ASSESSMENT AND REPAIR OF DISTRESSED KILN PIERS IN A CEMENT MANUFACTURING PLANT

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الخلاصـة :

لوحظ وجود تشققات قطريّة في بعض الدعائم الخرسانية في مصنع للإسمنت، مما حَثنا لإجراء دراسة تفصيلية لمعرفة وتشخيص أسباب هذه التشققات واقتراح طرق إصلاحها لضمان سلامة هذه الدعائم. إنَّ تقويم قوة الدعائم تطلبت إجراء مسح ميداني ومراجعة للقوى التي تؤثر على الدعائم. ولغرض معايير القوة ؛ أُخذت بعين الاعتبار الأخطاء خلال عملية الإنشاء والقوى المؤثرة أثناء تشغيل الأفران. وتُبيِّن الدراسة بأن الدعائم تتعرض لدوران ينتج غالباً بسبب فقدان اتصال أسفل الفرن بالقاعدة التي تدعمه، وتطلب إصلاح الوضع زيادة أبعاد الدعائم بقفص خرساني مسلح بسماكة (٢٥٠) مم، كما تمَّ وصل هذا القفص بالخرسانة القديمة وقاعدة الأساسات بواسطة قضبان حديد صلب مطلية بمادة أبوكسي، وكل قضيب قريب من الآخر، وتمَّ ملءُ الفراغات بين القضبان وكذلك تغطية هذه القضبان بالخرسانة النفاثة (شوت كريت).

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ABSTRACT

The observation of apparent structural distress in the form of well-formed diagonal cracks in a number of kiln supporting piers in a cement manufacturing plant led to a detailed investigation to diagnose the problem facing the piers and to propose remedial measures to enhance the safety of the piers. The strength assessment of the as-built piers included necessary field inspection and review of the forces acting on the piers. The strength criteria accommodated the adverse effect of possible constructional discrepancies and the operational loads. The study reveals that the piers are subjected to excessive torsional moment arising from a possible scenario of partial loss of contact of the tyre with one roller, and consequently the piers were required to be strengthened by providing additional wall thickness. The proposed strengthening scheme involves jacketing a new concrete wall of 250 mm thickness around the pier. The jacket was connected to the existing wall and foundation by closely spaced epoxy-grouted dowels and was cast by shotcreting using dry mix of Portland cement and aggregates.

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INTRODUCTION

Repair and rehabilitation work for concrete structures can broadly be classified into two categories: (*a*) repair in which damage due to deterioration and cracking is corrected to restore the original structural shape; and (*b*) repair which is necessary to strengthen the structural capacity of members whose load carrying capacity is either inadequate or suspect and whose strength has been severely impaired due to sustained damage. While the former is essentially a cosmetic restoration type work primarily aimed at compliance with the serviceability and structural integrity criteria, the second category deals primarily with the enhancement of strength and therefore complies predominately with the strength criteria. A repair scheme for this category is often challenging and enterprisingly innovative and demands a careful evaluation of the proposed methodology for its success. This paper deals with one such repair to strengthen the structural capacity.

In a cement manufacturing plant, some kiln-supporting piers of reinforced concrete box-type section showed visible signs of stress-induced distress after several years of operation. The worst pier appeared to have been twisted slightly at the top, causing well-formed diagonal cracks in the pier. A detailed investigation was initiated to diagnose the problem facing the piers and to propose corrective measures to restore the structural safety.

In this paper, the assessment of the structural distress has been presented to focus the need for adopting design criteria to accommodate the adverse effect of possible constructional discrepancies (and disparities) and the operational loads which may compromise the safety of a structure. The study reveals that the piers are subjected to excessive torsion resulting from the eccentricity of the load caused by possible loss of contact with support rollers. The required strengthening of the piers was achieved by jacketing a new thickness to the pier wall with shotcreting.

DESCRIPTION OF THE STUCTURE

The damaged structures involved are the kiln-supporting reinforced concrete piers located in a cement manufacturing plant. Each of the four parallel kilns operating in the plant is sloped at an angle of 1.7 degrees and is supported by six concrete piers. Figure 1 shows a photographic view of the structure. The kiln rotates at a slow speed, moving the charge to the lower end from the upper end where the raw materials are fed into the kiln. Natural gas flame at a temperature of around 1400°C is blown into the rotating kiln from the lower end.

