# EVALUATION AND REHABILITATION OF CONCRETE PIER

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**ABSTRACT:** This paper presents a case study of repair and strengthening work carried out on a defective reinforced concrete pier. The pier manifested severe cracking and spalling at the joint between its walls and platform. The distress was due to twisting of the platform as a result of torsional stress, which had also caused diagonal shear cracks to appear on the walls. A comprehensive investigation was carried out in order to determine the causes and extent of the problem and to define the remedial measures needed to rectify the problem and prevent future deterioration. The investigation included three phases: field investigation, which included soil borings, platform temperature measurement, and measurement of twist and tilt of the platform; laboratory investigation, which included soil testing and concrete core testing; and analysis of the structure. The repair work consisted mainly of providing a silica-fume-blended cement shotcrete jacketing system around the structure. The repair work was successful, economical, technically feasible, and not very labor-intensive.

## INTRODUCTION

+ A reinforced concrete tubular pier in a cement factory manifested severe cracking and spalling at the joint between its walls and platform. The distress was apparently due to twisting of the platform as a result of torsional stress, which had also caused diagonal shear cracks to appear on the walls. The twist was so significant that most of the vertical reinforcement bars passing through the defective platform/walls had been completely sheared.

The pier is one of six carrying a rotary kiln. The distress was serious and needed immediate repair because any movement of the platform could have affected the roller elevation and, consequently, the kiln-roller contact. This uneven contact could result in kiln misalignment and hence an unplanned shutdown of the kiln and/or probable further deterioration of the pier. The adjacent piers could also suffer deterioration, including complete failure. Any of these probabilities would cost a lot more than the cost of repairing the defective pier. Extensive testing and evaluation was carried out in order to devise an effective system for repairing the defective structure and increasing its capacity to resist the twisting moment.

The objective of this paper is to discuss the diagnostic evaluation process and the repair work performed to repair the distress, strengthen the pier, and protect it from further deterioration.

### Description of Problem

Each rotary kiln is 5 m in diameter and 191 m long. The weight of each kiln, around 3,700 t including the charge, is transferred to the piers through steel rollers that are mounted on the top surface of a 1.5-m-thick reinforced concrete platform. The platforms are built monolithically with 0.6-m-thick pier walls. Each pier is built on a 1.4-m-thick reinforced concrete mat foundation.

The pier being studied is 9.560-m long and 3.620-m wide. The height of the pier varies from 8.060 to 7.952 m for the purpose of accommodating the slope of the kiln. The pier is provided with a reinforced concrete cantilever walkway, 1-m wide, connected to its platform. The pier was built in 1980. The concrete used in the pier conformed to German DIN 1045 Standards, Class B25. The strength requirements for this class of concrete were 25 and 30 MPa (3,625 and 4,350 psi) for the minimum compressive strength of each cube and the minimum average of compressive strengths of each series of cubes, respectively. The specified minimum concrete cover to reinforcement was 30 mm. The main reinforcement of the walls consisted of two layers of 16 mm horizontal bars at 250 mm and two layers of 19 and 22 mm vertical bars in the long sides and short sides of the pier, respectively, both at 200 mm

At an early point in their lives, all the piers, including the one being studied, manifested various types of defects. Such defects included delamination, honeycombing, and cracking. These defects were repaired at that time; the delaminated and honeycombed areas were chipped out to the sound concrete and filled with cementitious repair material, and all cracks 0.2 mm and greater in width were injected with epoxy resin.

A few years later the pier being studied manifested severe cracking and spalling at the construction joint between its walls and platform due to twisting of the platform as a result of torsional stress. The twist was so significant that most of the vertical reinforcement bars passing through the defective platform/walls joint had been completely sheared, and the platform was left "floating" on the walls (Fig. 1). Diagonal shear cracks were also observed on the walls of the pier, on both exterior and interior surfaces, sometimes penetrating the full thickness of the walls. Some of these cracks were a recurrence of old cracks that had been repaired earlier, and some were new cracks. Fig. 2 shows the details of the pier and distress manifested. The inclined lines on the platform walls indicate the direction of the shear cracks only. For more specific details of the case refer to Tahir (1995).

