Pamban Bridge : aspects of project management-I

S. A. Reddi

The 2,345-m long Pamban Bridge across the Palk Strait, which was formally opened to traffic by the Honourable Prime Minister of India on October 2, 1988, becomes the longest bridge built across the open sea in India. The construction of this bridge posed several challenges, which were successfully overcome. These are highlighted by the author in this paper, which is divided into two parts. The first part, presented here, deals with major construction aspects of the bridge. The second part, which will be presented in the subsequent issue of the Journal, will deal with the special provisions for durability, which became essential due to the location of the bridge in hostile environments.

The Pamban Bridge, connecting the Rameswaram Island with Mandapam on the mainland is 2345m in length, and spans the Palk Strait, which is subject to frequent cyclonic weather conditions, high waves and periodic change in direction of flow of water. The project, posing considerable challenge, is one of the most difficult bridge construction jobs undertaken in the country. This is also the longest bridge built across the open sea in India.

The road bridge runs parallel to the existing railway bridge for the greater part of its length. The railway bridge built about 70 years ago had been sited at the most favourable location with very shallow water and near outcrop of rock at majority of the foundation locations, except for the navigable span. Thus, the location of the road bridge is at best second to that of the railway bridge and posed greater challenges during construction.

The project got off to a start in 1974. However, the work was suspended in 1978 after having been exposed to a variety of adverse contractual and environmental conditions. By this time, a number of foundations and piers, as also the superstructure for the first 14 spans had been completed. However, the balance works were all located in the midstream and formed the most difficult part of the project. The balance works were put to tender in 1982 and the contract awarded to another agency, namely, Gammon India Limited, in February 1984. The project has been completed in September, 1988.

Salient details

The total bridge length of 2345m is divided into three parts:

1. non-navigable spans on Mandapam side - 61 percent of the length (27.127m spans)

2. navigable and anchor spans - 14 percent of the length (maximum span 115m)

3. non-navigable spans on Pamban side - 25 percent of the length (27.615m spans)

The navigable and anchor spans rest on four well foundations and reinforced concrete (RC) box piers. RC open foundations and trestle piers are provided for the remaining spans. The general arrangement of the bridge is shown in Figure 1.

The superstructure for a typical non-navigable span consists of four precast prestressed I-beams with the deck slab transversely prestressed, resting on neoprene bearings. The navigable and anchor spans consist of in -situ single cell box girders hinged at the centre, and provided with articulations at either end to receive the suspended spans.

Footpaths are provided at either end of the 7.5m carriageway to carry very heavy pedestrian traffic, designed for 500kg/m^2 load. RC railings are provided at the outer end of the footpath. The road kerbs have been strengthened to receive crash-barriers at a future date. Suitably designed sodium-vapour lighting is provided for the full length of the bridge. The wearing coat is of high-grade concrete, lightly reinforced.

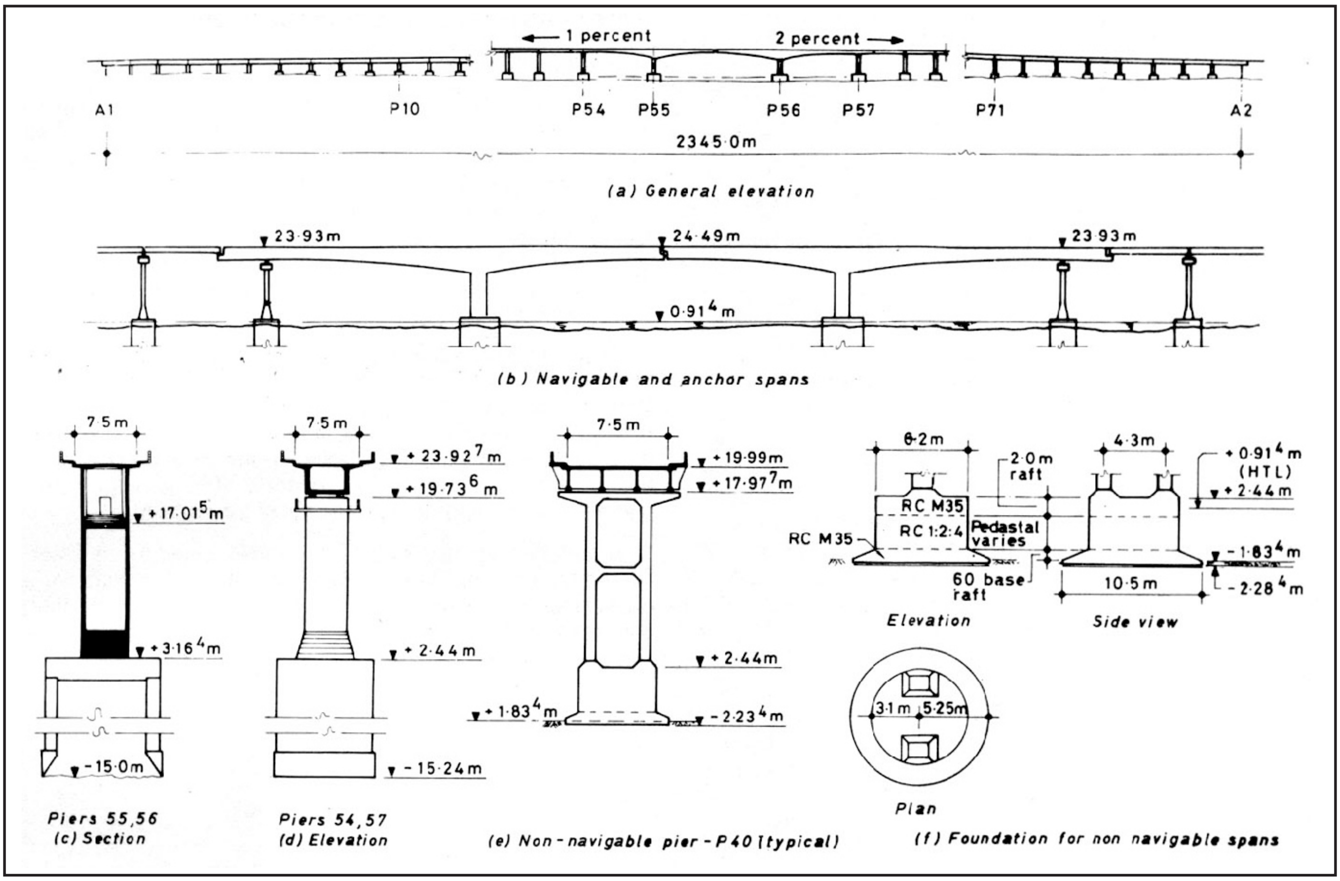


Figure 1. General arrangement of Pamban Bridge, which is the longest bridge built across open sea in India

Neoprene bearings have been provided for non-navigable span beams, except for the suspended spans resting on the articulations of navigable and anchor spans, where spherical TETRON bearings are provided. The deck rests on anchor piers of navigable span, separated by PTFE bearings. A cast steel central hinge connects the two mating cantilever arms of the 115m navigable span at the centre.

Special design provisions

The two-lane bridge is designed for the heaviest load envisaged in the IRC specifications. The design caters for two lanes of IRC Class 'A' loading with crowd load on footpath, or one lane of IRC Class '70 R' loading with crowd load on the footpath,. The design is also checked for crowd load of 500kg/m^2 for the full carriageway and footpath without other live load.

The bridge is in a grade of 1 percent towards Mandapam side and 2 percent towards Pamban side from the navigable span. The grade effect has been included even for level seating of the prestessed concrete beams over the neoprene bearings. Viaduct spans between piers 69 and 75 are horizontally curved with a radius of curvature of 275m. This has been provided with a view to facilitate crossing of the railway line between piers 73 and 74.

The wave effect is quite severe at the bridge location. A maximum wave height of 5.50m with a period of 9.5 seconds has been catered for in the design. The splash zone extends to about 6m above high-tide level. The wave height has been worked out on the basis of data obtained during the catastrophic tidal wave and cyclonic storm at the location in December, 1964, when the railway bridge was badly damaged and a number of girders of the railway bridge were torn out of the piers.

All non-navigable span piers are founded on rock at shallow depth. Circular concrete wells are provided for the navigable and anchor spans.