A kiln, made of thick steel shell lined inside with refractory bricks, is supported at each pier by steel rollers through a steel tyre. The rollers are installed on a concrete pedestal cast on the slab at top of the pier. As the hot gases are fed from the lower end, the outside air temperature at the pier top is hottest (about 80°C) at the shortest pier and it gradually recedes at successive taller piers. As the kiln operates around

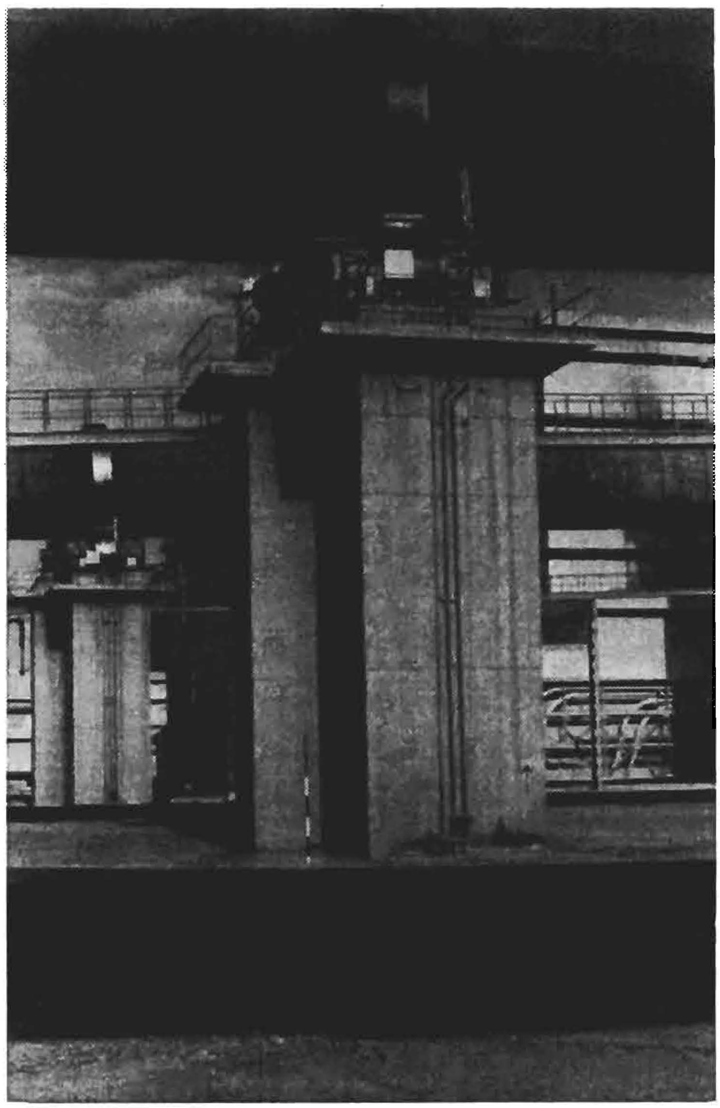

Figure 1. A View of the Kiln and the Supporting Pier.

the clock for several months before a shutdown, the piers at the lower end are therefore continually subjected to a hot thermal environment, the hottest one being at the shortest pier.

A typical pier is a hollow box-type section with an opening of 2.5 × 2.5 m at the ground level for access to the inside. The piers and their cross sections vary in shape and sizes, a representative one (Pier 1) is being shown in Figure 2. The 600 mm thick wall is orthotropically reinforced with vertical and horizontal reinforcement at each face. The top of each pier is covered by a 1500 mm thick RC slab built monolithically with the pier. This slab supports the kiln rollers and the necessary mechanical and electrical installations and equipment.

The pier is founded on a 1.4 m thick RC foundation slab, laid at a depth of about 1.4 m below the ground level with the top being flush with the finished ground level. Thus, there is no soil cover on top of the foundation slab.

NATURE OF THE PROBLEM

Apart from the isolated cases of corrosion damage sustained by different concrete elements, the piers were subjected to stress-induced cracking, as revealed by the development of numerous well-formed inclined parallel cracks. Evidence of shrinkage cracks was witnessed in all piers and these may have been nucleated with the stress induced cracks to form the final crack imprint. One of the piers appeared to have a small twist at the top due to the rotational displacement of the top slab supporting the kiln. This apparent twisting and the emergence of numerous well-formed diagonal cracks in the pier walls raised the concern of the owner about the structural safety of some of these piers.

A detailed investigation was therefore commissioned by the owner to diagnose the problem of the apparent structural distress and to propose, if necessary, remedial work to enhance the structural safety of the suspected piers.

