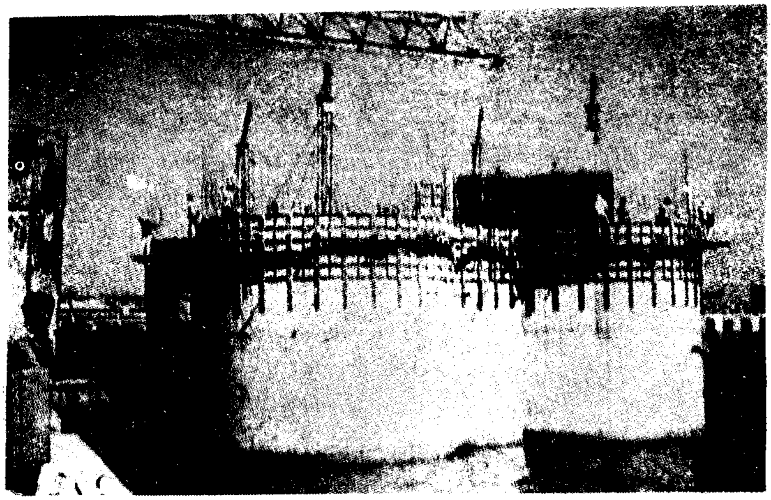


FIGURE 83 Joining and Increasing Height of Steining of Caisson in Floating Condition

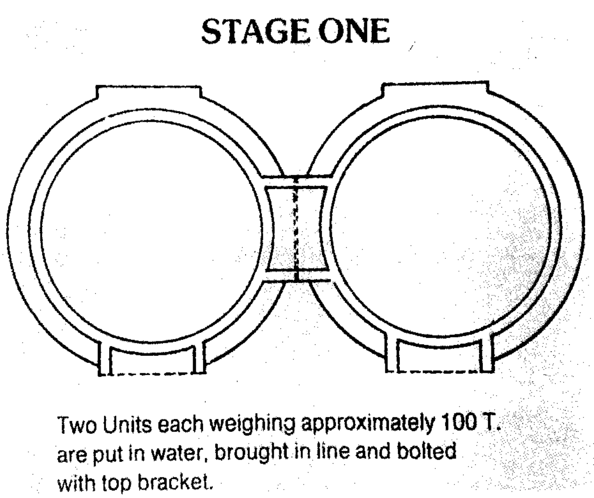
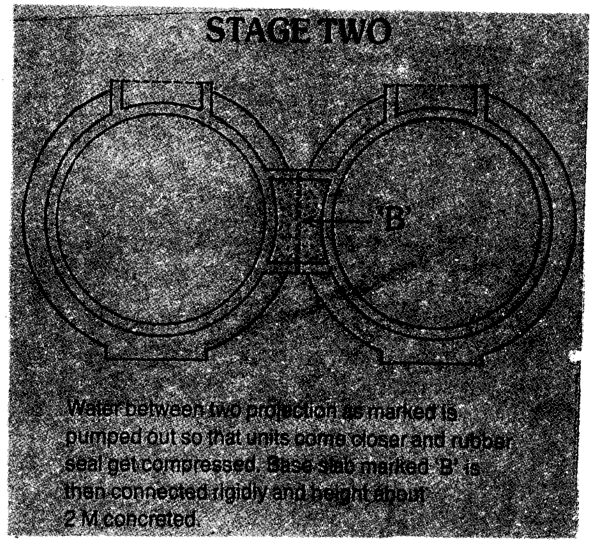


FIGURE 84 Joining of Caissons in Stages



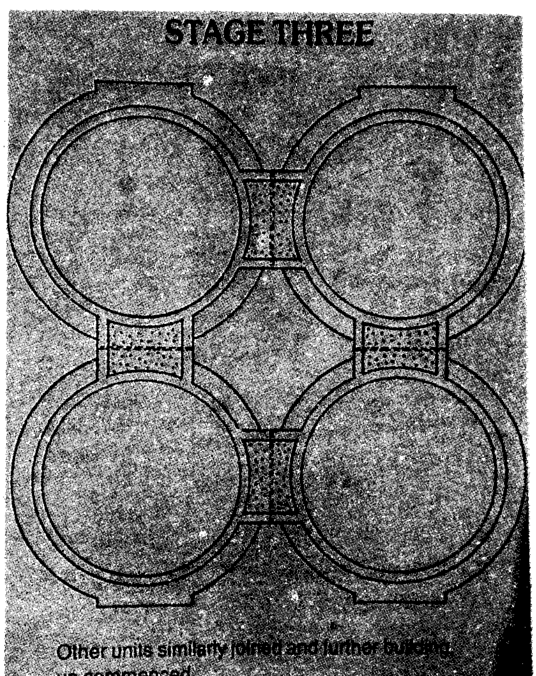


FIGURE 84 Cont..

