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Remedial Underpinning for Foundation of a Steel Tank—Construction Details

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Construction details of various operations involved in the remedial underpinning of the foundation of a steel tank for storing liquid ammonia are furnished. The tank, about 42 m in diameter and 17.5 m high, was built on a flexible RCC raft (tank bottom slab) supported by driven cast-in-situ concrete piles. During water load test, the tank foundation settled excessively and the bottom slab acquired a saucer shape configuration. A remedial underpinning scheme was worked out on the principle of 'stress relief through excavation' involving (a) construction of a water tight rigid basement, (b) lifting of excessively deformed tank bottom slab back to its original configuration and strengthening, and (c) repair of piles exposed during basement. The paper brings out the various operations implemented successfully against a tight time target of seven months. The satisfactory performance of foundation during water load test after repairs and subsequent fillings with ammonia has proved the soundness of the underpinning operation.

INTRODUCTION

The tank which has a capacity of 10 000 t (11 600 t with overpressure) for storing ammonia is located in the Willingdon Island, Cochin (Fig 1). The island is generally made up of soft clayey silt-sand mixtures dredged from sea bottom. The subsoil strata at the tank site is given in Fig 2.

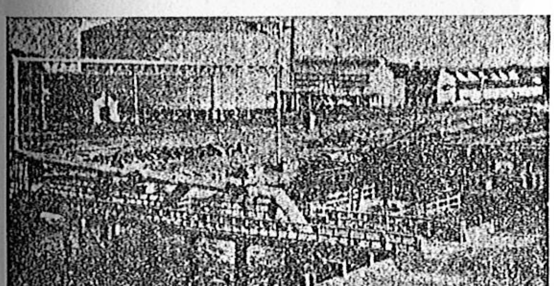

Fig 1 Ammonia tank at Willingdon Island, Cochin

The tank is circular in shape and was constructed on a flexible RCC raft (tank bottom slab) supported by 217 cast-*in-situ* concrete piles driven concentrically in circular rows (numbered A to H) as shown in Fig 3(a). The piles were of 50 cm diameter and were driven down through 26 m soft clay deposit into 8 m of stiff clay and finished 1 to 2 m below the ground level. The piling firm, based on their experience, available soil properties and a load test on similar pile in the area, assigned a safe load of 88 t on each pile. Over the pile tops, RCC columns of equal diameter were raised for supporting the tank 1 m clear off the ground level in order to avoid frosting of ground due to low temperature of liquid ammonia. A water load test was to be carried upto 16 000 t but when it reached a value of about 8 100 t, cracks were noticed in the outer row of columns. Also the tank bottom slab sagged to a saucer-shaped configuration showing 45 cm deflection at the centre. The contours of the deformed bottom slab of the tank are shown in Fig 3(b).

TANK STRUCTURE AND ITS FOUNDATION PRIOR TO UNDERPINNING

The container cup of the tank, 40 m in diameter, is covered by an outer shell of 41.6 m diameter. The cup is open at the top while the outer shell is provided with a dome shaped cover. Both the shell and the cup are made of special steel plates (with 5 mm thick bottom and walls of 8 to 12 mm thickness) welded together. An insulation layer has been provided around the cup and its bottom to sustain ammonia at —34°C.

The bottom slab of the tank was designed as a flat slab having reinforcement on both the faces. In order to facilitate drainage of ammonia, a camber of 10 cm was provided by keeping the slab 50 cm thick at the centre and 40 cm at the ends. A layer of sand was also provided between the underside of the outer shell and the bottom slab of the tank.

In the piles of the outermost row (A), reinforcement was provided to a depth of 25 m and in all other piles to a depth of 6 m only. The pile-column joints were found to have large eccentricity which was revealed during the excavation of basement during underpinning. The eccentricity was noticed in about 70% of the piles,

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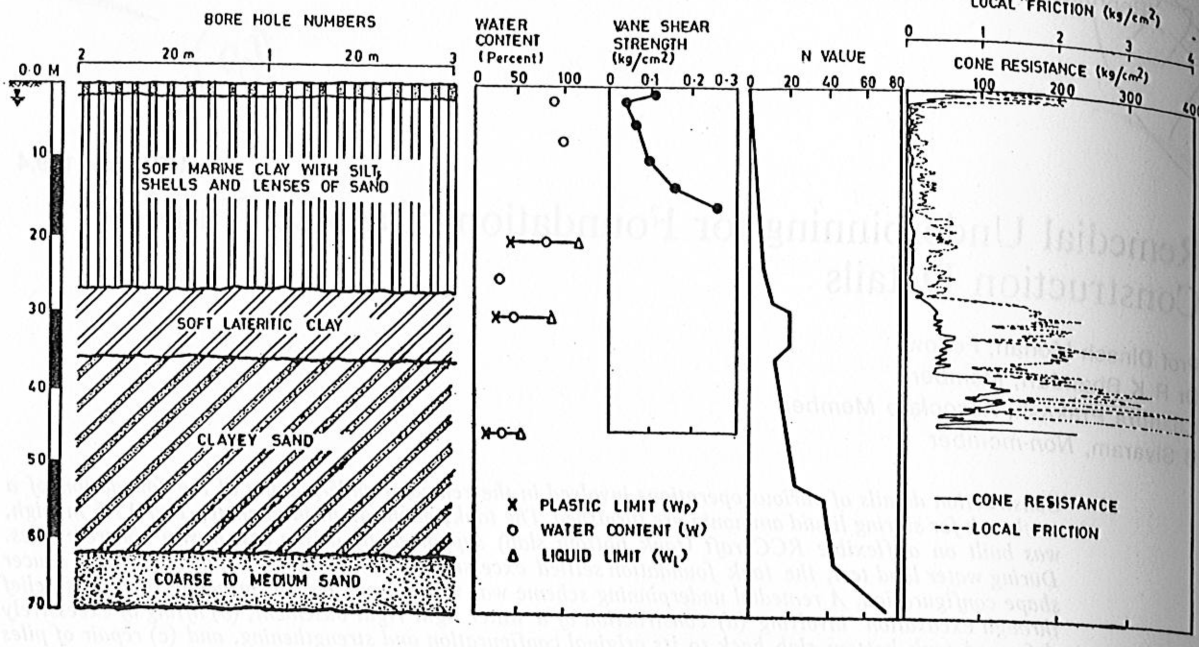