In order to provide uninterrupted movement of ships in the Pamban Strait, a central navigable span of 116m has been provided, with a vertical clearance of about 17m above hightide level. The deck levels at the end spans of the bridge are based on consideration of the height of splash zone. These criteria dictated the gradients provided on both sides of the navigable spans.

In order to ensure durability during the life of the structure, the following additional criteria have been enforced:

1. minimum thickness of any concrete component is 200mm

2. provision of increased cover to reinforcement (additional 25mm for the deck to 50mm in the tidal and splash zone)

3. elimination of grouping of prestressing cable ducts of the beams

4. box girders are designed to account for effects of thermal gradient of 10°C in the deck, in terms of BS 5400, Section 2

5. reinforcement for the box girder has been provided as per BS 5400

6. provision of massive foundations using high-grade concrete, Figure 1 (a).

The foundations are embedded at least 450mm to 500mm into the rock. All fissures in the rock are cement-grouted. A number of 36-mm diameter dowel bars of 2.4-m length are provided for each foundation. The top level of the top raft is kept above HFL. The construction joint at the interface between the foundation and the pier is, thus, always above HFL.

The substructure consists of reinforced concrete portal frames with two columns and a capping beam at the top, and brace beam at the middle. The minimum grade of concrete is M35.

The deck for superstructure of non-navigable spans consists of four precast prestressed beams of lengths ranging between 27m and 28m, with the deck transversely prestressed. Originally, the beams were planned to be placed in position side-by-side with a gap of 25mm to be filled with cement mortar before transverse prestressing, Figure 2 (a). The first 14 spans were accordingly constructed. The arrangement was subsequently revised as shown in Figure 2 (b) with in-situ gap slab of about 500mm. In order to minimise exposure to corrosion. no dowel bars were left projecting from the precast portion of the deck slab of the beam.

The interfaces were coated with epoxy prior to taking up the in-situ portion of the deck slab. The deck slabs of the first 14 spans completed with the original arrangement have been strengthened by providing 100-mm thick concrete overlay, suitably reinforced, in lieu of conventional wearing coat.

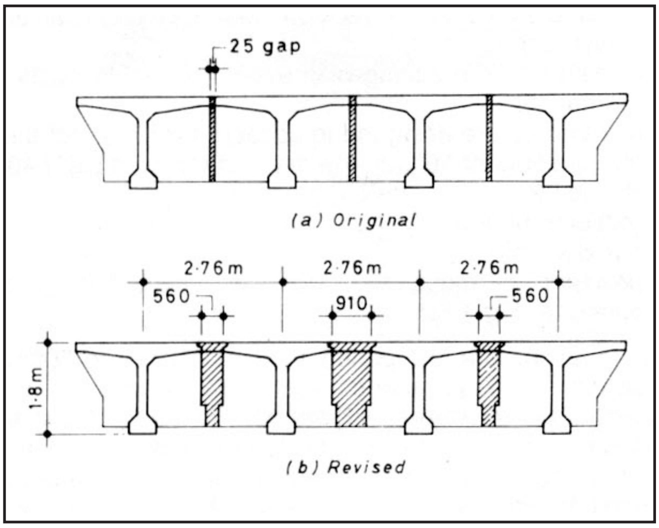


Figure 2. Assembly of precast beams

Neoprene bearings are provided for all the non-navigable and viaduct spans. The bearings are seated horizontally from the superstructure and with level contact with the soffit of the beam. During construction, it was noted that some of the bearings already installed for spans 1 to 14 by the earlier construction agency had shown distress, possibly due to the use of neoprene of questionable quality. The entire design was then re-examined. As a result, it was possible to reduce the thickness of the bearings from 106mm to 78mm with a view to provide better stability. The suspect bearings on spans 1 to 14 have all been replaced.

The navigable and anchor spans rest on four circular well foundations. Reinforced concrete box piers are monolithic with single-cell box girders cantilevering out of the two main piers Nos. 55 and 56. The cantilevers are connected by a central hinge between 55 and 56. At the other end, they continue over piers 54 and 57 where they are separated from the piers by suitable PTFE bearings.

The tips of the cantilevers facing piers 53 and 58 are articulated to receive the suspended spans of about 27m. Imported Rotoflon flange bearings, Figure 3, have been provided over pier Nos. 53 and 58 to cater to the three-way special movements at these locations.

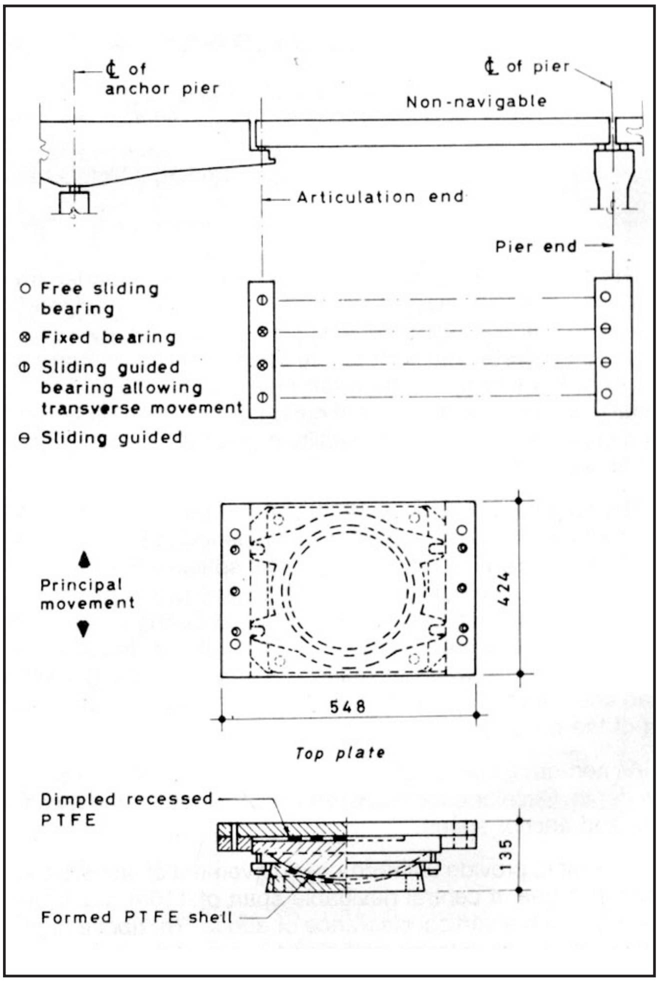
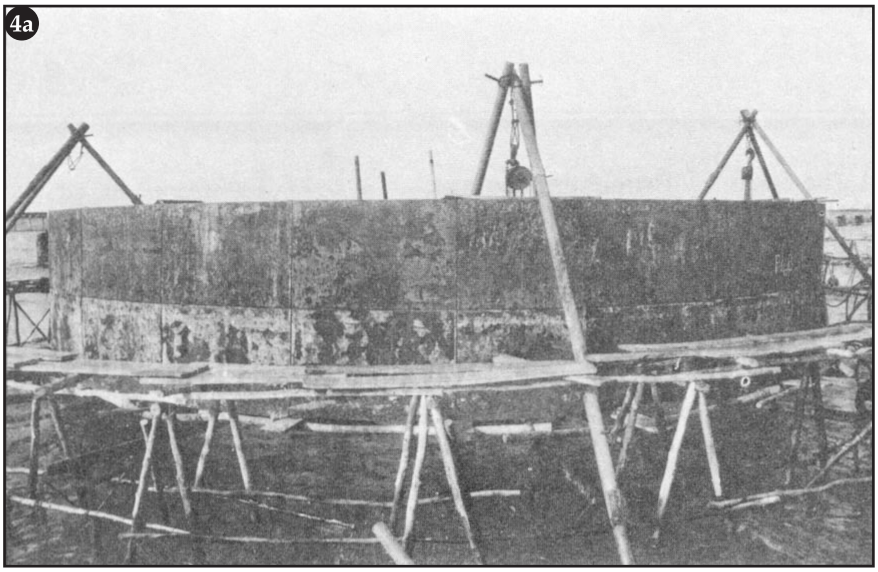
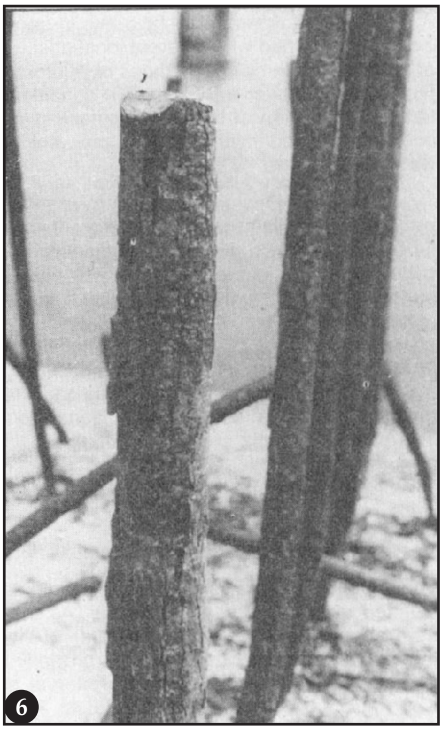
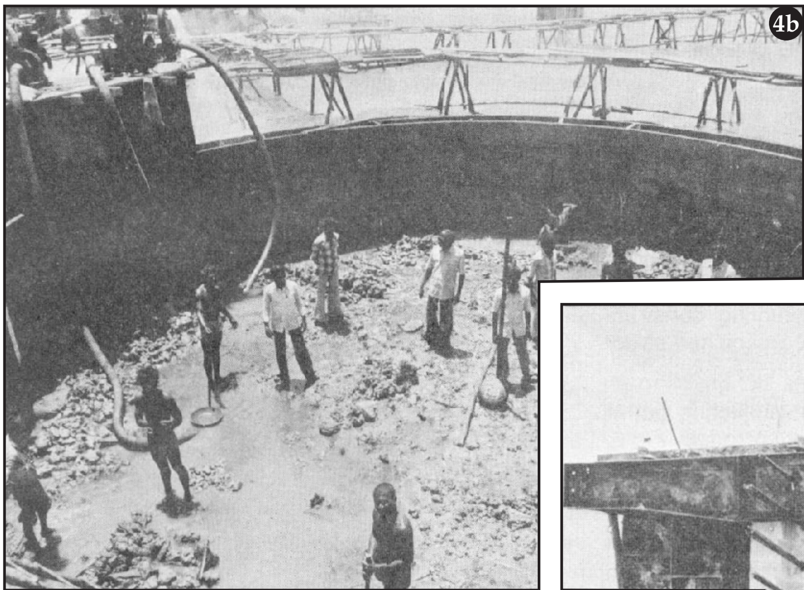
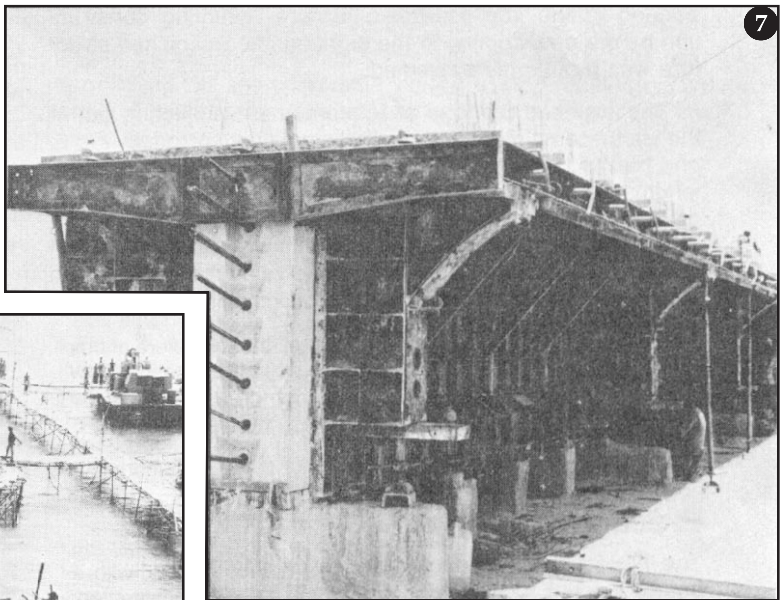