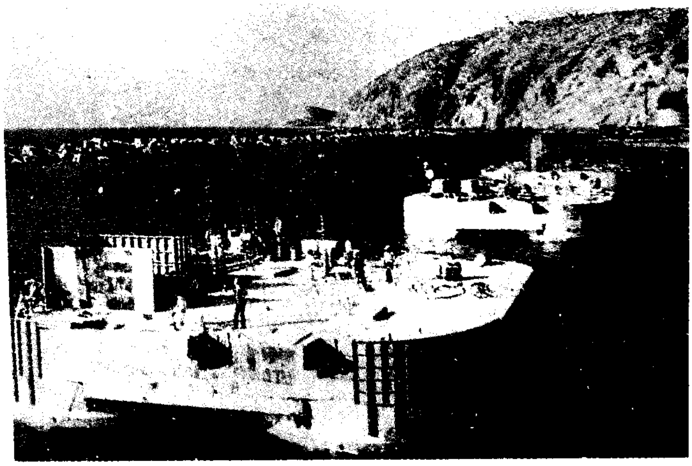


FIGURE 85 Completed Caissons in Position

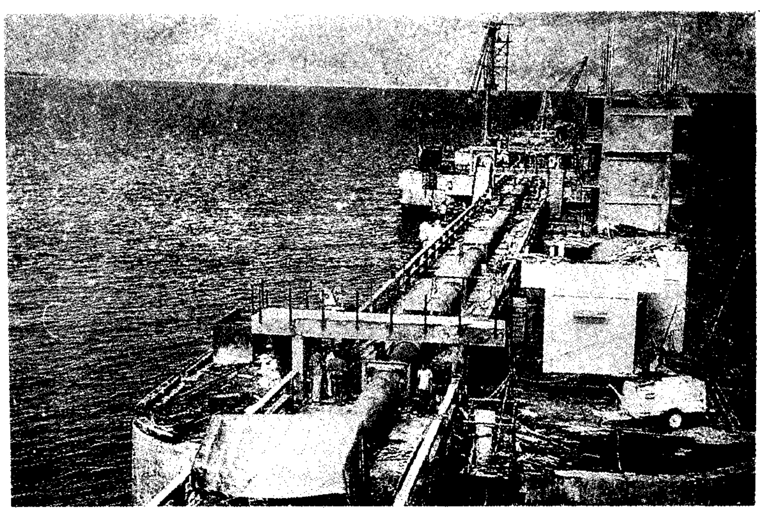


FIGURE 86 Pipeline Installation Work on Oil Berth nearing Completion

of 1,50,000 cum of sand from sea bed and installation of pipeline on oil berth, was completed in a period of 24 months. The oil berth was commissioned in 1985.

Construction Of Submarine Pipeline Across Thane Creek

National Organic Chemical Industries Limited (NOCIL), use Naphtha as feedstock material for their various products. They receive their Naphtha from BPCL refineries in Trombay area. This is conveyed to NOCIL plant situated in Thane Belapur area through a 24 km long, 219 mm internal diameter pipeline. For almost 15 years,i.e. from 1973 to 1988 this pipeline crossed Thane Creek over the existing road bridge. As the condition of superstructure of this road bridge deteriorated, PWD, Government of Maharashtra, instructed NOCIL to remove the pipeline from bridge. This necessitated construction of submarine pipeline across Thane Creek bed. Fig. 87 shows layout plan of NOCIL Pipeline.

Various stages of construction of this pipeline are described briefly below, as

a. Preparation of pipes and trench for laying pipe.
b. Actual laying of pipes.

Preparation of Pipes

As, the pipeline is passing through aggressive marine environment, elaborate arrangements were made to protect the pipeline from corrosion. These included the following

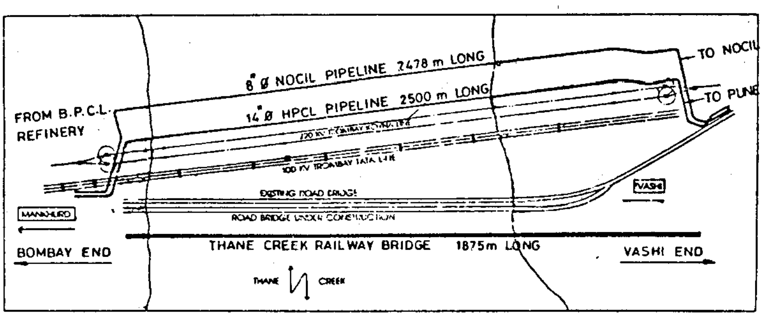


FIGURE 87 Layout of Submarine Pipeline across Thane Creek

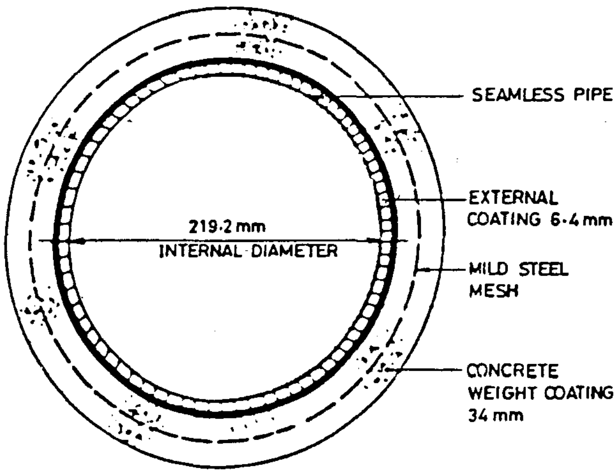


FIGURE 88 Cross Section of NOCIL Pipeline

a. Sand Blasting — To clean external surface of pipes.

b. Corrosion Coating — Coating with epoxy paint, coal tar enamel, reinforced fiber glass tissue and craft paper. Total thickness 6.4 to 10 mm.

c. Wet Coating — Coating with reinforced concrete of M-35 grade. Total thickness 30 to 34 mm.

d. Curing — By wrapping up concrete surface with wet gunny bags for 8 – 10 days. Fig. 88 shows typical cross section of pipe. Fig. 89 shows operation of coating and wrapping of pipes. Fig. 90 shows cold bending of pipes.