Fig 2 Subsoil characteristics at tank site

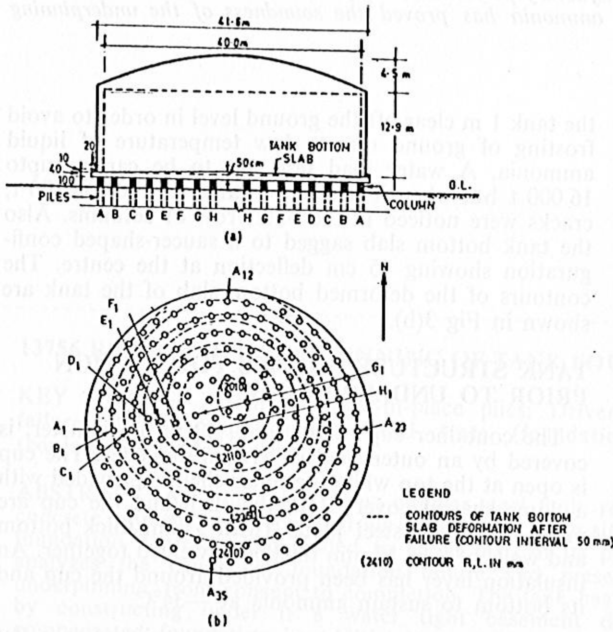

Fig 3 Details of ammonia tank

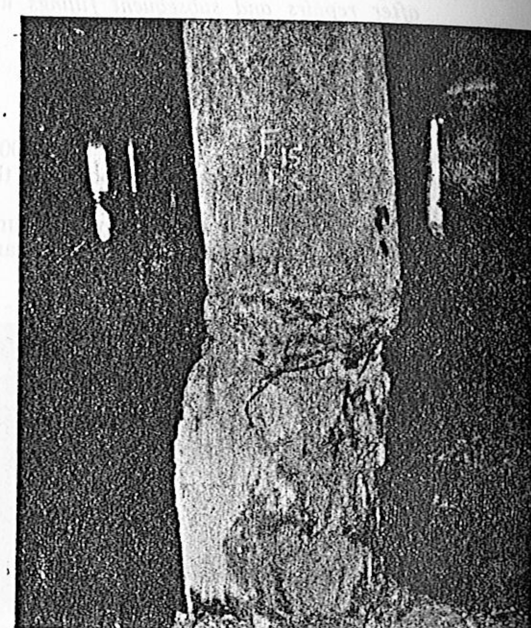

Fig 4 Defective pile-column joint

the minimum being 54% in Row D and maximum 77% in Row E. Also, about 77% pile-column joints had either bad concrete or clay gouge, the minimum being 50% in Row H and maximum of 90% in Row C. In a few piles, even below the joints, bad quality of concrete reduced pile size and other piling defects were noticed. The condition of a typical joint is shown in Fig 4.

PROPOSALS FOR REMEDIAL MEASURES

In formulating an appropriate remedial scheme, the main requirement was that the tank could be utilized for storing ammonia to its full rated capacity of 10 000 t or as close to it as possible but in no case it was to be less than 8 000 t. Also, the camber provided in the bottom of the tank was to be maintained. Other than these functional requirements, the implementation of the scheme was to be carried out within nine months comprising seven months of underpinning work and two months of water load testing. It was also desirable that the services, such as the connecting of pipe lines were to be dislocated to a minimum extent. Two alternative proposals were examined, namely (i) shifting the tank to new foundation—*inter alia* (it called for redoing of auxiliary services), and (ii) remedial underpinning of existing foundation. For remedial underpinning, it was required to (i) strengthen the foundation either by providing extra piles or by releasing the load on foundation, and (ii) bring the bottom of the tank in original configuration.

The proposal for strengthening the foundation by providing extra piles was not acceptable due to doubtful feasibility. The idea of obtaining the desired stress-relief on the foundation by providing a water tight basement was therefore favoured. Examining various alternatives from techno-economic considerations, it was decided to bring the bottom slab of the tank back to its original configuration by lifting it, using a battery of jacks. Main features of the scheme were :

1. Construction of a RCC basement integrally with existing piles (Fig 5). It was made deeper in the middle and shallower at the ends in order to reduce differential settlements and for achieving basement excavation without impairing the stability of surrounding structures, a cutoff wall and a system of well points were provided outside the excavation boundary.