Figure 3. Rotoflon bearings









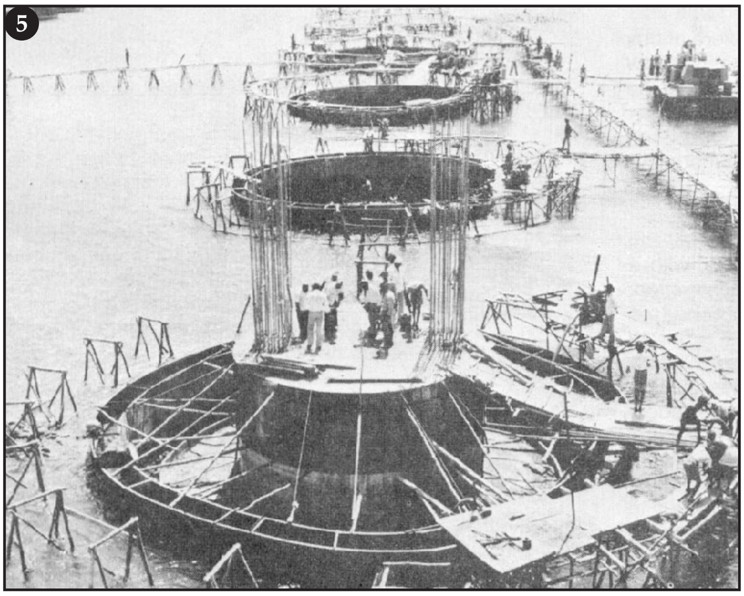


Figure 4. (a) Cellular steel cofferdam for foundation
Figure 4. (b) Foundation dewatered inside cofferdam
Figure 5. Completed non-navigable foundation
Figure 6. Corroded reinforcement
Figure 7. Casting bed of precast beam

Some aspects of construction

Foundations

All the non-navigable and viaduct span foundations consist of rafts, resting on rock at shallow depths. However, their construction proved most difficult, due to wind and wave forces and presence of sea water, though shallow. Special systems had to be devised for construction. Steel cofferdams were provided to facilitate dewatering and excavation of the bed at the foundation location to expose sound rock. The cofferdam was retained at each location till the foundation is concreted.

The cellular steel cofferdam, Figure 4 (a), is initially assembled above water level and suspended from wooden trestles. The bed rock is suitably trenched with the help of divers to receive the cofferdam. The cofferdam is then lowered on to the trench and the interface between the steel rings and the bed sealed with underwater concrete. The annular space was filled with sand up to water level for stability.

A battery of pumps was then commissioned for dewatering the foundation, Figure 4 (b). All loose soil and boulders were excavated and sound rocks exposed. Dowel bars were then driven into rock. A concrete levelling course of grade M20 was laid. The raft foundation is then constructed in the dry condition and the cellular cofferdam removed for use on the next foundation, Figure 5.

The system worked satisfactorily, in general. The pumping capacity varied with different locations; up to a maximum of 20 pumps with delivery ranges of 100mm to 150mm were provided. Wherever adverse wave conditions were encountered, stabilisation of the cofferdams was quite difficult. On occasions, the completed cofferdams were thrown out by the waves and had to be restored.

Well foundations are provided for navigable and anchor piers. The well diameters ranged from 9.4m for anchor piers 54 and 57 to 13.6m for the navigable piers 55 and 56. The construction was started by the first agency in the seventies but suspended halfway through. The partly-sunk wells were restarted after a lapse of a decade by the new construction agency. In view of the inordinate time interval during which the partly-cast steining was exposed to the fury of the sea, the wells after final sinking and plugging were filled with concrete for the full height as an additional durability measure. The bottom plug concrete was laid under 15m water using tremies. Heavily-reinforced well caps of thickness 2.25m and 1.526m were provided for the 13.6-m and 9.4-m diameter wells, respectively.

Substructure

The 74, H-shaped piers for non-navigable spans were constructed in lifts using steel shutters. As the heights varied from about 10m to 24m, the number of lifts varied to suit them. The construction joints were properly roughened and treated, ensuring as small a time interval as possible between successive lifts. Pier Nos. 26 to 32 were in various stages of unfinished construction when the work was suspended in the late seventies. Before resuming construction by the new agency in the eighties, the unfinished structure was thoroughly examined.

The exposed portions of reinforcement projecting out of the partly-completed piers were completely corroded, Figure 6, and had to be removed. In order to obtain the requisite lap lengths for the reinforcement, part of the already completed piers had to be demolished to expose the embedded reinforcement. Except for a shallow depth of 50mm to 100mm from the top surface, the embedded reinforcement was in excellent condition, thanks to the good quality of concrete.