TABLE 4
Salient Features of NOCIL Pipeline

1.	Steel pipe	ID 219.1 mm
2.	Internal design pressure	45 kg/cm^2
3.	Product conveyed	Naphtha at 65°C
4.	Total length	5918 m
5.	Length of submarine pipeline	2478 m
6.	Dead weight of submarine pipeline	372 tonnes.

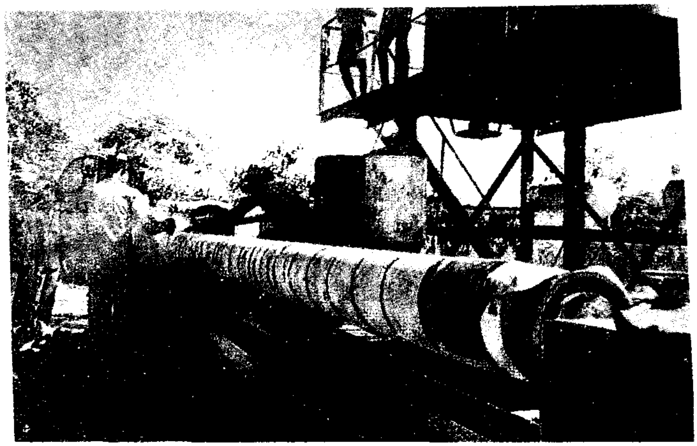


FIGURE 89 Operation of Coating and Wrapping of Pipes

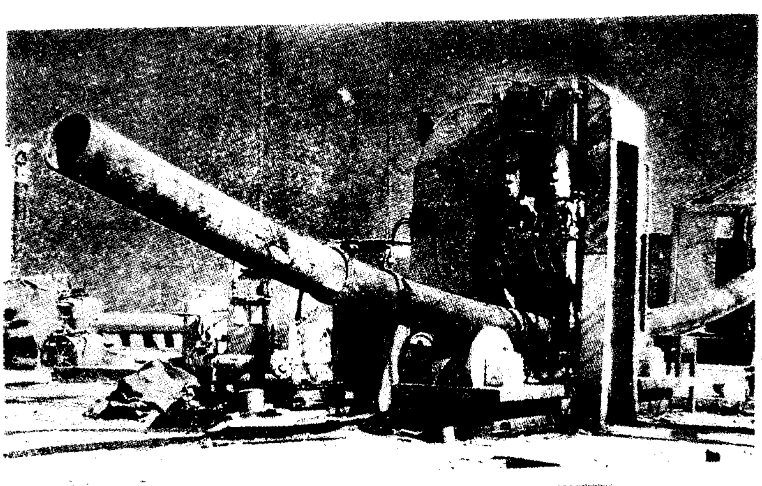


FIGURE 90 Cold Bending of Pipes using Pipe Bending Machine

Table 4 briefly summarises the salient features of this pipeline.

Welding and Stringing of Pipes

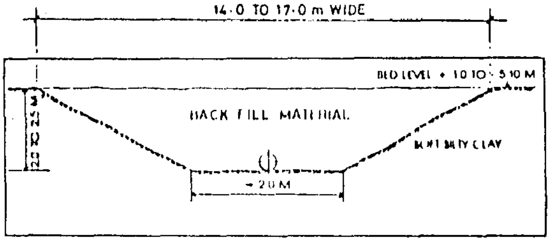
Generally, 29 or 30 number of pipes were welded together, to form a string of around 194 m length. In all, 13 strings were made at the stringing yard, for laying in the 2478 m long submarine section. Fig. 91 shows pipe strings kept ready for pulling.

Preparation of Seating Bed for Pipeline

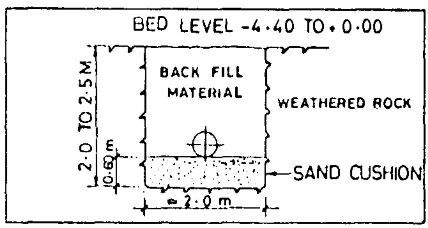
The pipeline was laid in a specially made trench of 2 m width and depth between 1.2 m to 1.7 m below creek bed.