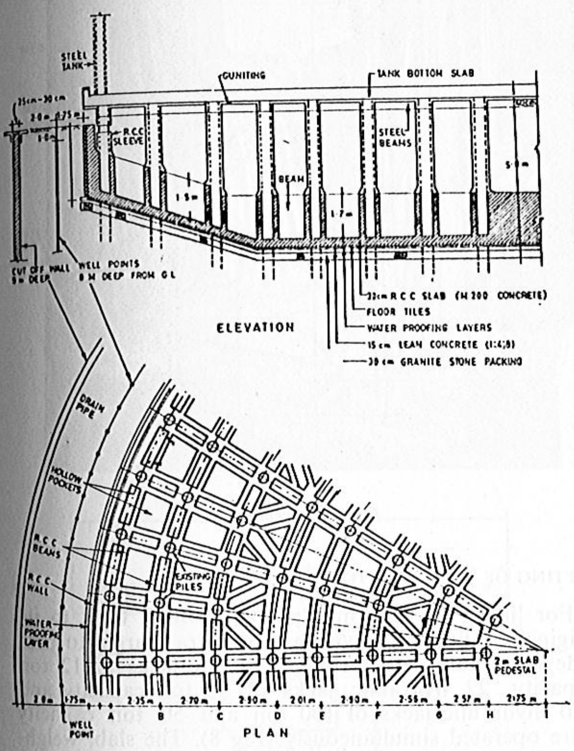

Fig 5 Details of basement

2. Lifting of the deformed bottom slab of the tank back to its original configuration.

3. Repair and strengthening of tank bottom slab and the exposed piles.

4. Final water load test.

The settlements, piezometric variations, load transfer through pile columns and the contact pressure distribution were measured.

MODIFICATIONS DURING EXECUTION

At first, the underpinning scheme was based on a safe pile capacity of 45 t. Fifteen piles (atleast one in each row) were load tested. From the test results it could be inferred that the outer periphery piles (Row A) behaved much better, possibly in view of the deeper reinforcement in them. The behaviour of piles in all other rows was very erratic, some showing ultimate capacity as low as 20 t. The following design features were, therefore, included in order to meet the new situation :

1. The basement was made rigid to avoid pile to pile differential settlement.

2. Piles in Row A were isolated from the basement by providing sleeves. This would minimize the development of heavy bending moments in basement slab.

3. The water load test was limited to a maximum of 11 600 t which corresponds to the ammonia load of 10 000 t.

EXECUTION DETAILS

Excavation for Basement

The entire excavation for basement was carried out manually because of the presence of piles. It was about 3.3 m deep at the ends and 5.3 m deep in the central portion and was done in steps. For the first 1 m, upto the ground water table, it could be achieved by providing surface drains. Another 1 m earth could be excavated by guiding water towards the centre into a pit and resorting to occasional pumping. Further, the excavations in the centre portion was extended upto about 3 m below ground level. In this case also the water was controlled by occasional pumping. Because of the limited space available around the tank, it was necessary to excavate the earth at a slope of 1:1. A cut-off wall and well point drainage system (Fig 6) were provided around the excavation to ensure stability of its slope as well as to facilitate digging under water. The provision of cut-off wall also helped in safeguarding the stability of adjacent structures. The construction of cut-off wall and installation of well points were started simultaneously with excavation.

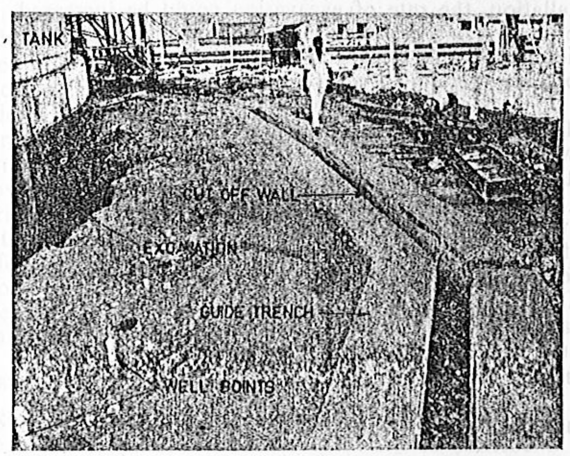

Fig 6 Cut-off wall and well points

Cut-off Wall

In order to avoid any fall of top soil during trenching, a RCC guide trench (0.5 m deep) was first constructed. Thereafter, the trench for cut-off wall was dug by employing both the CBRI process[1] and by using grabs designed specifically for the purpose to speed up the work. The CBRI process involves making of overlapping boreholes by jetting bentonite slurry through a pipe having a cutter fixed at its bottom end. In the present case, boreholes of about 25 cm diameter were made to a depth of 9 m. While employing grabs, two boreholes of 25 cm diameter were made at a specified distance using auger. Thereafter, the soil between the boreholes was scooped out upto 9 m depth using a suitable grab operated by a mechanical winch.

In both processes, alternate panels of 1.4 to 2.0 m length were cut. The presence of shells and sticky clay caused some difficulty in the initial stages, but by using higher capacity pump and cutters with longer teeth while using the CBRI process and by modifying the cutting edge of the grab, satisfactory progress could be achieved in both cases. The alignment of cut-off wall was revised suitably to avoid any disturbance to the intervening services and structures although this led to an increase in the number of joints. After cutting the trench and its proper cleaning, reinforcement cage to 6 m depth was lowered into it and concreting was done using a tremie.

Installation of Well Points

The well points were installed between the cut-off wall and the outer periphery of the tank. Boreholes of 20 cm diameter and 8 m depth were first made and cleaned by a water jet. Strainers of 1 m length coupled to extension pipes were next lowered into the boreholes. To avoid choking, continuous outflow of water was maintained through the strainers. The annular space between the borehole and the strainer was then back-filled with 2:1 sand and gravel mixture. This method of installation of well points was necessary as compared to the conventional method of jetting them into the ground in view of the clayey nature of subsoil. It was difficult to pump up 8 m in a single stage because of possible cavitation. The header pipe was, therefore, kept 1 m below the ground level.

Laying of Base Course and Lean Concrete

With the completion of cut-off wall and well point installation, the rate of excavation could be increased. The heaving of soil due to excavation was minimized by dividing the basement area into about eight sectors. Soon after completing excavation in one sector, a 0.3 m thick layer of 40 mm stone ballast was laid to perform the dual function of providing a base course as well as a capillary cut-off. The laying operations were first finished in the sloping portion and subsequently in the flat portion of the basement. Suitable gradient was provided to guide the seepage water towards the central pit from where it could be pumped out. After laying the base course and cleaning the exposed piles, an overlay of 0.15 m thick 1:4:8 concrete was placed. In order to obtain a smooth surface for water proofing treatment over the concrete base, a 1:3 cement plaster coat, about 12 mm thick, was applied.