The piers for the navigable and anchor spans were of box section profile, rendered solid for the splash zone and hollow above. The water-cement ratio was restricted to 0.38 and the pier section was heavily reinforced. As such, superplasticiser was used to improve the workability of concrete. The 24-m tall piers were concreted in nine lifts, using steel shuttering.

Precasting yard on Mandapam side

The project envisaged precasting and launching of all the beams except for the navigable and anchor spans, with a view to have better control over quality of construction.

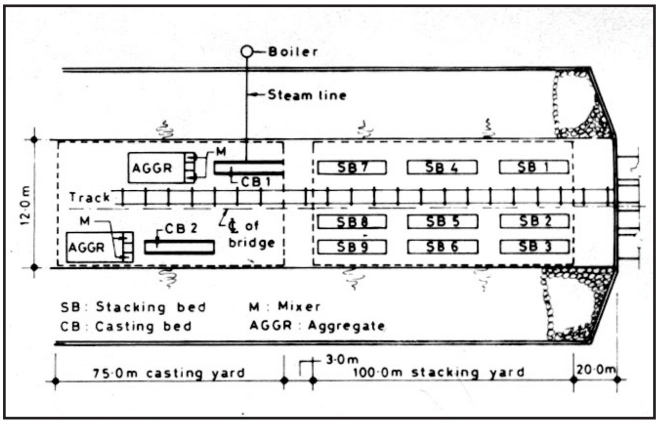


Figure 8. Mandapam side casting yard

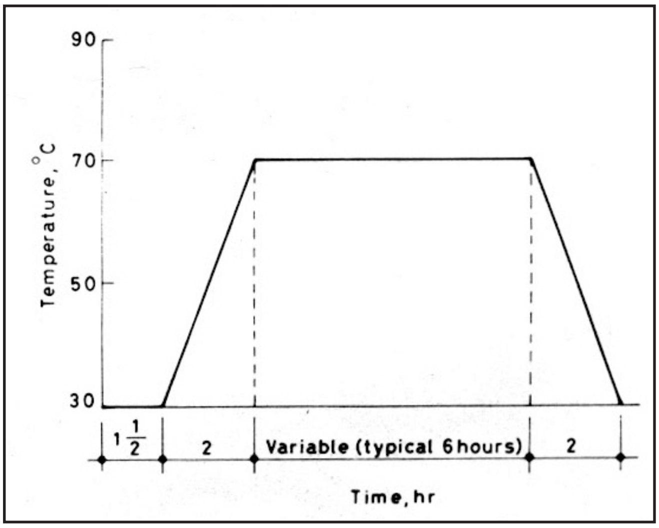


Figure 9. Typical steam-curing cycle

Accordingly, a central precasting yard was established on Mandapam side. Earlier, this was of a conventional type with a number of precasting beds and side shutters originally established by the first construction agency. This system was streamlined with a view to further improve the production and comply with the durability criteria. When the second agency started work in 1984, the number of beds had been drastically reduced to only two with 1 ½ sets of side shutters. The design of the side shutters was also streamlined to ensure that one side at the far end is hinged to the soffit. Thus, only the non-hinged side had to be completely removed and re-erected after each beam was cast.

Precasting beams

The casting and stacking yard on Mandapam side was located on approach embankment at the road level immediately behind the abutment, Figure 7. Only a 12-m width of the approach embankment was available for locating the casting and stacking beds as well as the concrete mixers and stacking space for aggregates and cement. The available space was judiciously used to obtain the layout for two precasting beds and eight stacking beds. The stacking beds were located closest to the abutment followed by casting beds and the material stacking yard in the rear. Thus, the entire precasting activity was confined to an area of about 70m x 12m. Such a compact layout has resulted in. more efficient and controlled precasting activities in addition to saving on infrastructure items such as water, power supply lines, etc.

A coal-fired boiler was provided to generate steam for steam curing. Due to lack of space on the embankment, the boiler was located at the foot of the embankment opposite the precasting bed. The layout was so designed as to minimise the length of steam pipelines. Figure 8 shows details of the layout of the precasting yard.

The steam-curing cycle was very carefully designed to facilitate accelerated development of concrete strength during the first 24 hours. A typical steam-curing cycle is given in Figure 9. With the use of this cycle. It has been possible to attain a cube strength of 30 to 35MPa at the end of the steamcuring cycle, i.e., about 12 hours after concreting. It is worth noting that the presteaming period is limited to between 1 to 1 ½ hours only as against the conventional time of 4 to 5 hours. The presteaming period has been designed after determining the initial setting time of concrete, which under site ambient conditions ranges between 60 to 80 minutes for the high-strength OPC used at Pamban Bridge. The injection of steam was started immediately after the initial setting of concrete.

Control cubes were exposed to actual on-site steamcuring conditions identical to those of the beams and were regularly tested for strength prior to commencement of prestressing operations. Tarpaulins supported on simple framework were used to contain the steam around the precast beams for the necessary curing-cycle time, Figure 10. Though this simple arrangement resulted in leakage of steam at a few places, the overall economies were in favour of such an arrangement.

In order to cut down the time cycle of precasting beams, the non-tensioned reinforcement steel was prefabricated into a cage as a parallel activity on a temporary stand directly in front of the casting bed. The prefabricated cage was rolled in position as soon as the casting bed was cleared of the beam already cast. The prefabricated cages are normally handled by cranes, Figure 11. However, as no crane was available at Pamban site, these were rolled into position manually using M.S. bars as rollers.

The precasting of beams was initially started with conventional method of bending and tying non-tensioned steel and water curing. The prefabrication of cages and steamcuring arrangements were progressively introduced in stages. With these refinements, it has been possible to produce 12 beams per month or one beam every alternate working day, ultilising only two casting beds and 1 ½ sets of shutters. Originally, only one bed and one set of shutters were planned for this output. However, the marginal increase was necessitated due to resistance of staff and workers to any change in the conventional system of working.

The Mandapam side precasting yard catered to the requirement of spans 1 to 53 (212 beams). The precasting yard arrangement was then shifted to the top of the deck near span No. 40, Figure 12, for precasting the 8 beams required for spans 54 and 58. The beams required for spans between pier 58 and Pamban side abutments (84 Nos.) were precast on tubular steel scaffolding system at the respective span locations and side shifted into position. As the spans between 58 and 68 were permanently under water, special foundation was provided to support the scaffolding system.

The coarse aggregates were supplied from quarries at a distance of 150km from Mandapam side. The sand was from Vaigai River at a lead of about 80km from Mandapam. Water for mixing and curing of concrete had to be obtained from limited sources (wells) at a distance of about 5km from either end of the bridge as the water available at the bridge site was not fit for mixing and curing concrete. The chloride content of the water as also the pH value were regularly monitored for compliance with specifications. The construction activity had to be slowed down on several occasions due to non-availability of water complying with chloride-content limitations.

Launching of beams

The precast beams for the non-navigable spans weighed about 80t each. After casting a typical beam on the casting bed, steam curing, and prestressing the first stage cables within 24 hours, the beam was side shifted on to a parallel track at the centre of alignment, pulled forward towards the bridge and then again side shifted on to the stacking beds. These operations were carried out manually, except for the use of hydraulic jacks for lifting and lowering of beams. After the second-stage prestressing and grouting of the ducts in the stacking bed, the beam is again side shifted on to the central track and moved forward to the abutment position. At this stage, the beam is transferred on to a wider rail track (gauge width 5.5m) resting on the first and third beams of the prestressed deck. The beam