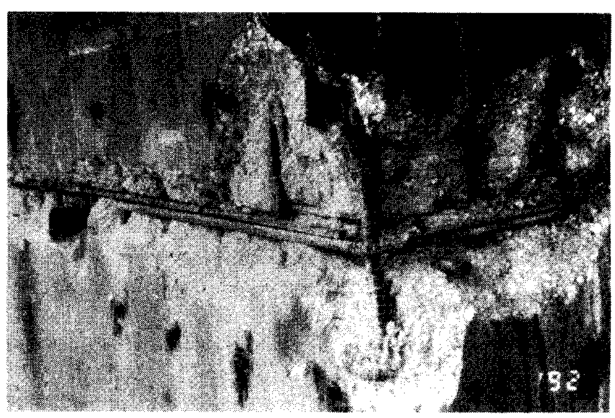

FIG. 1. Damaged Platform/Walls Joints

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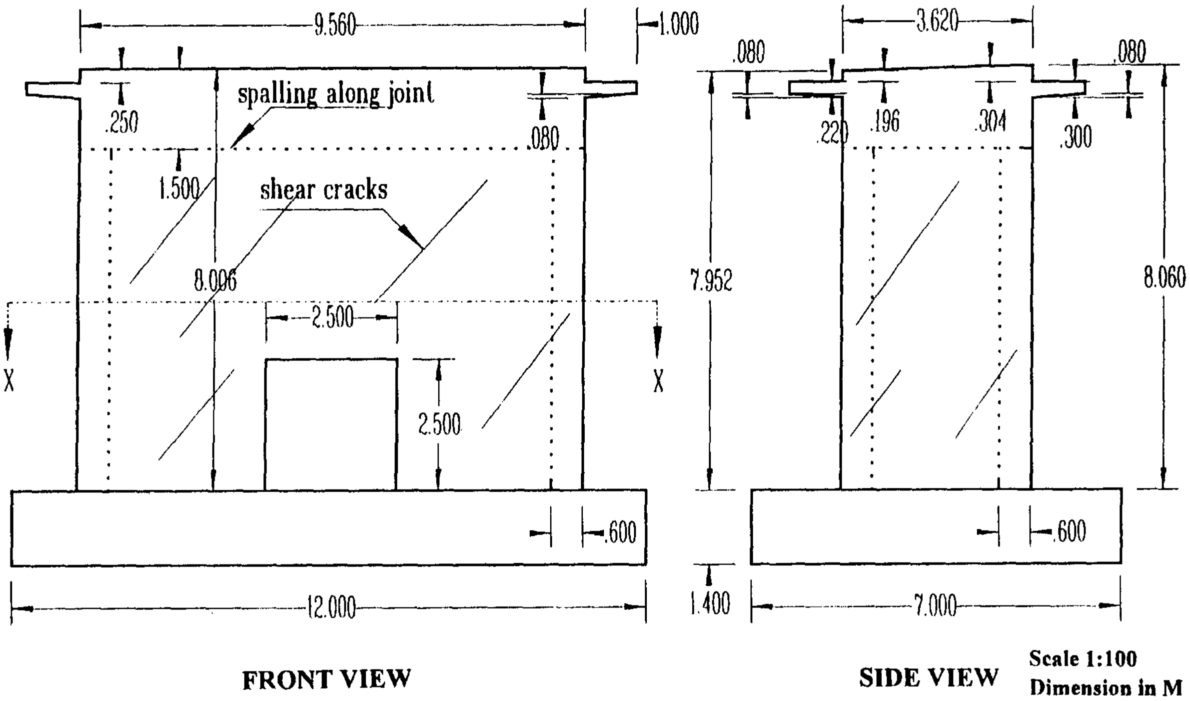


**FIG. 2. Details of Pier and Distress Manifested**

## INVESTIGATION

Due to the apparent complexity of the problem, a comprehensive investigation was carried out in order to determine the causes and extent of the problem and to define the remedial measures needed to rectify the problem and prevent future deterioration. The investigation included three phases: field investigation, laboratory investigation, and analyses.