In the rocky portion, the pipeline was rested on a 600 mm thick sand pad. After laying the pipeline, these trenches were backfilled with well

FIGURE 91 Strings of Pipe kept ready for Pulling



SECTION OF TRENCH IN SOIL



SECTION OF TRENCH IN ROCK

FIGURE 92 Details of Trench for Submarine Pipeline across Thane Creek for NOCIL

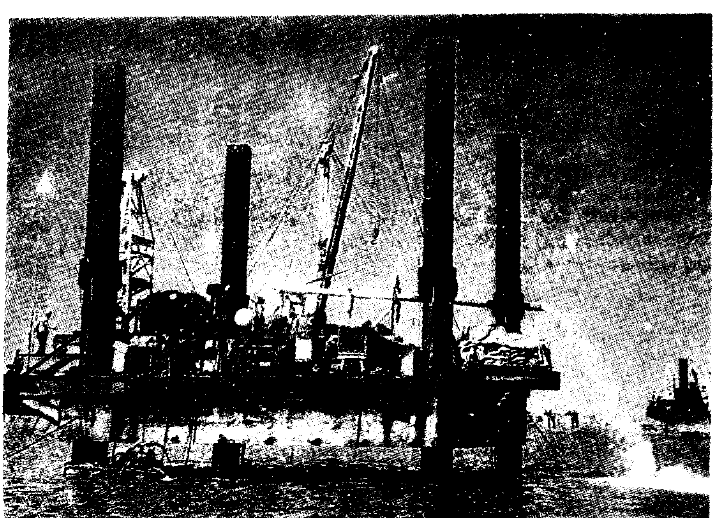


FIGURE 93 Excavation of Trench in Rock using RH6 Rig on Jackup Platform

graded cohesionless backfill material. Fig. 92 shows details of trench made for seating the pipeline.

Excavation of trenches was carried out by using hydraulic excavators, dragline, rock breakers and even manually also.

In the midstream portion, for a 200 m length the trench was prepared using a reverse circulation rig, which is normally used for construction of diaphragm walls, mounted on a jack up platform. Fig. 93 shows trench excavation using RH-6 rig on jack up platform.

Laying of pipeline

The stringing yard was on the Vashi side of Thane Creek. From here, the pipeline was to be laid across Thane Creek, in the already excavated trench. For this purpose a 200 tonne linear winch was used.

This 200 tonne winch and empty reel winder were mounted on specially made pedestals, on pile foundations on Bombay end. These were operated with hydraulic powerpacks.

A 38 mm diameter messenger wire rope was laid between Vashi end to Bombay end, all along the trench, connected to winch and taken back to Vashi end. Positioning of this wire rope in the trench was done with help of EDM and Laser beam.

A 64 mm diameter wire rope reel was mounted on the stand at Vashi end. This wire rope was also laid in similar manner, as adopted for 38 mm diameter wire rope. The free end of 64 mm diameter wire rope was attached to the pulling head, which was fitted with specially made buoyancy tanks, to keep the pulling head above bed level during pulling.

The pulling of the pipeline was commenced with the pulling of the first string. The speed of pulling by Linear Winch was carefully controlled by hydraulic power packs. After the 1st string was pulled from the launching ramp, its other end was connected to the beginning of the 2nd string. In this manner, sequentially, all the strings were pulled. Maximum pulling force required during the pulling was 120 tonnes. Pulling of all the 13 strings i.e. 2478 m long submarine pipeline section was completed in 5 days. Fig. 94 shows pulling of pipe string using winch.

Testing of Pipeline

After the laying of entire pipeline was completed, it was hydrotested with water upto 71 kg/cm^2 pressure for 24 hours. Upon successful completion

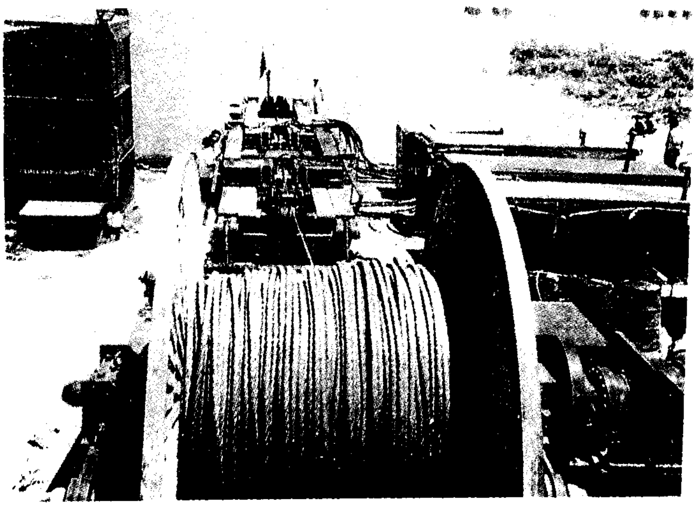


FIGURE 94 Pulling of Pipe Strings using 200T Linear Winch

of hydrotesting, the ends of rerouted pipeline were connected with the existing pipeline. The entire work was successfully completed in a period of 12 months, using imported 200 tones linear winch with a minimum guidance from expatriate technicians by AFCONS. The pipeline is in operation since June 1989.

Bed Protection Work In River Hooghly with Use of Geomattresses

Ministry of Surface Transport (M.O.S.T.) decided to develop Port of Haldia in late '70s, as necessary draught required for ships at Calcutta Port, could not be maintained, on account of heavy siltation in Hooghly river. Haldia which is at a distance of about 90 kms down stream of Calcutta, had sufficient draught at that time.

Calcutta Port Trust undertook comprehensive measures in 1981, to ensure a draught of 7.90 m for 200 days, at Calcutta Port in a year and a draught of 10.67 m for 320 days, in a year, at Haldia Port.