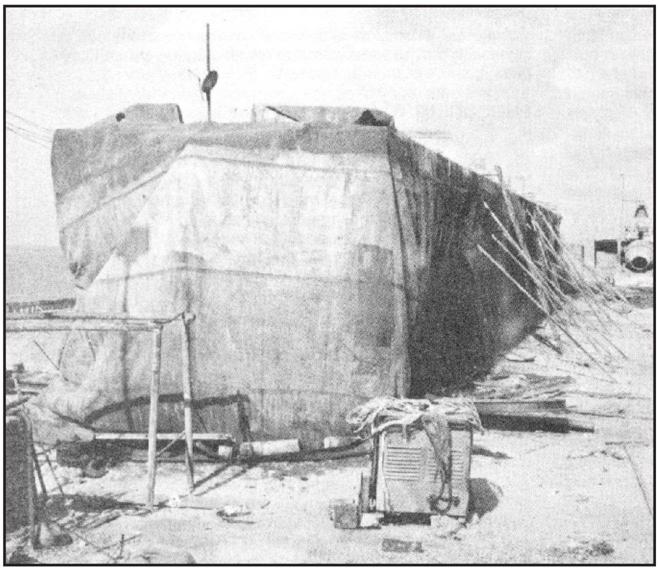


Figure 10. Tarpaulin hood for steam curing

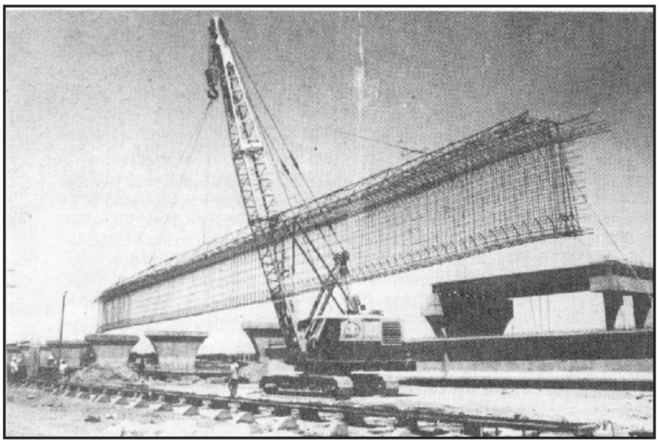


Figure 11. Prefabricated reinforcement cage

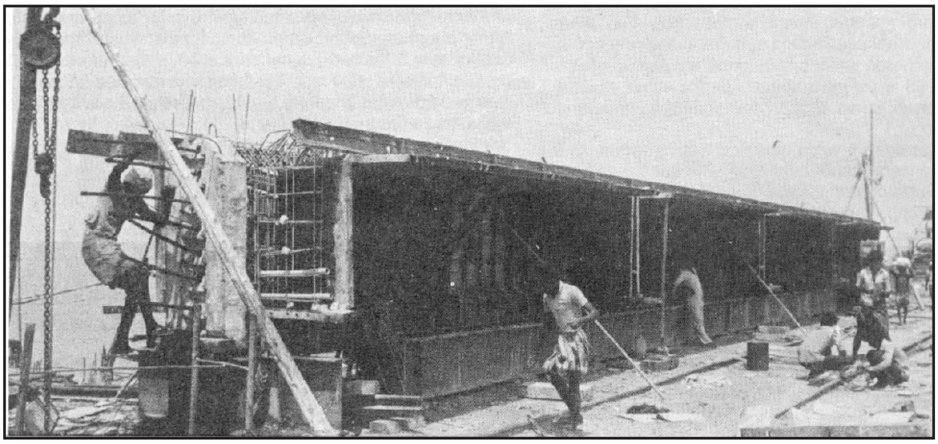


Figure 12. Casting bed on the completed deck

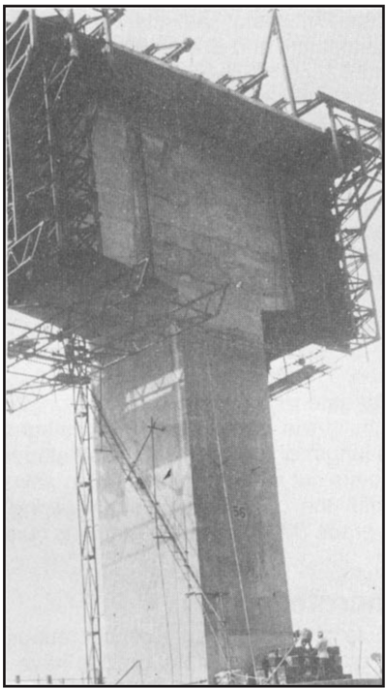


Figure 18. Cantilever gantry fixed on pierhead

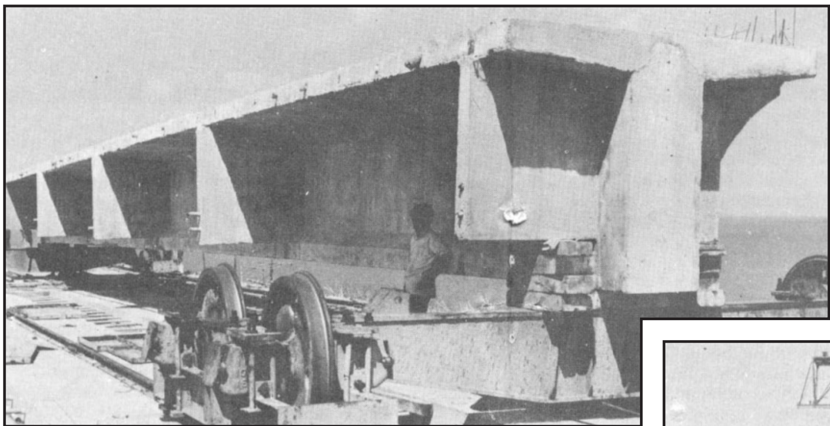


Figure 13. Hauling of precast beam over deck

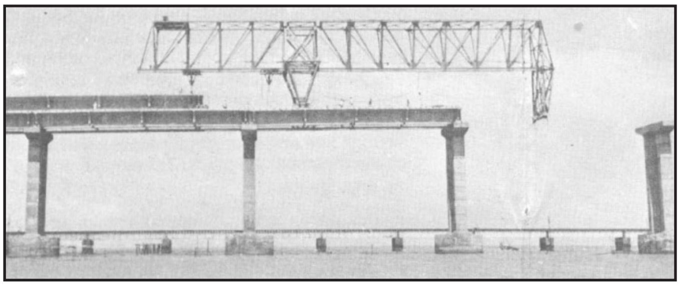


Figure 14. Auto-launching of truss

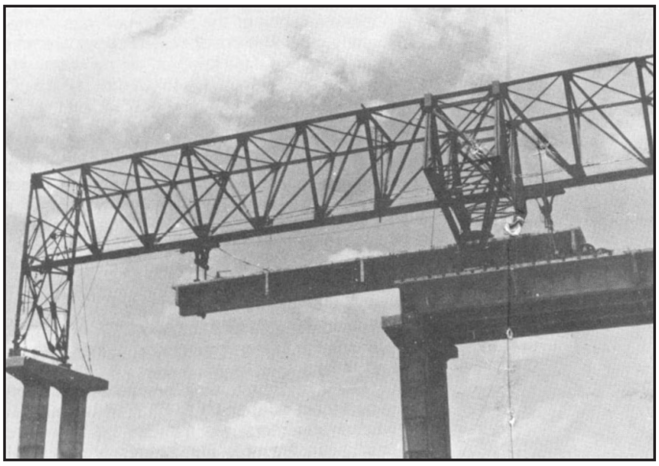


Figure 15. Launching of precast beam

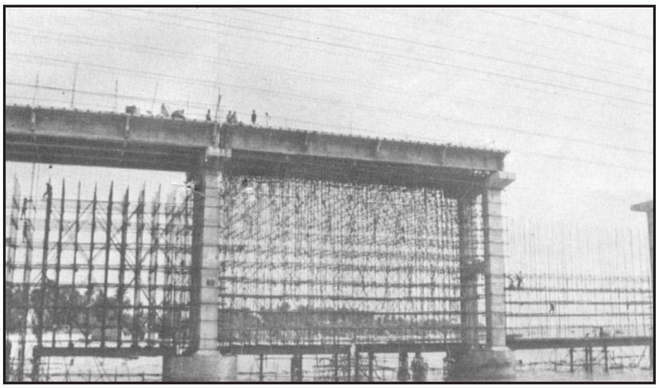


Figure 16. Tubular staging for Pamban-side beam precasting

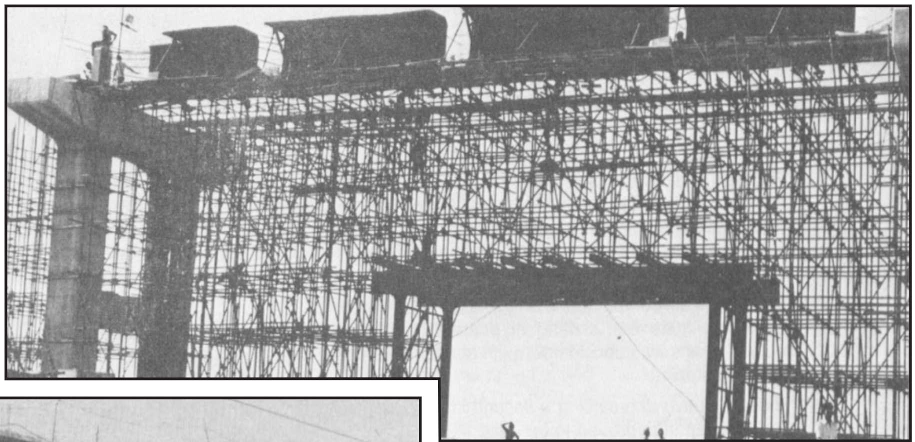


Figure 17. Tubular staging for Pamban-side beam precasting over railway crossing

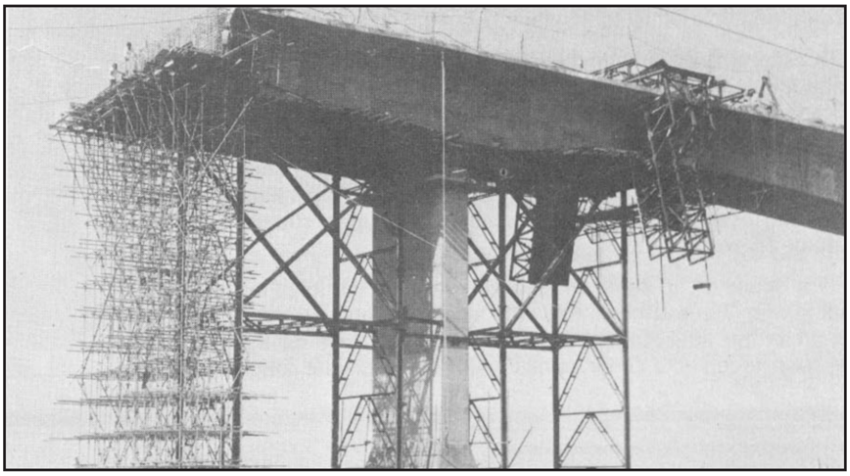


Figure 20. Steel staging for deck over anchor span

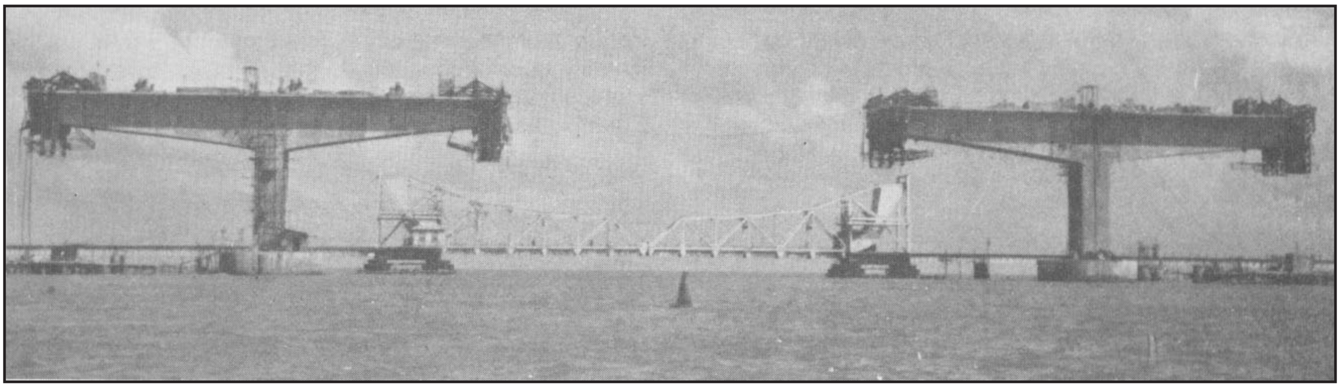


Figure 19. Span 55 and 56 in progress

is then pulled forward up the gradient using power winch for being fed to the launching truss, Figure 13.

The launching truss, Figure 14, has been specially designed and fabricated for the bridge. The truss has got a triangular profile and is provided with a central and a front trestle. The truss itself is first auto-launched across the span. During auto-launching of the truss, one of the precast beams is hooked on to the rear to serve as a counterweight. Once the truss is launched, the front trestle rests on the forward pier. The front end of the beam to be launched is then pulled forward with the front end suspended from the truss and the rear end still supported on rail-mounted bogies. As the front end of the beam advances sufficiently, the rear end is also hooked up on the underside of the launching truss. The beam is then pulled forward further and lowered on to the pier cap with the help of sand jacks, Figure 15.

The beam is thereafter side shifted to the final location and placed on the bearings. This process is repeated till all the four beams are placed in position to complete one span. The launching truss is then moved forward to the next span. This process of launching the truss and the four beams for one span is generally achieved in a period of 7 to 10 days. Thus, during the peak construction period, it was possible to launch 12 beams over three spans every month.

This launching speed corresponds with the speed of production of beams in the precasting yard. With this synchronisation of precasting and launching activities, there was no need for storage of a large number of beams in the stacking yard awaiting launching. Prolonged storage of beams before launching also results in excessive upward hogging. This problem has also been overcome by the system adopted. After launching four beams in a typical span, the crosscables are threaded, gaps shuttered and concreted. The cross-cables of 12/7 configuration are spaced at approximately one-metre intervals and the prestressing is carried out within three days of concreting.