### Field Investigation

*Soil Borings*

A total of three soil borings were made to determine the soil stratigraphy and to obtain soil samples for chemical testing. Two of the borings were drilled near the pier being studied, whereas the third one was drilled near the corresponding pier of an adjacent kiln to serve as a control site for comparison.

The site was found to be underlain by two strata to the maximum 3.3 m depth explored. The surface stratum consisted of calcareous silty fine to medium sand with limestone fragments known as marl. The surface marl was underlain by limestone. The limestone was weak to moderately weak, based on a field estimate and unconfined compression test results. The rock quality varied from poor to excellent, based on rock quality designation (RQD) measurements.

The dynamic cone penetration tests (DPT) were performed at three locations. Locations 1 and 2 were again near the pier being studied, and location 3 was near the corresponding pier of an adjacent kiln. The tests were performed in accordance with the Swedish Geotechnical Society standards. Based on the DPT results, the surface marl was medium dense, with refusal occurring at depths of 0.3, 1.2, and 0.1 m in borings 1, 2, and 3, respectively.

*Platform Temperature Measurement*

The temperature variation across the depth of the pier's platform was measured. In order to monitor the temperature, 17 points were selected; 12 of these were on the top of the platform, and one was on each of the remaining five sides. The temperature at 10 of the points on the top of the platform was taken at three levels: the surface, 50 mm below the surface, and 100 mm below the surface. At the remaining points the temperature was measured only at the surface.

The temperature measurement was done in May. The variation along the depth of the platform was nonlinear. The maximum difference in temperature observed was 16°C. The temperature increased to a maximum inside the platform and gradually reduced at the bottom surface to slightly less than at the top surface. In the original design analysis, a temperature difference of 52°C had been considered in this reinforced concrete platform.

*Measurement of Twist and Tilt of Platform*

The displacement of the platform corners, from vertical lines projected upward from the corners of the pier, was measured every 1 m. Fig. 3 illustrates the corner displacement. The twist notation used shows the movement as clockwise when viewed from the top of the pier.

### Laboratory Investigation

Sulfate content was determined by the calorimeter method in accordance with ASTM D 516, and chloride content was determined by the silver nitrate method in accordance with ASTM D 512. The sulfate contents of samples recovered from borings 1 and 2 were found to be 0.028 and 0.013%, respectively, and the chloride contents of samples from the same borings were 0.039 and 0.018%, respectively. pH values ranged from 7.7 to 7.8. By comparing these results with the requirements of ACI 318, the subsurface soils were not considered aggressive to the reinforced concrete foundation. Identification tests performed on the granular soil occurring at the site consisted of sieve analyses and determination of the percentage of material passing the 0.075-mm (No. 200) sieve.

From the results of the borings, dynamic penetration tests, and laboratory tests on soil samples collected from the site, the following conclusions were made:

1. The pier foundation lay on a limestone stratum—the same type of stratum as that of the control location.

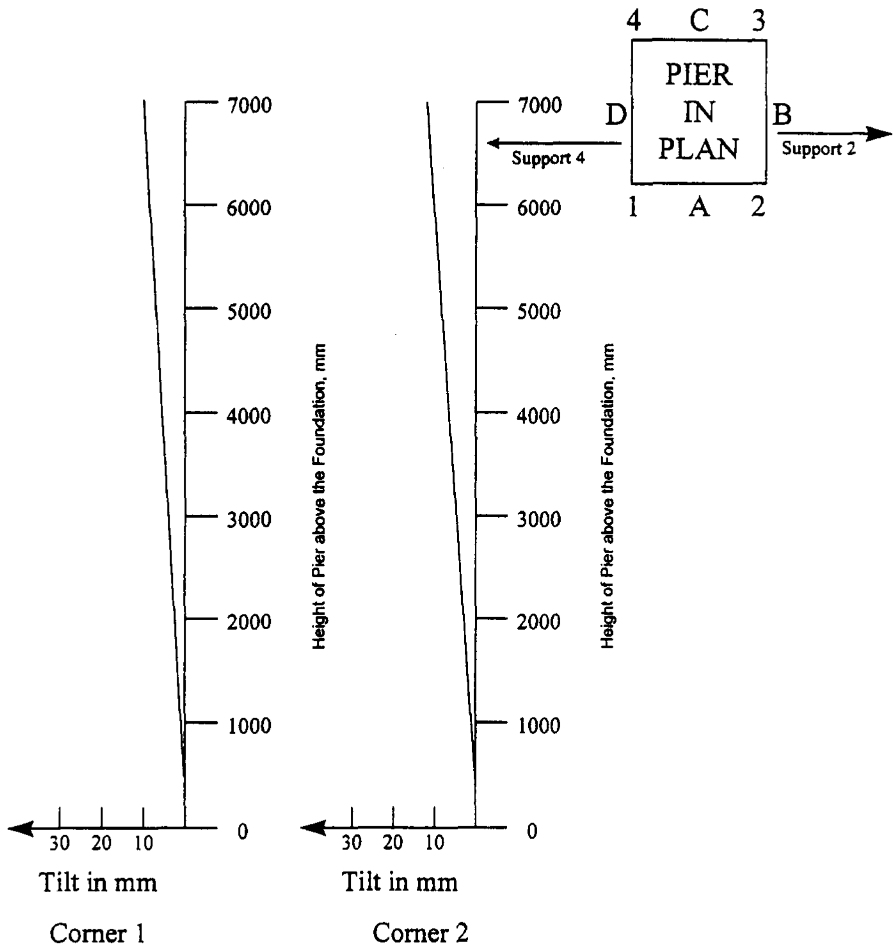


**FIG. 3. Twist of Pier Observed along Face**

2. The safety factor of the bearing pressure of the limestone below the foundation of the pier was found to be at least two.
3. The settlement of foundations laid on limestone strata is usually very small.
4. Chemical analysis of soil samples at the site indicated that the soil was not unduly aggressive to the reinforced concrete foundation structure.
5. Ground water was not encountered, moisture being one of the contributing factors in the corrosion of reinforcing steel.

For the concrete testing, a total of four cores were extracted from the pier for chemical and physical testing, three of which were from the platform and one from a wall. Tests conducted on the cores included visual inspection, determination of chloride content, sulfate content, cement content, compressive strength, and density.

Chloride and sulfate tests were performed in accordance with BS 1881. The permissible chloride content in concrete may be taken to be 0.4% by weight of cement (Pullar-Strecker 1987). Based on this limit, the chloride content in the present concrete is within acceptable limits.

The sulfate content in concrete should not exceed 4% by weight of cement as recommended by ACI. Core numbers 3 and 4 had sulfate contents in excess of this on their top and bottom sides, respectively. Core number 3 was taken from outside of the pier and Core number 4 was taken from inside. The excessive sulfate content in both cases, therefore, is toward the exposed face of the pier.

Cement content tests were performed in accordance with BS 1881. The cement content used in the concrete of the pier exceeded 400 kg/m$^3$, which was judged adequate. The type of cement used in the pier could not be determined.

The compressive strength tests were performed on portions of the concrete cores. The length of each core, when capped, was twice its diameter. The specimens were tested for compression in accordance with ASTM C 39. The compressive strength of all cores was below the design strength of 25 MPa. The average concrete density of the platform was 2,140 kg/m$^3$ with a standard deviation of 32 kg/m$^3$ and a coefficient of variation of 1.5. The density of concrete with commonly used aggregates ranges from 2,160 to 2,500 kg/m$^3$. The concrete density in the platform was slightly below this range.