DETAILED INVESTIGATION

Following a preliminary inspection and observation of the problems facing the piers, it was clear that the major problem of structural distress in the form of well-developed inclined cracks is attributable to stresses developed under the adverse loading conditions during the operation of the kilns. These cracks most likely nucleated with randomly oriented finer shrinkage cracks to form the final imprint. Apart from these visible cracks, isolated areas of pier walls, mostly at the bottom near the ground level and the top slab, were damaged due to corrosion of reinforcement (Figure 3). Based on the preliminary investigation, a detailed insitu investigation was planned which included, among others, a detailed condition survey, sample collections for insitu concrete strength and quality, chemical analysis of concrete samples, and collection of applicable data [1, 2]. The condition survey and the inspection revealed that the minor corrosion damage to the pier walls which was confined mostly to small areas near the base of the pier has not contributed in any way to the distressed condition of the box-shaped piers.

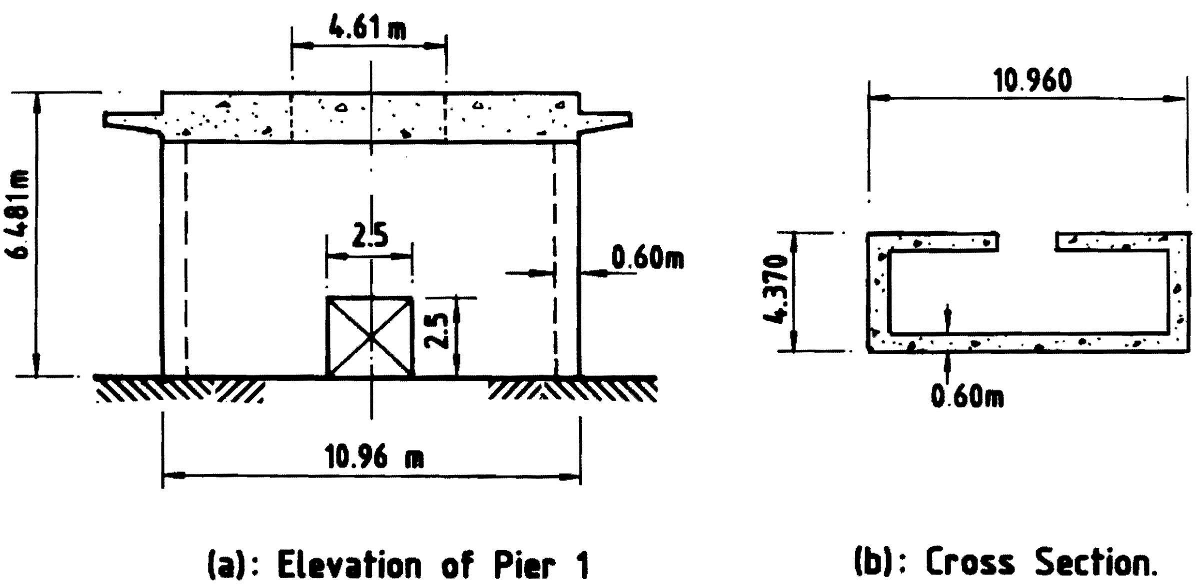

Figure 2. Pier 1: Elevation and Cross-section.

The focal point of the detailed investigation was therefore shifted to an assessment of the structural condition of the as-built piers, with the aim of arriving at a conclusive decision with regard to the need strengthening. Only this part of investigation and the proposed repair scheme to strengthen the piers are highlighted in this paper.

STRUCTURAL ASSESSMENT

The piers are subjected to operating loads from the slowly rotating, inclined, kiln and to environmental effects of temperature and wind. The high temperature of the kiln at the lower end produces an environment of excessive heat, to which the pier walls and the top slab are continually exposed. The expansion of the kiln at the operating stage and the contraction at the idle or shut-down stage (during routine maintenance) appreciably alter the values of forces exerted on the pier.

Possible constructional anomalies in the placement of the support bearings may also adversely affect the load actions. An unintended small misalignment or imperfect leveling produces an undesirable amount of twisting at the top of the pier. Evidence of some twisting action at the top of a pier led to this postulation that possible constructional discrepancy which may give rise to torsional moment should be considered in a conservative, safety-conscious assessment.

A review of the old documents revealed that a set of load combinations was contemplated and forces acting accordingly were conservatively estimated at the initial design stage. It was not clear if all of the load combinations had been used for the design of a pier itself. The calculated loadings, included the effect of the temperature of the kiln and the dynamic effect of the revolving kiln.