The precasting yard on Mandapam side and the launching truss were used only for the first 54 spans. Separate staging arrangement was provided on Pamban side for precasting the beams required for non-navigable and viaduct spans, Figure 16. The following considerations dictated such parallel arrangements:

1. the construction schedule envisaged parallel activities on both Mandapam and Pamban side

2. the launching truss on Mandapam side could service Pamban side spans only after the navigable spans are completed. The navigable spans were most difficult to construct and, as such, were not scheduled to be completed in time for facilitating the use of launching truss on Pamban side

3. there were six curved spans on Pamban side. Use of launching truss and shifting of precast beams for curved spans was considered quite complicated and time-consuming

4. the alternative of providing a second launching truss for Pamban side proved to be quite expensive compared with the steel-staging arrangement.

The scheme of precasting the Pamban side beams on staging did not lend itself for prestressing from both the ends due to constraints of space required for accommodating the prestressing jacks. As such, these beams were designed to be prestressed from one end (forward) only. Though there was initial reservation for one-end stressing of all the cables, this was overcome by suitable design checks. During actual execution, such one-end stressing proved to be more effective than the conventional two-end stressing and no difficulties were experienced in realising the stress levels as envisaged in the designs.

The span between beams 73 and 74 crosses the railway track. The construction of these spans had to be specially planned to conform to the railway regulations. The staging. system was specially designed to facilitate construction of the superstructure including precasting the beams without stoppage of railway traffic underneath, Figure 17. The staging system was required to be approved by the Regional Commissioner of Railway Safety prior to execution, and the procedure involved considerable delay.

Navigable and anchor spans

The spans are cast in-situ. The pierheads at 55 and 56 are cast monolithic with the box girder. The pierhead, 4m along the length of the bridge plus 1.8m of box decking on either side,

Table 1. Schedule of concrete grades

Component	Grade of concrete			
	1:2:4	M35	M43.5	M45
Well foundations				
Curb		*		
Steining	*			
B. Plug	*			
Filling	*			
Well cap		*		
Open foundations				
Mudmat	*			
Pedestal	*			
Raft		*		
Piers		*		
Superstructure				
I-Beams (Mandapam)			*	
I-Beams (Pamban)				*
Box beam				*
Finishing items				
Kerbs		*		
Footpath		*		
Wearing Coat		*		
Railing		*		

was cast on brackets fixed to the top of the piers. In view of the large depth (7.225m) of box girder at the pierhead location, the webs were concreted in stages. The concrete is mixed on barges below and hoisted for placement.

The first cantilever gantry is then mounted on the pierhead and the first segment of 3m concreted. The gantry is then moved forward to make room for the second gantry to be fixed, Fig 18. A segment of 3m was cast on the opposite side. Now, the pair of gantries was operational on each span to ensure casting of segment of 3m on either side of the pier with a time cycle of 10 to 15 days, Figure 19.