Pile Repairs

As the excavation progressed, the condition of exposed piles and pile-column joints could be examined. Piles having large eccentricity, bad concrete at joints and ill placed reinforcement were common. The work of pile repairs was taken up simultaneously so as to complete it before lifting of the bottom slab of tank. For repairing the defective piles and joints, the repair work was categorized into columns of eccentricity, namely, (a) less than 8 cm, and (b) more than 8 cm. In the first case, piles having poor joints were cleared of bad concrete and during this process, the self weight of the slab was supported with clamps (Fig 7). After removing the bad concrete, reinforcement was straightened. Also wherever necessary, a RSJ (having plates welded at both ends) was added and extra reinforcement was welded. A rich concrete mix of high slump was then poured after providing appropriate shuttering. In case of piles with larger eccentricities, the reinforcement in the pile section was extended vertically upto about 50 cm below the slab. Then it was cast in the form of round concrete column. Exposed piles having any other defect were also repaired suitably.

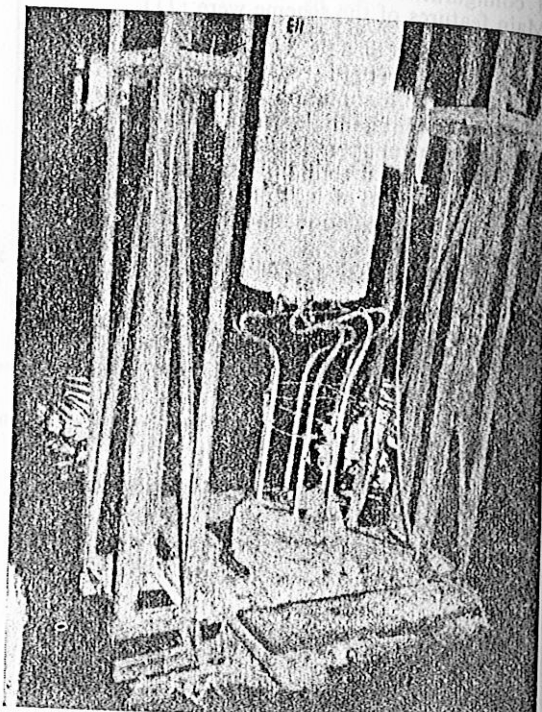

Fig 7 Pile repairs

Lifting of Tank Bottom Slab

For lifting the bottom slab of the tank back to its original configuration with a little extra margin to provide for future settlement, 350 screw jacks of 12 ton capacity, 22 hydraulic jacks of 30 ton capacity and two hydraulic jacks of 100 ton and 50 ton capacity were operated simultaneously (Fig 8). The slab, weighing about 1 300 ton, was lifted successfully by 55 cm which was 10 cm more than the elevation of earlier camber at the tank centre.

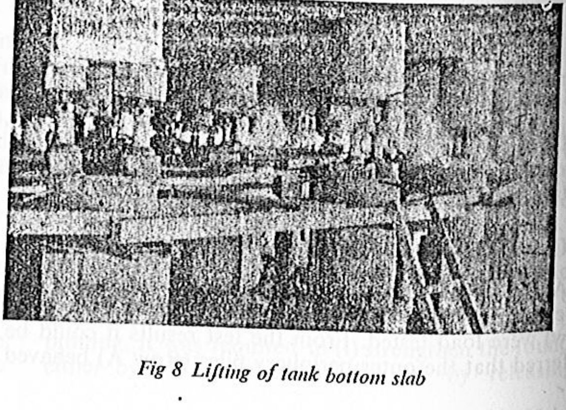

Fig 8 Lifting of tank bottom slab

For placing the jacks, the columns (excepting those in Row A) were cut about 50 cm below the tank bottom slab [Fig 9(a)] to provide a clear gap. The cutting was carried out by hand chiselling. It was staggered suitably and the weight of the slab during cutting was temporarily supported with a steel frame resting on a clamp fixed to the piles. The reinforcement bars were also cut and bent suitably. The top and bottom surface of the cut portion was finished with concrete. Two screw jacks of 12 t capacity were then introduced with sufficient wooden packings and the steel frames were removed [Fig 9(b)]. Between the two screw jacks adequate space was provided to accommodate a hydraulic jack. To facilitate manipulation of jacks, a wooden platform was provided connecting the columns at about 1.5 m below the tank bottom slab. Cutting of columns did not deflect the bottom slab of the tank to a measurable degree.