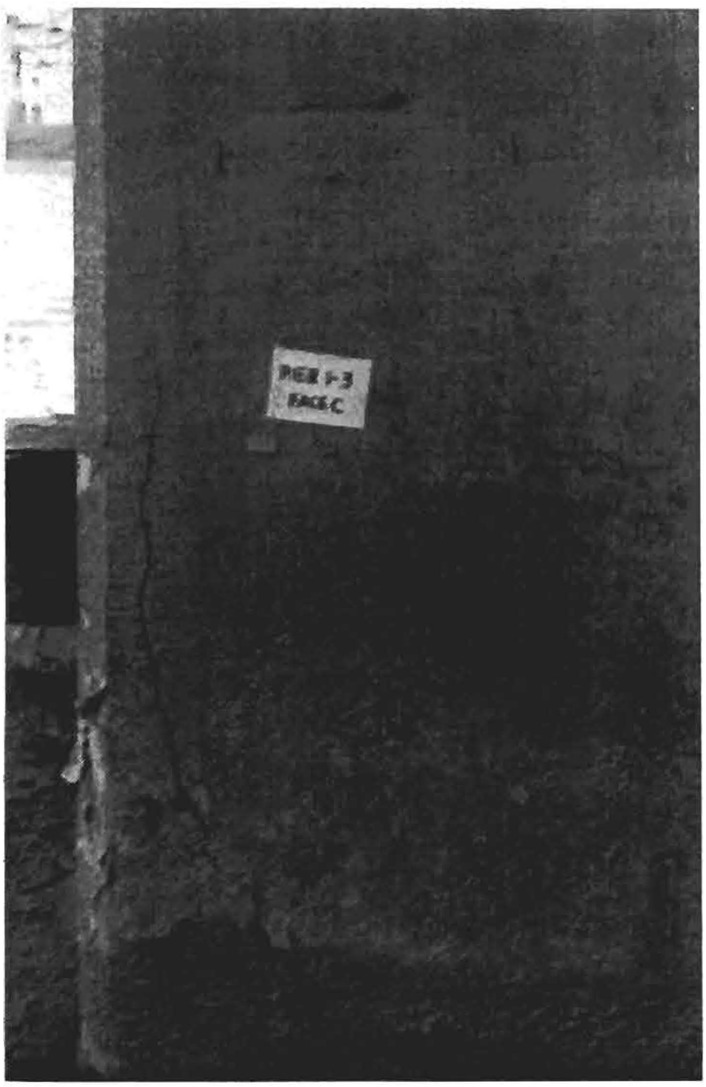

Figure 3. Corrosion Damage in Pier.

The forces in the three directions, vertical, longitudinal, and traverse, exerted by the rotating kiln on the roller bearings are shown schematically in Figure 4. The nomenclature of the forces shown is as follows:

W	=	maximum vertical load on each roller at different conditions (W varies)
$VP1$	=	total vertical load on rollers including all charges at the operating condition = $2W$
$VP2$	=	total vertical load on rollers considering the thermal deformation of the kiln shell = $2W$
$VP3$	=	vertical load on one roller, considering the unlikely extreme, possibility of the tyre having contact with only one roller = W
$HP1, HP2$	=	force acting in the longitudinal direction of the kiln
$HP3$	=	transverse force produced at the roller when the tyre contacts with only one roller. This force includes the effect of thermal deformation of the kiln shell.

The case of equal and opposite transverse forces (horizontal thrust) at the rollers when the tyre has perfect contact with both rollers does not have any effect on the pier (Figure 4), as the rollers are supported on a steel bed fixed to the top slab.

For the structural assessment, it was considered to be prudent to assume that some constructional discrepancy may have occurred in the vertical and horizontal alignment of the roller bearings. The original design postulated a possible worst scenario of loss of contact of one tyre with the roller for which the tyre reaction will be on one roller only. This extreme loading possibility introduces significant torsional moment at the pier top.

Load Combinations

The following three load combinations were considered in assessing the strength:

(*i*) Load Combination 1: VP1 + (HP1 or HP2) + wind load

(*ii*) Load Combination 2: VP2 + (HP1 or HP2) + wind load

(*iii*) Load Combination 3: VP3 + (HP1 or HP2) + HP3 + wind load.

Wind load was included as the kiln operates nonstop for a long period of time lasting sometimes more than six months.