The laboratory tests on the concrete cores indicated that the strength of the platform concrete varied from 12.1 to 18.2 MPa, which is below the normal requirements. The reason for the low concrete strength could be poor construction practice, poor mix design, poor materials, or a combination thereof. No signs of reinforcement corrosion or concrete deterioration due to sulfate attack could be found on the pier.

## Structural Analysis

Structural analysis was carried out to evaluate the compression and twisting capacities of the tubular structure with the existing quality of concrete and the current detailing of the reinforcement. Evaluation of these structural capacities would make it possible to assess whether the existing structure was adequate in compressive strength to support the kiln load in the charged state and also to assess the critical value of the twisting moment induced in the structure. Both of these evaluations would subsequently help in making recommendations for the construction of additional concrete sections as well as the placement of additional reinforcement to the platform/walls interface.

The carrying capacity of the structure for the vertical compression caused by the charged kiln was found to be sufficient. However, that the concrete had a low strength and was subjected to conjoined tension due to torsion that could further reduce its compressive strength warranted strengthening for compression.

Horizontal torque is resisted concomitantly by the concrete and the horizontal hoop reinforcement in conjunction with the vertical reinforcement. The main torque resistance is provided by the horizontal hoops, the function of the vertical reinforcement being to help realize the potential of the horizontal hoops. The resistance of the concrete was ignored because of its low strength. The torque applied by the rollers, on the other hand, could not be calculated as there were no data on the differential strain that apparently caused the twisting moment. However, that the system had failed in torsion clearly indicated that the resistance provided by the existing reinforcement was not adequate and that it was less than the applied torque of the rollers.

Based on the findings of the investigation, it was decided that, as a permanent remedy to the stress in question, existing deteriorated concrete be repaired and additional reinforcement be added to resist the torsion. However, with the restoration and repair of the platform/walls joint, the torque transfer mechanism would also be restored, resulting in a resumption of the twisting moment in the lower portion of the tubular structure, which might aggravate the existing cracks. This meant that limiting the repair and strengthening work to the deteriorated platform/walls joint only, without strengthening the lower portion of the structure for torsion, would not be enough to solve the problem.

Two alternative techniques were formulated. The first alternative was to remove the cover to the reinforcement and repair the distress using a silica-fume-blended cement shotcrete. The second alternative was to provide a silica-fume-blended cement shotcrete jacketing system around the structure. The latter alternative requires more new reinforcement than the first alternative. The design calculations pertaining to the torque capacities of the two techniques far exceed the requirements. In the first alternative, all the concrete cover of the exterior surfaces of the pier (approximately 210 $m^2$) would have to be removed to leave a 25 mm or so gap behind existing reinforcement bars. Such a process would be extremely difficult and would require large amounts of time and labor. Thus it was judged to be not economically sound. Nor was it technically feasible; chipping by pneumatic hammers would result in microcracks all over the chipped surfaces that would later weaken the bond between the parent concrete and the new repair concrete.

Although the second alternative involved more new reinforcement than the previous technique, it was still more economically sound as it did not involve the labor-intensive chipping work. It was also more technically feasible since it would provide more torque resistance and would not result in microcracking. Therefore, the second alternative was selected.

## CONSTRUCTION OF REPAIR WORK

### Surface Preparation

The job started with the erection of scaffolding all around the structure, inside and outside, followed by sandblasting of all its surfaces. Sandblasting was done for the purposes of proving roughened surfaces for proper bonding of the shotcrete, removing all bond-breaking substances such as dirt, slurry, oil, and old cement wash, and exposing cracks and other defects for repair. All surfaces were blown with compressed air prior to any subsequent work in order to remove excess sand and loose debris.