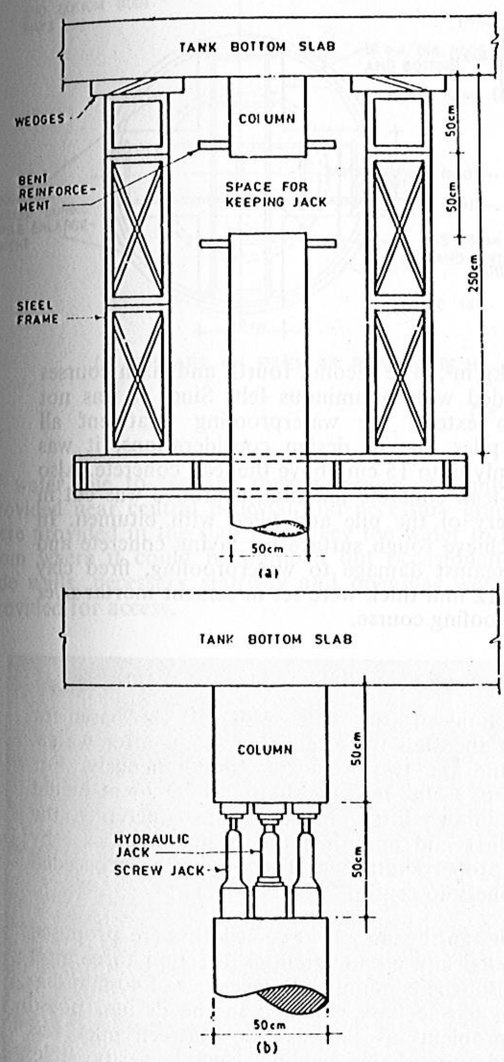

Fig 9 Cutting of gaps in column and placement of jacks

To provide reference for measuring the lift, reduced level of slab bottom near each column was recorded with a levelling instrument. The distance between the cut faces of each column was also measured. Since the ultimate capacity of some of the piles was as low as 20 t, it became necessary to check pile movement also during lifting. For this purpose, a mark provided on each pile exactly one metre above the reference block on the lean concrete base served as a reference point.

Initially, hydraulic jacks were kept one on each pile of Row G and one on alternate piles of Row F. For operating the jacks, three persons, two for screw jacks and one for hydraulic jack were employed at each location. The lifting operation was controlled by verbal instructions which were asserted by use of megaphone. First, a trial was made and a 2 cm lift was achieved within an hour when 88 persons operated the jacks. Subsequent to this successful trial, full scale lifting operation was taken up. The total lift of 55 cm was achieved within 10 days. After each 10 cm of lift, wooden packings were introduced under the jacks. In case the gap increased by more than 30 cm, the blocks of 10 cm were replaced by a solid block of 30 cm. After completion of jacking operation, gaps in some of the columns were as much as 100 cm. In order to guide the lifting operation each day, the level of slab bottom was monitored at selected locations and successive slab positions were plotted. Positions of the hydraulic jacks were readjusted to the regions of the slab which required greater lifting. The pile movements were also checked and when any movement of pile was observed, the hydraulic jacks were shifted to adjacent piles. While lifting the bottom slab of the tank beyond its original configuration, hair line cracks were noticed in 17 piles of Row A. In two cases, vertical cracks in the column above jacks were also seen. The cracks in piles of Row A were repaired subsequent to lifting. The cross-section and contours of the bottom slab of the tank before and after final lifting are shown in Figs 10(a) and (b), respectively.

Renewal of Columns and Provision of Collars

Subsequent to the lifting operation, steel I-sections (150 × 150 mm) of suitable lengths with plates welded at their ends were wedged in the space provided for hydraulic jacks [Fig 11(a)]. Screw jacks were then removed and the pile reinforcements were realigned and welded in position [Fig 11(b)]. Reinforcement cages required for providing collars on the piles were then tied ; shuttering was provided and concreting done. The collars [Fig 12(a)] supported the top beams [Fig 11(c)]. In collars, a notch was provided on four sides which accommodated the reiforcement emanating from the enlargement of pile section. Thus, a proper bond between collar and pile could be achieved. The provision of notch also facilitated pouring of concrete during enlargement of pile section. Since the columns in the outermost row were not cut, their collars were different as shown in Fig 12(b). In this case, the reinforcement for collar was welded to the exposed reinforcement of columns and the collar was cast circular in shape.

Basement Construction

Since the pile load tests revealed variable carrying capacities of piles, it became necessary to provide a stiffer basement in order to minimize differential settlements. To reduce bending moments due to the fixity provided by the piles in Row A, these were isolated from the basement by providing sleeves. The basement details are shown in Fig 5.

Waterproofing

Waterproofing treatment comprising seven courses was taken up on the dry plastered surface of lean concrete, after completion of lifting operation. The first, third, fifth and seventh courses of bitumen were applied