The box girder is of variable depth. As the depth reduced to about 6m, after a few pairs of segments, it was possible to concrete the full section of the box girder in one operation without construction joints. As each pair of segments was concreted, a few cables were stressed to transfer the dead weight of the segment to the piers. Two pairs of gantries were used for simultaneous construction of the superstructure of spans 55 and 56.

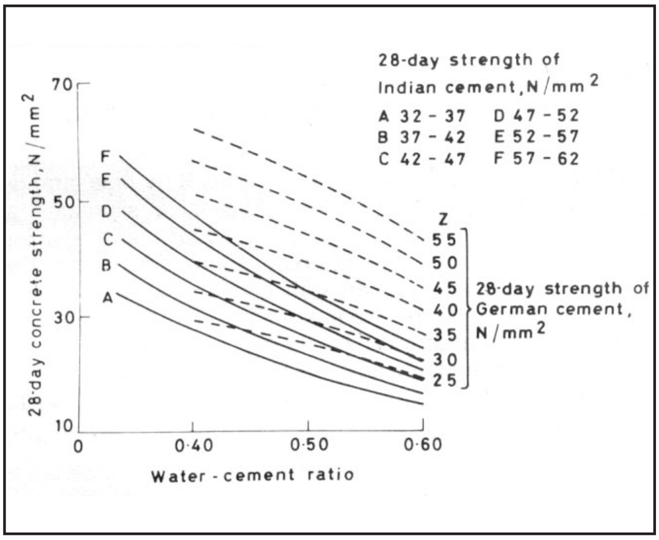


Figure 21. Concrete strength is a function of water-cement ratio and type of cement

Figure 22. Reinforcement congestion in deck slab

The spans over piers 54 and 57 were constructed on staging, Figure 20, and continuity with spans 55 and 56 established in the final stages. In order to compensate for the long-term creep and other stress-loss effects on the longitudinal profile of the deck road surface, extensive camber calculations were made and proof-checked by the Indian Institute of Technology, Madras. Precambers were accordingly provided during construction of each pair of segments. Over the passage of time, it is expected that the deck will settle down to theoretical profile after undergoing time dependent losses of stress.

The tips of the mating cantilevers between piers 55 and 56 were finally joined with the help of specially-designed cast steel hinges. A finger-type steel expansion joint, duly protected against corrosion by epoxy coatings, was provided for the deck at the hinge location and at the articulation locations beyond piers 54 and 57.

Finishing items

From Mandapam side abutment to navigable span, the various finishing items such as kerbs, footpath, wearing coat, railing, etc., could be taken up only after all these spans were launched as the precast beams had to be transported on the completed deck earlier. However, on the Pamban side, the finishing activities could be taken up earlier.

The footpath and railing kerbs were cast in-situ, while components for the footpath and railing were precast and assembled in position. The concrete wearing coat was progressively laid in alternate panels with a central longitudinal joint along the length of the bridge. In the last stages, the wearing coat for full length of the span was concreted in one operation and the joints cut with concrete-cutting saw. This was done to expedite the construction. The wearing coat was laid using high-grade (M35) concrete that was cured for 28 days.

Grades of concrete

Table 1 gives schedule of various grades of concrete used in realising the bridge project. The various grades have been decided upon after taking into account the requirements of structural strength as well as durability. Lower grades of concrete have been used for components which are permanently submerged under water. For all other components which are above low-tide level and in the splash zone, M35 grade has been adopted.

The concrete in the foundations and the substructure is exposed to water and soil containing substantial amounts of sulphates. For this part of the concrete, sulphate-resisting Portland cement has been used with tricalcium aluminate limited to about 5 percent. As there is no Indian Standard as yet for sulphate-resisting Portland cement, corresponding ASTM specifications have been used for guidance in procurement and evaluation of cement.

For the concrete in superstructure, high-strength ordinary Portland cement conforming to IS:8112 has been used. In this context, it may be noted that high-strength concrete cannot be produced from low-strength cement. A definite correlation exists between strength of cement and that of concrete made therefrom, Figure 21. Recognising this factor, the contract documents stipulated the issue and use of highstrength ordinary Portland cement for higher grades of concrete in the superstructure.

Considering the overall importance of the structure in general and the durability requirements in particular, a full-fledged concrete testing laboratory was established at the project site. The mechanical properties of the cement and aggregates, as well as the slump and cube-crushing strength of concrete, were regularly monitored at site. The site facilities also included provision for frequent testing of the pH values of water as well as the chloride content in the water and aggregates.

All the compressive strength results were statistically evaluated for acceptance of the concrete. Trial mixes were also based on use of low values of standard deviation based on earlier experience on similar projects carried out elsewhere by the constructor. Long-term analyses of the cube strength for M45 grade of concrete indicate the standard deviation in the range of 3MPa to 4MPa. This degree of control has resulted in optimising the design of the mix. The cement content for M45 grade was limited to between 450kg/m^3 and 500kg/m^3.

Use of admixtures

Admixtures have been extensively used as workability agents for concrete used in the deck. Being high-strength concrete, by the very nature of the mix design the resultant workability is rather low, the slump ranging from 10mm to 20mm due to limitation of water-cement ratio and cement content. Superplasticisers have been used in order to improve the workability and not for reducing the cement content, as the mix has already been designed for optimum cement consumption.

Extensive preliminary trials have been conducted with various grades of superplasticisers available in the market before selection. The trials included workability and strength of concrete with various dosages and also study of the rate of loss of workability with time. The rate of loss of workability also depends on the type of cement used. As such, trials were repeated wherever there was a change in the source of cement.

In order to limit the chloride content in the concrete, it was decided to use only those admixtures which are totally free from chloride. Apart from studying the manufacturers' chemical analysis reports, independent verification has also been obtained through a government laboratory.

A total quantity of about 22000 litres of admixtures has been used. The cost of admixtures work out to Rs. 32/m^3 of concrete at 1987 prices. Site experience indicates that there has been substantial improvement in the work ability of concrete with the addition of superplasticisers. However, such improvement in work ability was not directly reflected in higher slump values. Whereas the increase in slump with various dosages of admixtures has been found to be only marginal for the high-

Figure 23. Protective coating to reinforcement

grade concrete under high ambient temperatures, the vibration of concrete using super-plasticisers was found to be more effective and the resultant surface finish was far superior.

Some portions of the deck slab are also provided with highly congested reinforcement, Figure 22; but for the use of super-plasticisers it would have been virtually impossible to effectively place the concrete in such areas.

Treatment of reinforcement

Only mild steel bars were used as reinforcement for all the RC components of the structure and for non-tensioned reinforcement of the prestressed beams. M.S. bars are considered more corrosion-resistant compared with high-yield deformed bars. Even mild steel bars exposed to highly aggressive atmosphere at Pamban get corroded very fast. Such a phenomenon takes place even during the limited interval of time between bending and tying of steel and concreting. In order to minimise the effects of corrosion during storage, the reinforcement bars were stored at a central yard at Ramanathapuram at a distance of about 40km from site. The steel was drawn in small quantities as and when required.

In order to provide protection against corrosion during the interval between bending, tying the steel and concreting, all the mild steel bars were given a special protective coating as per the specification developed by Central Electrochemical Research Institute (CECRI), Karaikudi.

Table 2. Relaxation test results of high-tensile steel

Sample	:	1	2	3	4
Specimen type	:	7- ply	strand	12.7-mm	dia
Nominal area, mm^2	:	98.7	98.7	98.7	98.7
Applied load. N	:	128597	128597	128597	128597
Loss in stress at 100 hrs., N/mm^2	:	28.7	35.8	40.0	33.29
Percent loss in stress	:	2.13	2.75	3.07	2.55
Loss in stress at 1000 hrs, N/mm^2	:	46.8	53.36	57.98	50.53
Percent loss in stress due to relaxation at 1000 hrs	:	3.6	4.09	4.45	3.88

The bars after being bent to shape are first derusted using a derusting solution. The bars are then cleaned using cleaning powder and water mixed with sodium bicarbonate and wiped with cotton waste. The patented phosphatic jelly is then applied to the surface of the bars. This is followed by application of a cement-based inhibitor solution by brushes. After 12 to 24 hours of air-curing, a sealing solution is applied. This process is repeated to provide a second coating, Figure 23.