### Platform/Wall Joint Repairs and Strengthening Work

A 600-mm-wide concrete strip around the platform/walls joint, inside and outside, 300 mm on each side of the joint, or the area of deteriorated concrete, whichever was larger, was removed using chipping hammers to a sufficient depth to provide at least a 25-mm clear gap behind the reinforcement bars (see Fig. 4). Low-mass hammers were used in order to minimize the formation of microcracking and to avoid weakening the surface. Chipped areas were sandblasted in order to completely remove residual fractured fragments on the surface and clear the platform/walls joint.

The reinforcement bars exposed were found to be in excellent condition as far as corrosion was concerned. However, approximately two-thirds of the vertical bars passing through the joint were completely sheared due to the torsional stress. The sheared reinforcement bars were restored back to continuity by using lap bars, as shown in Fig. 5.

As an additional measure to strengthen the joint, the walls and the platform were "stitched" together on the exterior surface with 16-mm, U-shaped reinforcement bars inserted in the concrete to a depth of 200 mm using epoxy grout (Fig. 5). One U-bar was provided at 300-mm by inserting 20-mm bars in the underside of the platform and in the walls around the platform/walls joint and then welding them together. The scheme was then covered with shotcrete around the joint.

All exposed reinforcement bars were cleaned and treated with an anticorrosion active single component epoxy primer. The joint and the surrounding cracks were then injected with low-viscosity epoxy resin to restore their structural integrity.

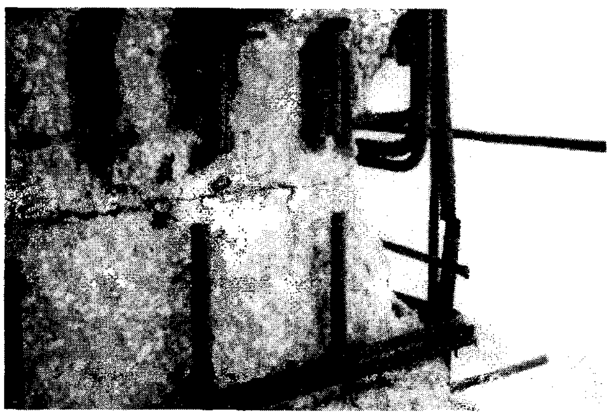

**FIG. 4. Southwest Corner of Joint after Chipping of Surrounding Concrete**

**FIG. 5. Platform/Walls Joint after Installation of Stitching Dogs**

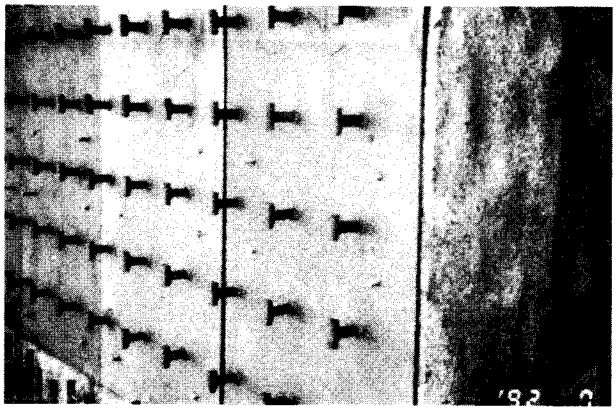

FIG. 6. Insertion of Chemical Anchors for Installation of Steel Plates

Injection was carried out from inside and outside of the joint to ensure complete filling. The low-viscosity epoxy resin was injected through fittings inserted in drilled holes around the joint.

Following completion of the reinforcement and epoxy injection work, the chipped area was filled with silica fume shotcrete. Immediately after shotcreting, the new concrete was covered with burlap and polyethylene sheets and kept wet for a period of two weeks for curing purposes. Four cores taken later through the joint revealed good epoxy penetration and good bonding between the existing concrete and new shotcrete.