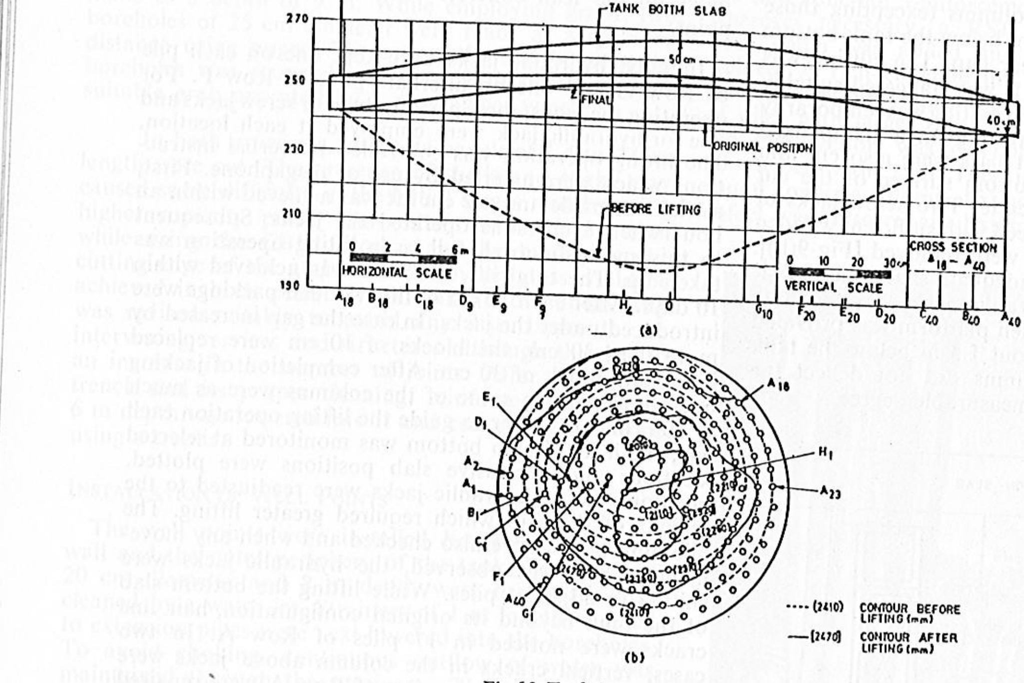

Fig 10 Tank bottom slab after lifting

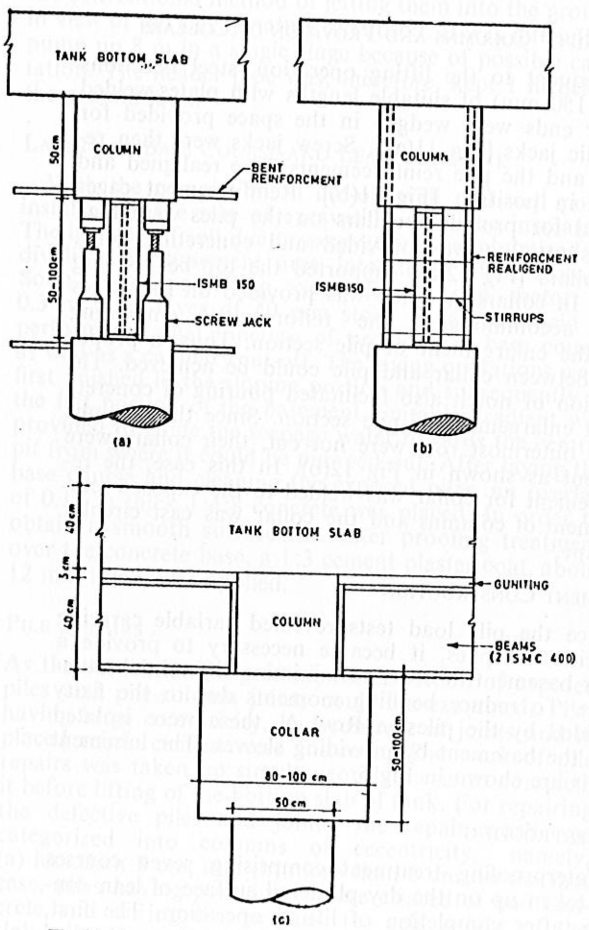

Fig 11 Renewal of columns and provision of collars

hot at 1.5 kg/m². The second, fourth and sixth courses were provided with bituminous felt. Since it was not possible to extend the waterproofing treatment all along the piles due to design considerations, it was extended only upto 15 cm above the lean concrete. Also at the pile-lean concrete junction, a groove was cut in the periphery of the pile and filled with bitumen. In order to achieve rough surface for laying concrete and to guard against damage to waterproofing, fired clay tiles about 12 mm thick were set in cement mortar over the waterproofing course.

Construction of Slab, Beams and Side Walls

Construction of the side wall of the basement along with the slab were taken up soon after waterproofing and the two were cast simultaneously. For concreting of walls, moving shutters of 1.75 m height were used in two lifts. For slab-beam concreting, the slab was first laid and then the beams were cast by providing proper shuttering. The concreting proceeded from periphery to centre.

In the design, beams of 40 cm width were proposed both in radial and circumferential direction to connect each pile. But the bending and placement of 40 mm diameter high tensile bars, adopted in the design, posed practical problems as the distance between piles was varying and no workable bending schedule was possible. The problem was solved by adopting two beams of 20 cm width instead of a single beam between the piles (Fig 5). It also improved the speed of construction. No bending was therefore required in radial direction and only slight bending in circumferential direction. It led to an increase in shuttering on account of increased area but it was compensated by reduction in quantity of concrete and steel. A view of the beam-slab construction is shown in Fig 13. The central portion surrounded by piles of Row H was cast as a pedestal. For collection