The treated rods were required to be used within 30 days. These rods were also required to be handled very carefully and rough handling was avoided to ensure that the coating did not peel off during handling and fixing. The treatment proved very effective in preventing corrosion of reinforcement until the bars are embedded in concrete.

Prestressing

Freyssinet system of prestressing was adopted for the bridge deck. The navigable span and anchor spans were prestressed using cables consisting of 12 Nos. 12.7-mm strands each. Corrugated galvanised-steel ducts of 70-mm diameter were provided for these cables. The ducts were manufactured at site, using a special machine in order to ensure use of fresh ducts without getting corroded. The ducts were made of extra thick material (thickness ranging from 0.28mm to 0.3mm) as an added protection against corrosion.

The prestressing strands were monitored regularly for the various properties. Apart from the manufacturer's batch test certificates for ultimate tensile strength, stress-strain relation as well as E values, random, checks were independently carried out at the site for these properties. Intial tests for ultimate tensile strength of the strands conducted at an independent laboratory in Madras yielded erratic results. On investigation along with the manufacturer's representative, it was discovered that improved gripping devices were required for testing strands. After these were provided, the results were quite consistent. Unfortunately, there are no Indian Standard specifications for methods of testing strands, use of standard grips, etc.

Relaxation tests at 1,000 hours were regularly insisted upon. This is not normally done for less important jobs. Typical 1,000-hour relaxation values obtained during the tests are given in Table 2.

For non-navigable and viaduct spans, cables consisting of 12 Nos. 7-mm diameter H.T. wires were used. A large quantity of H.T. steel was already in the possession of the owners as they were procured a few years earlier. In view of the prolonged storage of the wires, independent tests were repeated for various properties of the H.T. steel and confirmatory results obtained prior to their use on the bridge.

The prestressing operations were carried out successfully without any major problems. The 12/7 cables were all stressed to the desired pressures and elongations without any difficulty. There has been no case of any breakage of wires. Some difficulties were experienced during stressing of the

Table 3. Grout pump specifications

Nominal output	:	1200 1/hr
Working pressure	:	3 MPa
Electric motor	:	2 hp
Weight	:	195 kg
Type of pump	:	Positive displacement, reciprocating type

Table 4. Grouting record

Job name:...	
Span No:	Cable No ....
Date of cable installation:	Date of grouting ....
Type of cement: OPC/HSOPC	
W/C ratio:	
Temperature: Mixing water	Grout ....
Time : Start	Finish ....
Equipment: Grout mixer	Grout pump ....
Cable duct: Diameter	Length ....
Volume of grout in litres	Regrouting ....
Grouting pressure	
Cement consumption Theoretical	Actual ....
Pre-grouting checks:	
Free of blockage: Inlet : Yes/No	Outlet : Yes/No
Vents : Yes/No	Cable duct : Yes/ No
Leakage observed Yes/No	Sealed : Yes/No
If cable duct blocked: Remedial measures	....
Grouting observations:	
Passage of grout through vents	: Yes/No
Passage of grout through outlet	: Yes/No
Any equipment failure :	
Remarks:	

12/13 strand cables in the formative stages. Problems encountered included breakage of strand wires, shortfall in elongation, excessive slippage at anchorages, etc. These problems were encountered during the initial stages of operation, when the operators were in the learning stage. With sufficient training and experience these problems have been totally overcome. Use of strand cables for prestressing bridge decks have been rather limited so far in India.

The broken strands have been replaced by new strands before prestressing and locking. Excessive slippages at anchorages were brought under control by extensive cleaning of strands and wedges before prestressing and locking. The strands were initially coated with cement slurry to protect against corrosion during the interval between threading and stressing. The strands could not be effectively cleaned prior to stressing. In consequence, gripping of wedges was not fully effective, leading to excessive slippages. Having identified the reasons, coating of cement slurry was dispensed with.

Though provision was made in design for dummy cables to compensate for shortfall in extensions, there was no occassion for use of such dummy cables as in practice there were no shortfalls. All such dummy cables for non-navigable and viaduct spans were withdrawn and the empty ducts grouted and finished. In the case of navigable spans also, there were no shortfalls to be made good. However, the dummy cables were also stressed at the last stages. This was done with a view to providing some additional level of prestressing to compensate for the unanticipated losses of prestressing progressively at later stages.

Grouting of cable ducts

The Pamban Bridge is exposed to highly corrosive environmental conditions. In terms of IS:4180, the corrosion of mild steel at Pamban, where the structure is fully exposed to the sea, is 0.395mm per year. Corresponding figure for Bombay is 0.0787. Thus, the structure at Pamban is likely to be exposed to about five times as much corrosion compared to similar structures at Bombay.

Considering the difficult environmental conditions, a conscious decision was taken to ensure very effective grouting of the cable ducts. For all beams of non-navigable spans, it was decided that all the cable ducts will be grouted within 10 days of concreting the beams. However, this was not possible in the case of cables of the box girders in the navigable span due to the sequential nature of cantilever construction.

For the non-navigable span beams, the design and construction methods were suitably modified to ensure that the grouting is carried out effectively within the stipulated time. The design modifications included reduction in number of stages of prestressing to two and redispositon of cable ducts to avoid grouping.

As per original designs, prestressing was to be carried out in two stages involving a minimum time interval of 28 days before grouting could be taken up. In the revised scheme, the time interval was reduced to about 8 days by streamlining the casting-yard design and by the use of steam curing to facilitate very early stressing. By steam curing the precast beams, it has been possible to prestress the first- stage cables necessary to carry the self-weight of the beam within 24 hours of casting. The beams were shifted to stacking yard the next day. The second-stage cables were then prestressed within the next 5 or 6 days. Thereafter, all the cables were grouted in the stacking yard itself, limiting the interval between casting and grouting to generally between 7 to 8 days. The beams were then shifted for launching.

Strict quality assurance was ensured for the preparation and injection of the grout. The water-cement ratio was limited to 0.40. Initial attempts at grouting using such a low water cement ratio were not quite successful, due to high ambient temperature conditions prevailing at the bridge site. After initial trials, attempts were made to lower the temperature of the grout. This was realised by using chilled water for mixing the grout, limiting the temperature of water to between 10°C to 15°C. This resulted in lowering the grout temperature to around 25°C which has, in turn, improved the flow characteristics of the grout.

The containers handling the grout were suitably insulated to avoid rise in temperature of the grout during the operations. The grout was mixed using high-speed stirrers and inlected into the ducts with the help of an electrically operated reciprocating pump. The pump specification is given in Table 3.

The ducts were successively flushed with water and compressed air to remove any loose particles which were deposited inside. Such water-flushing also ensures lowering of the temperature of the walls of the duct. Grout is then injected at a pressure of about 3MPa. The injection was always carried out from one end and continued till the grout of the same consistency emerged from the far end.

With these precautions and improvisations, it has been possible to effectively grout the cables using the very low water-cement ratio of 0.40. No admixtures were used for preparation of the grout, considering the fact that the grout, unlike the concrete, is in direct contact with the high tensile wires and even small quantities of impurities such as chlorides present in the admixtures may not be tolerated. Hence, it is desirable not to use admixtures for the grout.

Grouting of cable ducts of the cantilever box section for the navigable and anchor spans was taken up only after completion of the entire span. This precaution was necessary to avoid interconnected leakage of grout between various cable ducts if carried out at intermediate stages. In view of the large volume of grout involved in grouting the long-span cable ducts of the navigable span, the equipment used for mixing and injecting the grout had to be suitably upgraded.

The grout was mixed in large quantities using colcrete mixers. While medium-length cable ducts were handled using the electrically-operated reciprocating pumps, grouting of longlength cable ducts was carried out with the help of Colmono pumps. With other precautions similar to those detailed for non-navigable spans, it has been possible to grout even the very long cables (126m) using a low water-cement ratio of 0.40. In the case of cantilever spans, injection of grout was taken up simultaneously from both the ends and continued till all the intermediate air vents were monitored for grout flow and plugged. All the grouting activities were very carefully documented and analysed for effectiveness of grouting in each case.

Table 4 gives a typical proforma of the documentation adopted.

(to be Continued)

S.A. Reddi, Chief Engineer (Projects), Gammon India Limited, Bombay 400 025.

(Source: ICJ November 1988, Vol. 62, No. 11, pp. 568-579)