As an additional measure to further strengthen the platform/walls joint, 8-mm-thick mild steel plates, 1.2 × 0.6 m, were bonded on the concrete surface around the joint as shown in Fig. 6. T-shaped shear connectors were welded on the exterior side of the steel plates before bonding at 200-mm intervals both ways to ensure a proper bond between the plates and the new shotcrete. Bonding to the existing concrete walls was done by using 12-mm-diameter chemical anchors (eight bolts per plate) and epoxy adhesive conforming to ASTM C881-90 Type IV, Class C, Grade 3. To remove any rust, scale, oil, dust, grease, or other impurities that could prevent complete adhesion with concrete, the steel plates were grit-blasted just prior to bonding. Care was taken not to expose the plates after bonding to any disturbance until the adhesive attained its full strength.

## Repair to Existing Structure

A condition survey of the existing structure was carried out to locate defects that needed repair. Defects found were mainly cracks and hollow-sounding areas resulting from delamination of concrete. These defects were marked on the structure for necessary repairs.

All cracks having a maximum width of 0.1 mm or more were injected with low-viscosity epoxy resin. The entire length of the crack was injected, even if portions had a width of less than 0.1 mm. Where cracks penetrated the full thickness of the wall, injection was done on active cracks with the continuous presence of torsion as a source of cracking, hence its failure. The epoxy injection was carried out in conjunction with a strengthening system that should resist torsional stresses and hence eliminate the cause of the cracking.

Unsound concrete at hollow-sounding areas, on the other hand, was removed by chipping. The perimeter of each area to be chipped was saw-cut prior to chipping in order to minimize damage to the surrounding concrete. Concrete was always removed to such a depth as to maintain a clear gap of 25 mm behind the reinforcement bars. All edges were tapered to leave no square shoulders at the perimeter of a cavity. Chipped areas and rebars were treated in the same way as in the area of the platform/walls joint as discussed earlier. The chipped areas were then dampened and filled with shotcrete.

## Strengthening for Torsional Stress

The method used to strengthen the tubular box of the structure against torsional stresses involved installation of dowel bars, placement of main reinforcement and shrinkage reinforcement bars, and building a 240-mm-thick silica fume shotcrete jacket around the structure from the top of the foundation to the top of the wall passing through the cantilever walkway.

Dowel bars were inserted in the existing concrete walls and foundation to transfer loads from existing members to the new shotcrete walls. Holes 250-mm-deep were drilled into existing walls for the insertion of 16-mm-diameter, grade 60 deformed dowel bars using epoxy mortar. Standard 90° hook dowels were used and spaced 200 mm horizontally and 170 mm vertically (around 30 pieces per $m^2$) in a zigzag pattern. Hook dowels were used due to the impossibility of housing the required length of straight pieces in the specified thickness of the shotcrete wall. All holes were cleaned with water and left to dry completely before pumping the epoxy mortar. The nozzle of the pump of the epoxy mortar was provided with a tube of sufficient length to reach the end of the hole being filled so that the holes were filled from the closed end outward. During insertion of the bars in the partially filled holes, bars were pushed and twisted so that the mortar oozed out around them ensuring complete contact. Holes were also drilled in the foundation for the insertion of each main vertical reinforcement bar. After installing the dowel bars, the main reinforcement, comprising horizontal and vertical reinforcement, was installed. The horizontal reinforcement consisted of loop bars, 20 mm in diameter at 85-mm intervals.