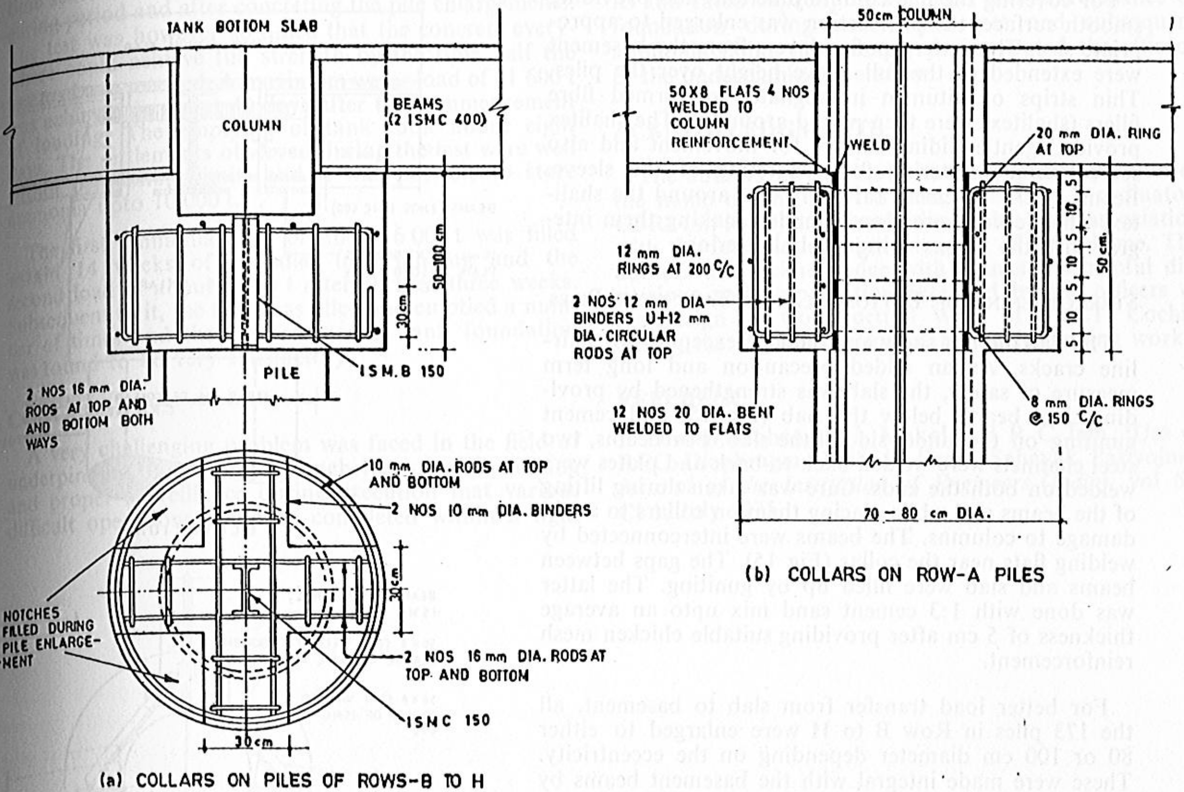

Fig 12 Details of collars

of water due to seepage during service, a sump was provided near central pedestal and necessary gradients were provided in the slab to convey the water to sump from where it could be pumped out occasionally. In side walls, necessary stairway and openings were also provided for access.

Fig 13 Construction of basement

Subsequent to the construction of side wall, the waterproofing treatment was applied on its outer face by extending the layers from the basement to full height of the wall. A brick wall of 115 mm thickness was then constructed, sandwiching the waterproofing treatment and providing it a protection.

Sleeving of Piles in Row A

In order to minimize the development of heavy bending moments in basement raft, the piles in Row A were isolated from the basement by providing sleeves (Fig 14).

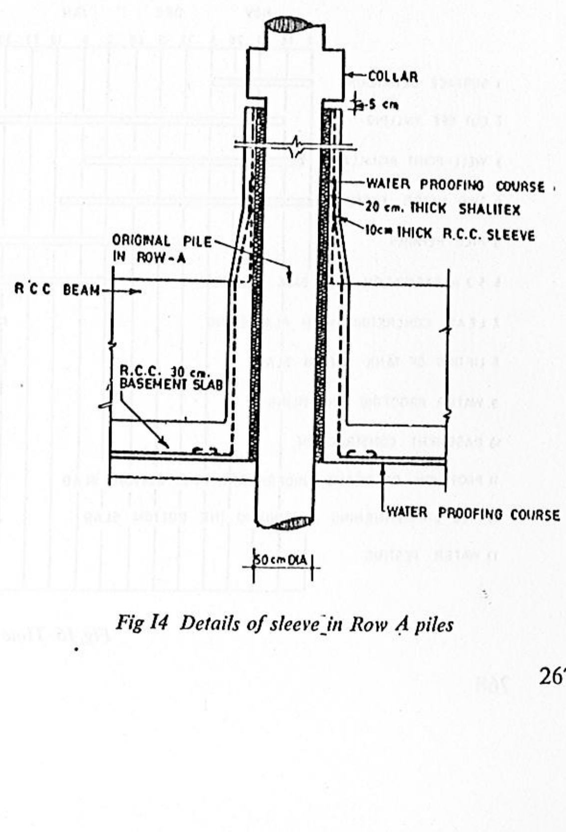

Fig 14 Details of sleeve in Row A piles

For covering the pile column junctions and providing smooth surface, the pile section was enlarged to appropriate size. The waterproofing layers from the basement were extended to the full sleeve height over the piles. Thin strips of bitumen impregnated preformed fibre fillers (shalitex) were then placed around it. The shalitex provided gentle sliding surface for movement and also served as a shuttering for concreting of the sleeve. Beams of the basement raft were cast around the shalitex. The sleeves were then extended making them integral with the beams using steel shuttering.

Strengthening of Bottom Slab of Tank and Piles

The slab did not show any damage except a few hairline cracks. As an added precaution and long term measure of safety, the slab was strengthened by providing steel beams below the slab (Fig 15) and cement guniting on the under side of the slab. For beams, two steel channels were welded back-to-back and plates were welded on both the ends. Care was taken during lifting of the beams and while placing them on collars to avoid damage to columns. The beams were interconnected by welding flats near the collar (Fig 15). The gaps between beams and slab were filled up by guniting. The latter was done with 1:3 cement sand mix upto an average thickness of 5 cm after providing suitable chicken mesh reinforcement.

For better load transfer from slab to basement, all the 173 piles in Row B to H were enlarged to either 80 or 100 cm diameter depending on the eccentricity. These were made integral with the basement beams by extending the reinforcement from beams upto the collar at top. Part of the reinforcement was tied with the reinforcement in hollow space provided during collar construction. A time schedule for the complete work is presented in Fig 16.

WATER LOAD TEST AND AMMONIA FILLING

Water load test was to be conducted after the completion of underpinning works. But, in view of the tight