It was decided, to construct a guide wall of 2800 m length at the Northern end of Nayachara island and a guidewall of 7000 m length at Southern end.

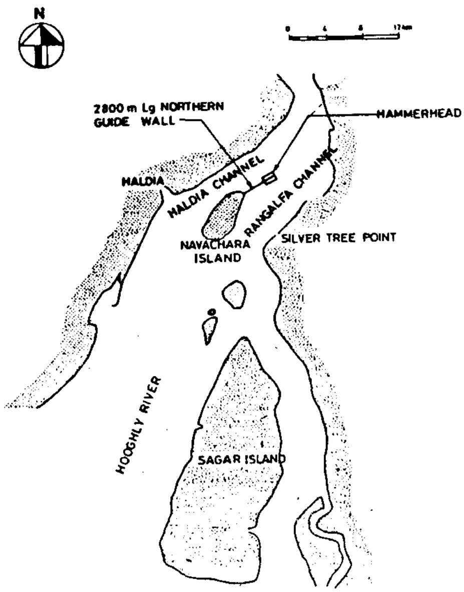


FIGURE 95 River Training Work at Nayachara - General Layout

The technique for construction of this 2800 m guide wall using composite R.C.Hexapod, Bamboo bricks did not prove successful. As, it was found out that the guide wall had subsided due to very mobile nature of sand bed. This called for proper bed and bank protection.

The projected bottom protection around the head (hammer head) of the guide wall was to be constructed by a new technique, with the help of mattresses made of geofabric, fascine wieps and topped with ballasting material stones. This was the first time in India, that such kind of work was undertaken by any Indian construction firm.

Hammer Head Protection

Hooghly river which is about 825 m wide at Calcutta, becomes almost 9000 m wide at Haldia. Nayachara island is located at a distance of about 3.6 km from Haldia. The work involved construction of a 2800 m long

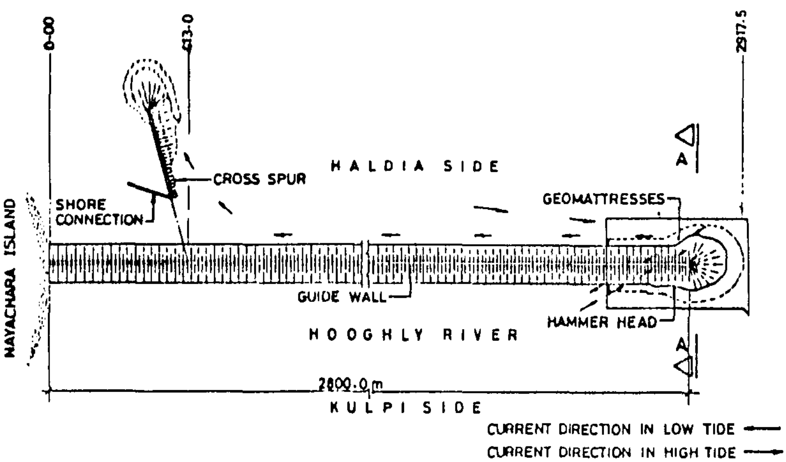


FIGURE 96 Scheme of River Training Work in Hooghly River at Nayachara Island

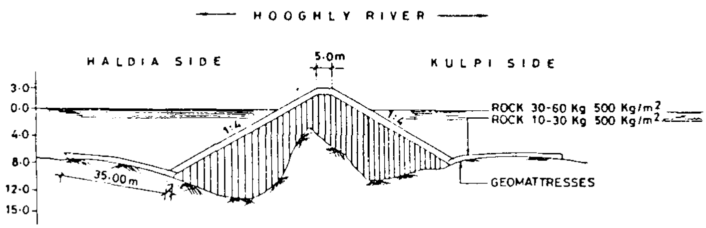


FIGURE 97 Section of Guidewall Protected by Geomattresses

guide wall and bottom protection of hammer head. In all, 31 geomattresses varying from 17.25 m × 30.25 m to 22.75 m × 50.25 m were sunk, covering an area of almost 24,000 m². Fig. 95 shows location of guide wall and hammer head. Fig. 96 shows plan of river training work at Nayachara island. Fig. 97 shows cross section of guidewall protected by geomattresses. Fig. 98 shows detailed layout of geomattresses for bed protection of guidewall.

The geomattress was a combination of one layer of nonwooven and one layer of woven polypropylene geofabric. Nonwooven geofabric was used below the woven fabric.