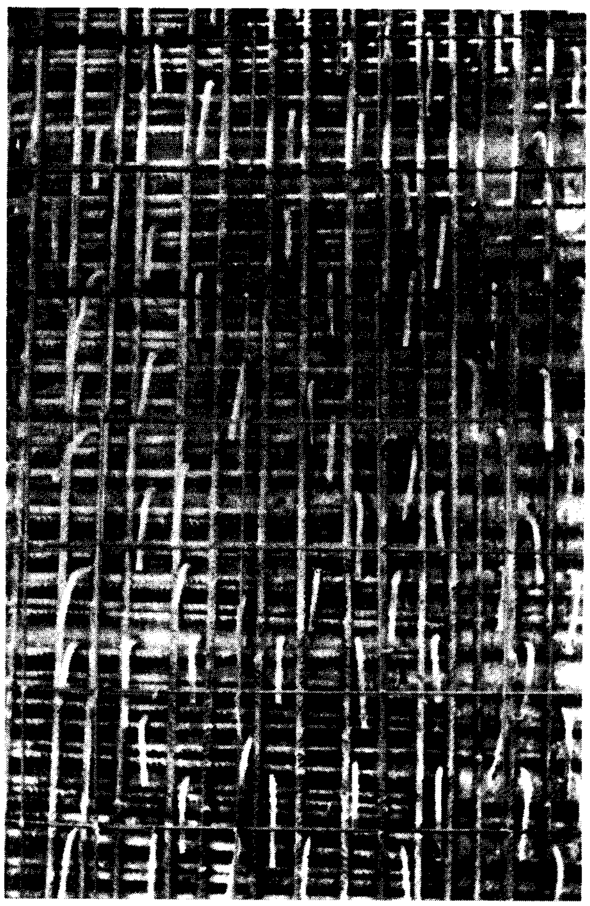

FIG. 7. Main and Temperature Reinforcement Bars Installed around Pier

Vertical reinforcement bars, 20 and 22 mm in diameter were then placed at 100-mm intervals, on the long and short sides of the structure, respectively. Vertical bars were inserted in the drilled holes in the foundation extending to the top of the wall through drilled holes in the walkway. Each vertical bar had one splice. Splices were located at various heights in order to avoid having a weak plane at the splice level.

In addition to the main reinforcement, a mesh of 10-mm diameter rebars, at 300-mm intervals both ways, was provided to serve as temperature and shrinkage reinforcement. This reinforcement was placed so that a 50-mm shotcrete cover was maintained. Fig. 7 shows the structure after all reinforcement bars were installed and prior to shotcreting. Intersecting bars were rigidly tied to each other with 16-gauge tie wire and adequately supported to minimize vibration during shocrete placement. Splices provided for vertical as well as horizontal rebars were approximately 1.2-m-long. This exceeded the maximum requirement as found from splicing calculations.

After all the reinforcement bars were in position, adequate ground wires were provided to establish the thickness, surface planes, and finish lines of the shotcrete. Concrete surfaces were then dampened in preparation for placement of shotcrete all around the exterior vertical surfaces of the walls and platform to build a new wall of 240-mm thickness. The wall was built using the dry-mix shotcrete process in several horizontal lifts approximately equal in height. The dry-mix process was selected for this job because it is more suited to restoration work than the wet-mix process.

Shotcreting work was carried out at night to avoid the high July daytime temperatures. Special attention was paid to the curing of the newly placed shotcrete. Prolonged curing was needed in view of the use of silica fume. Wet curing was started upon completion of each shotcrete lift and continued for 14 days. Curing water was supplied round the clock through a perforated tube running around the top of the lift being cured. Further, the shotcrete surface was covered during the curing period with one layer of burlap and one layer of polyethylene. Upon completion of curing, all surfaces of the pier were cement-washed to match the other buildings in the area.

## SUMMARY AND CONCLUSION

This paper presents a case study of concrete repair and strengthening work carried out on a defective reinforced concrete pier. A comprehensive investigation was carried out to determine the causes and extent of the problem and to define the remedial measures needed to rectify the problem and prevent future deterioration. The investigation revealed that the strength of the concrete was much below the requirements. The carrying capacity of the shotcrete for the vertical compression was found to be sufficient. However, that the concrete had a low strength and was subjected to conjoined tension due to torsion warranted strengthening for compression. That the system had failed in torsion clearly indicated that the resistance provided by the existing reinforcement was not adequate.

Two alternatives were considered. The first alternative was to remove the cover to the reinforcement and repair the distress using a silica-fume-blended cement shotcrete. The second alternative was to provide a silica-fume-blended cement shotcrete jacketing system around the structure. The second alternative was more technically feasible since it would provide more torque resistance and would not result in microcracking. Also it did not involve the labor-intensive chipping.

## ACKNOWLEDGMENT

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