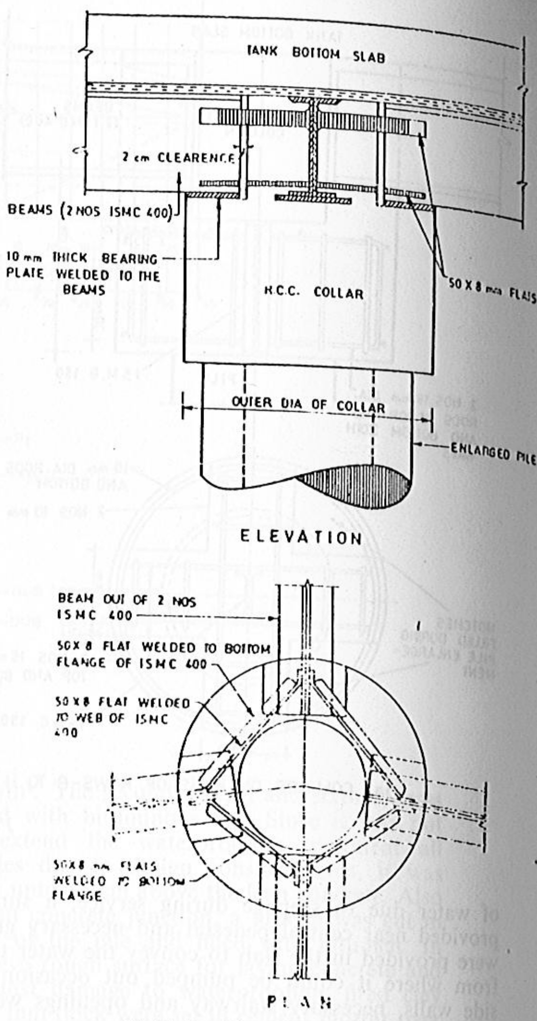

Fig 15 Provision of top steel beams

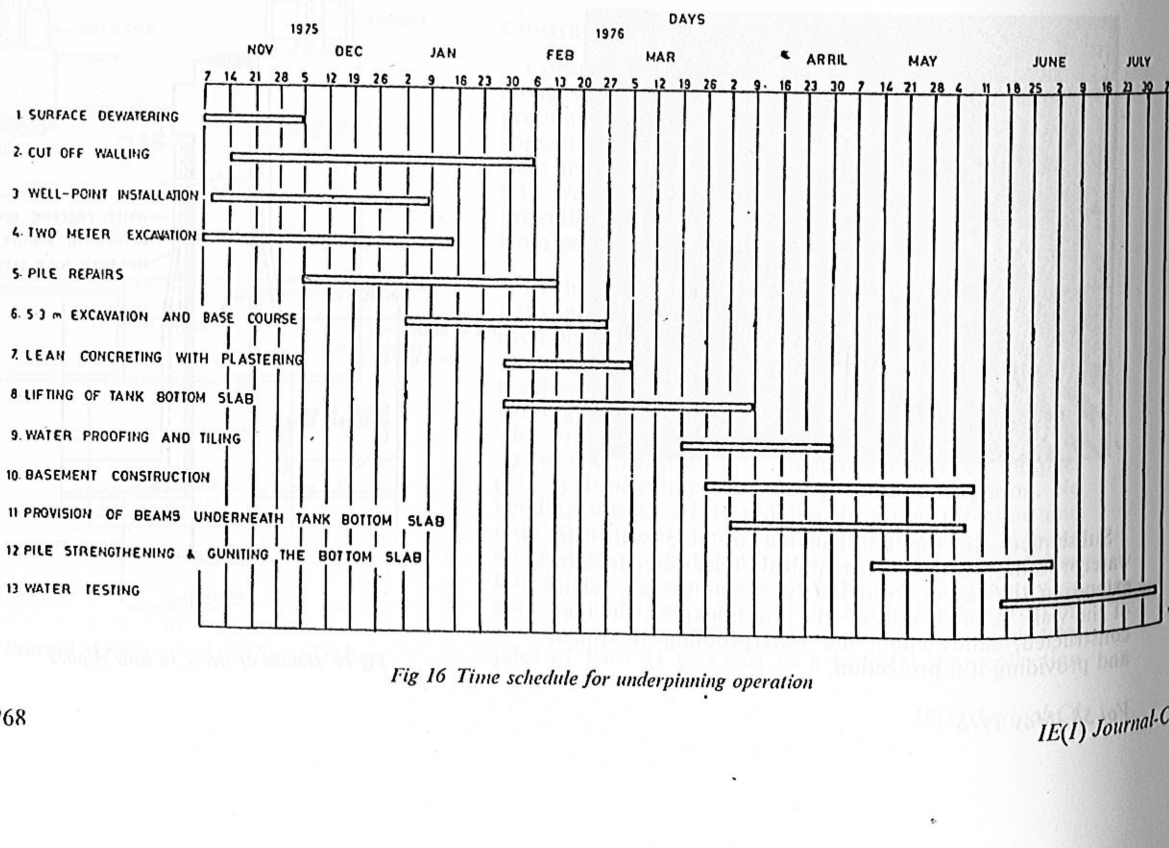

Fig 16 Time schedule for underpinning operation

time schedule, it was started without even allowing full curing period and after concreting the pile enlargements. The test was however so timed that the concrete everywhere could achieve full strength by the time half the test load was reached. A maximum water load of 11 600 t was achieved in about 40 days after the commencement of loading. The emptying of tank took about eight days. The settlements observed during the test were well within permissible limits and it was possible to store ammonia upto 10 000 t.

The first ammonia load of about 6 000 t was filled within 14 weeks of remedial underpinning and the second load of about 8 600 t after further three weeks. Subsequent to it, the tank was filled and emptied a number of times and the performance of tank foundation was found to be very satisfactory.

CONCLUSIONS

A very challenging problem was faced in the field of underpinning. It was only through very careful planning and proper surveillance during execution that various difficult operations could be completed within a tight time schedule. The tank could eventually be restored to its full rated capacity of 10 000 t. The performance of foundation during water load test and subsequent ammonia filling have amply demonstrated the efficacy of the underpinning operation.

ACKNOWLEDGMENTS

The authors would like to make a special mention of the name of Shri G R S Jain, Scientist Co-ordinator, CBRI (on leave) and thank him for his close association with the formulation and execution of the scheme. The authors also acknowledge with thanks the helpful discussions and co-operation extended by the officers of the Design and Construction Wing of FACT, Cochin during the implementation of the underpinning work.

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