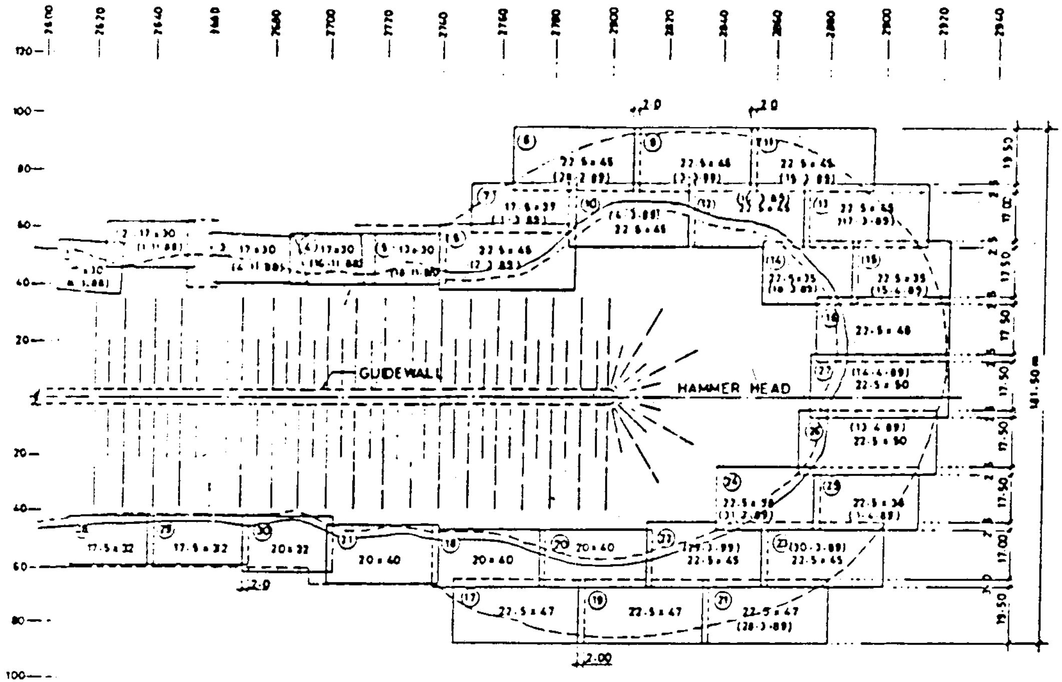


FIGURE 98 Detailed Layout of Geomattresses for Bed Proterction of Guidewall

Construction Methodology

It was decided to protect the river bed near the toe portion of hammer head embankment, over a length of 35 m. Geomattresses were to be placed on the river bed, at a depth of 6 to 8 m below water surface. For this purpose, the area to be covered with geomattress was divided into 31 cells, generally of min.17.25 m × 30.25 m to max. 22.75 m × 50.25 m size. Construction of geomattress is briefly described below

i. At the beginning geotextile mattress was prepared. Geotextile material used was non-woven polypropylene and one woven textile. Woven and nonwooven textiles were stitched together. Geotextile had following properties.

Water permeability	—	80 litres/m^2/sec at 100 mm head.
Retention of fines	—	D90 = 150 microns
Tensile strength	—	Warp : Min. 4 kN/cm^2
	—	Weft : Min. 2.5 kN/cm^2
	—	Width of Geotextile Min : 5 metres.

Required number of geotextile pieces were sewn together, to form required area of the size of the panel, providing an overlap of 5 cm at each lap and along outer edges on 4 sides.

ii. Fascine Wieps were tied to the geotextile in a regular grid of 1 m × 1 m. Locally available Baens were used for this purpose. Brush wood branches of 5 mm thickness and 11 m length, were bundled together to form a bundle of 10 cm diameter. These bundles were tied in a pattern of 1 m × 1m above geotextile. These bundles were tied with the help of 12 mm diameter polypropylene wire rope.

The function of these Fascine Wieps was to act as reinforcement to geotextile, to act as cushion for dumping stones on it, to help in maintaining geotextile straight, in a unwarped/unbent position and to provide buoyancy during floating.

iii. Preparation of geotextile, attachment of Fascine Wieps was done on shore. The geomattress was carefully brought to water edge with help of winches. Fig. 99 shows Geomattress being towed from land to river.

2 Nos. of 18 m long, 60 cm diameter, pipes joined together, and filled up with concrete, was used as a supporting towing device during towing. These towing devices were attached to both the ends of geomattress and they acted as ballast beams. Geomattress was towed from shore to the

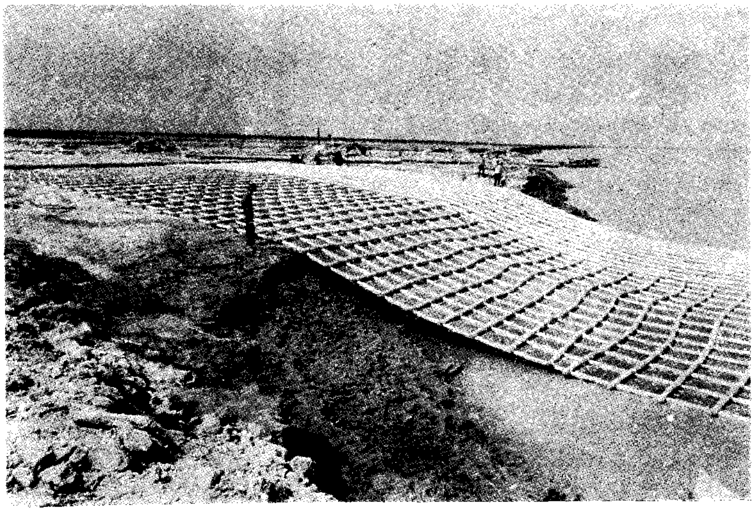


FIGURE 99 Towing of Geomattresses from Ground to River

desired location in floating condition, by attaching it to a tug boat with 250 HP engine. Fig. 100 shows a close up view of ballast beam and geomattress. Fig. 101 shows Geomattress being towed in the river.

iv. At the location of placement, the positions of both the mooring and sinking units was checked with trisponder, sextant and EDM. Stone supply pontoon was positioned between the two mooring and sinking units. Fig. 102 shows final positioning of geomattress prior to sinking. Fig. 103 and Fig. 104 show schematic details of different stages in sinking of geomattress on river bed.

These mooring and sinking units consisted of pontoons with a loading capacity of 120 tonnes each. 4 anchors were used to hold each pontoon in position. The 2 pontoons were interconnected with wire ropes to maintain desired distance between them. With help of 8 numbers of 5 tonne capacity hand winches the ballast beam was sunk to river bed. Fig. 105 shows a close up view of stone supply pontoon.

The sinking of one end of geomattress was followed by dumping of stones on the mattress. In the first stage of dumping, stones of 10 kg to 30 kg weight, were put from the supply pontoon on mattress. This was followed by dumping in second stage, when stones of 30 to 60 kg were placed. Depending on size of mattress, sinking stones for holding the mattress on ground varied as shown in Table 5. Fig. 106 shows a

FIGURE 100 A Close-up View of Ballast Beam and Geomattresses

TABLE 5
Details of Geomattress Panels

Parameter	Minimum	Maximum
Size of mattress	17.25 m x 30.25 m	22.75 m x 50.25 m
Area of mattress	521.8 m^2	1143.1 m^2
1st Dump 100kg/cm^2	52.18 tonnes	114.3 tonnes
2nd Dump 200kg/m^2	104.36 tonnes	228.6 tonnes
Total Weight of dumped stones.	156 tonnes	342 tonnes

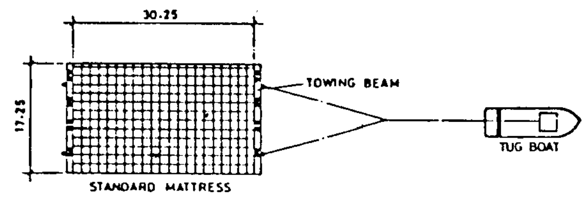


FIGURE 101 Towing of Geomattresses from Shore to Mooring Unit in the River

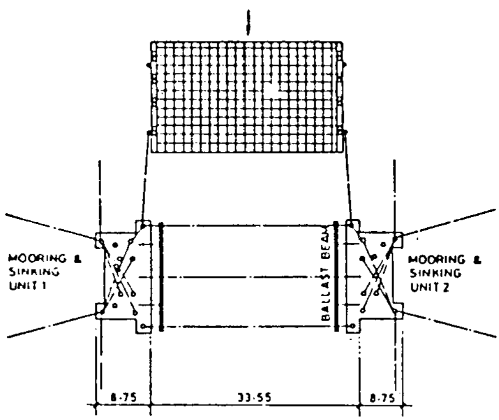


FIGURE 102 Positioning of Geomattresses between Mooring and Sinking Units at Placement Location

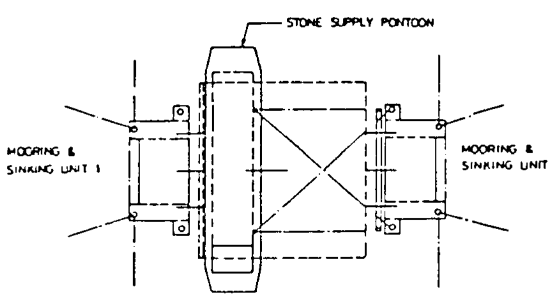


FIGURE 103 Lowering of Ballast Beam on One End Starting of Dumping Stones on Geomattresses

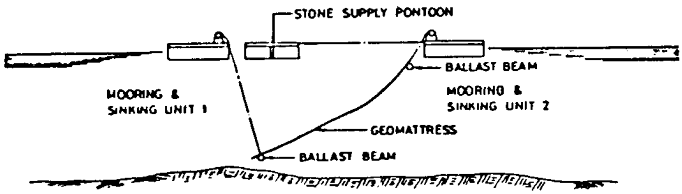


FIGURE 104 Lowering of Geomattresses on River Bed

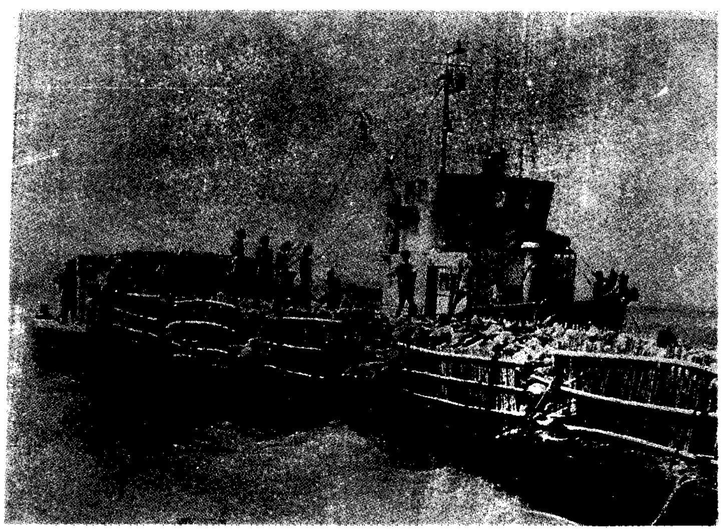


FIGURE 105 Close-up View of Stone Supplying Pontoon Dumping of Stones on Geomattress

view of partially sunk geomattress.

v. After the ballast beams had touched river bed and the sinking of stones on geomattress at rate of 300 kg/m^2 was completed, the ballast beams were retrieved. This was done by pulling out 20 mm ϕ rods, by tug boat and gradual lifting of ballast pipes, alongwith tow pipes with winches on each towing and mooring unit.

In this way, all the 31 geomattress panels were placed.

FIGURE 106 View of Partially Sunk Geomattress

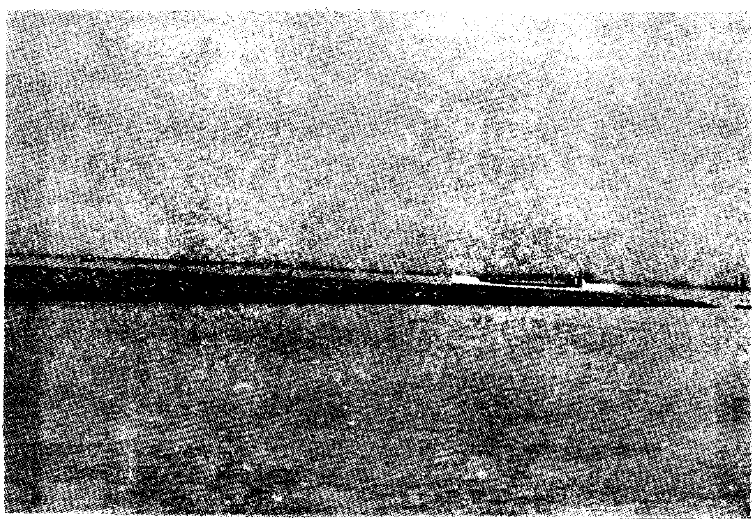


FIGURE 107 View of Completed Guidewall

Construction of Hammerhead

After completing the bottom protection works, the core of hammerhead was constructed by dumping laterite boulders and 10 to 30 kg stones, from loaded barges. The core had side slopes between 1V : 2.5H to 1V : 6H and a top level at +2.0 m level. The core was protected, by placing a 1 m thick layer of 30 – 60 kg quarry stone/boulders. The crest was at + 3.0 m level and had a width of 5 m.

Construction of Guide Wall

The guide wall from Ch 560 m to Ch 2600 m, which was partially constructed with bamboo cages, was completed by dumping and evenly spreading laterite boulders.

The guide wall had a crest width of 5.0 m and top at + 3.0 m level and side slopes of 1V : 2.5 H. Fig. 107 shows a completed view of guidewall.

After completion of guidewall, it has been observed that 40% of flood tide was passing through the portion between Nayachara Island and Haldia Port known as Haldia Channel and 60 % of flood tide was passing through Rangalfa Channel.

The uniqueness of this method was that, except sophisticated survey equipments and geofabric materials, most of the accessories, plant was locally developed by AFCONS. Maximum use was made of local materials and labour intensive techniques, which are more appropriate for developing country like India. The work of laying in position 24,000 m^2 of geomattresses and dumping of 4,50,000 tonnes of boulders, laterite stones etc. was completed in a period of 24 months, by June 1989.

Concluding Remarks

In this lecture, an attempt has been made to highlight some of the innovative measures adopted by AFCONS in execution of tunnels, bridges, slope stabilisation and special type of marine works. Similar contributions have also been made by other construction firms, which have not been mentioned here, because of space limitation.

Dr. Karl Terzaghi had said in his Presidential address delivered at the First International Conference on Soil Mechanics and Foundation Engineering (1936), 'Successful work in Soil Mechanics and Foundation Engineering requires not only a thorough grounding in theory, combined with an open eye for possible source of error, but also, an amount of observation and measurement in the field, far in excess of everything attempted by preceding generation of engineers. Hence, the centre of gravity of research has to shift from the theoretical study and the laboratory, into the construction camp, where it will remain'. These words of wisdom are perhaps more true today, than ever before.

India has undertaken an ambitious development programme to improve its basic infrastructure. To meet the demands of growing population and industry, many new types of structures will have to be constructed on available landmass. Some of these may be

- underground tunnels for transportation,
- underground tunnels for water supply,
- marine outfalls for sewage disposal,
- submarine pipelines for petroleum products,
- long offshore jetties for gases, fuels,
- underground powerhouses,
- road/railway bridges for transport.

Construction of these structures is going to pose challenging problems before the engineers. These challenging problems, though exciting, require concerted efforts on the part of the geotechnical engineers to undertake adequate investigation, collection of data, analysis of each problem, application of available means to derive an economical solution, on the basis of long experience and the intuitive judgement.

It is hoped that this presentation will help creating in fraternity of civil engineers an insight in handling such geotechnical problems and devising innovative techniques to their successful solutions economically.

Acknowledgement

The author thanks all his colleagues in AFCONS, who have been working in the execution of these projects.

The author is grateful to Mr.A.H.Divanji, Managing Director, AFCONS, for his kind permission to present this lecture. Grateful thanks are also due to various clients, who extended whole hearted cooperation in providing the necessary technical data.

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