The values of the design forces, VP1 to VP3 and HP1 to HP3, vary from pier to pier whose box-shaped cross sections are also different. For example, the design forces prescribed in the original design for the short-height Pier 1 at the lower end are as follows: VP1 = 11920 kN; VP2 = 15500 kN; VP3 = 7830 kN; HP1 = HP2 = 1800 kN and HP3 = 3640 kN.

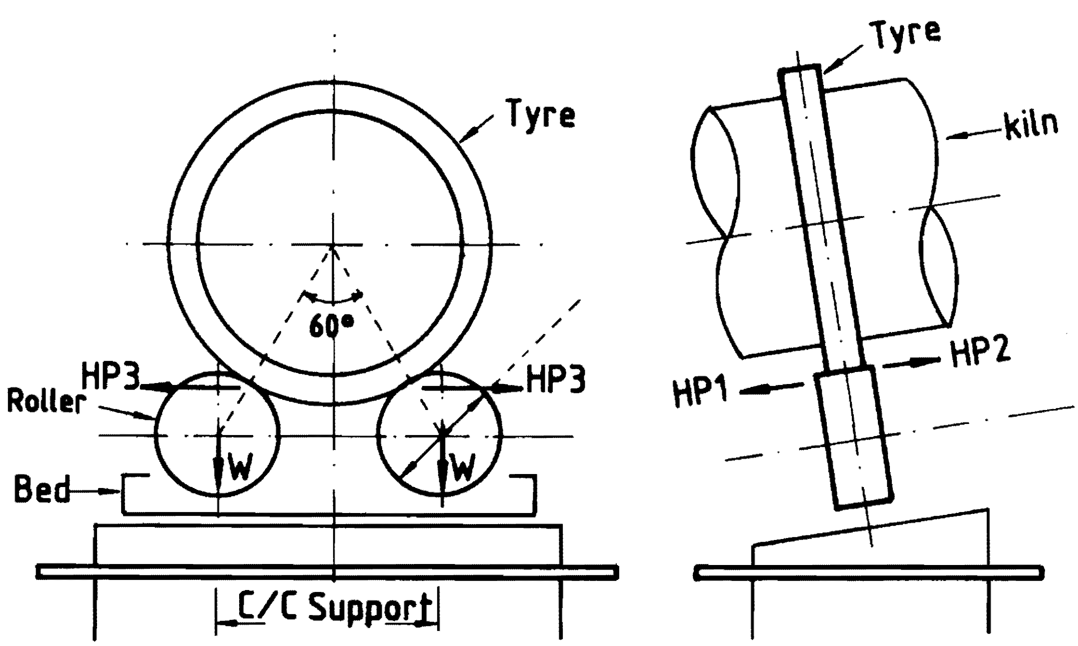

Figure 4. Forces Acting on Roller Bearings.

No independent verification of these design loads was made and these prescribed forces were used to check the strength of as-built piers using adverse loading conditions. Pier 1 is subjected to the largest forces among all piers and incidentally is also one of the highly distressed one. Figure 5 shows the location of the forces acting on the pier.

The Approach

In computing the stresses in the pier in accordance with a prescribed load combination, the most difficult problem faced was the loading of HP1 when the tyre makes contact with only one roller, as this force would then produce a high torsional moment. A preliminary calculation made it clear that the as-built piers cannot resist the full torsional moment produced by the eccentric longitudinal force resulting from a total loss of contact of one tyre, as the closed box section is considerably weakened at the base due to a large opening for access. This implies that the original design did not cater for such an extreme possibility.

A careful examination of the operating conditions and input from the owner convincingly implied that the extreme case of a total loss of contact of one tyre is highly improbable, as the kilns have been operating perfectly with both tyres in contact with the rollers. This led to the conviction that only a partial loss of contact is conceivable, and should therefore be considered, by assuming that the tyres do not exert equal forces on the two rollers while rotating. It was decided to consider, as the worst possible case, 50% of the torsional moment which would occur if the tyre has a complete loss of contact with one tyre during operation. Additionally, in order to account for the adverse effect of a small constructional discrepancy in the horizontal alignment of the bearings, a possible error of 3° in alignment as shown in Figure 6 was assumed. Inclusion of this introduces additional torsional moment due to transverse roller thrust.

A wind load of 450 N/m^2 on the structure was considered to act simultaneously in the lateral and longitudinal directions, considering a skewed wind direction. This caters for the possibility of windy conditions under which the kiln may operate.

The stresses at the operating loads were calculated using elastic properties of the thin-walled cross sections based on gross concrete sections and the applicable elastic formulas. The critical section at the base with the large opening as shown

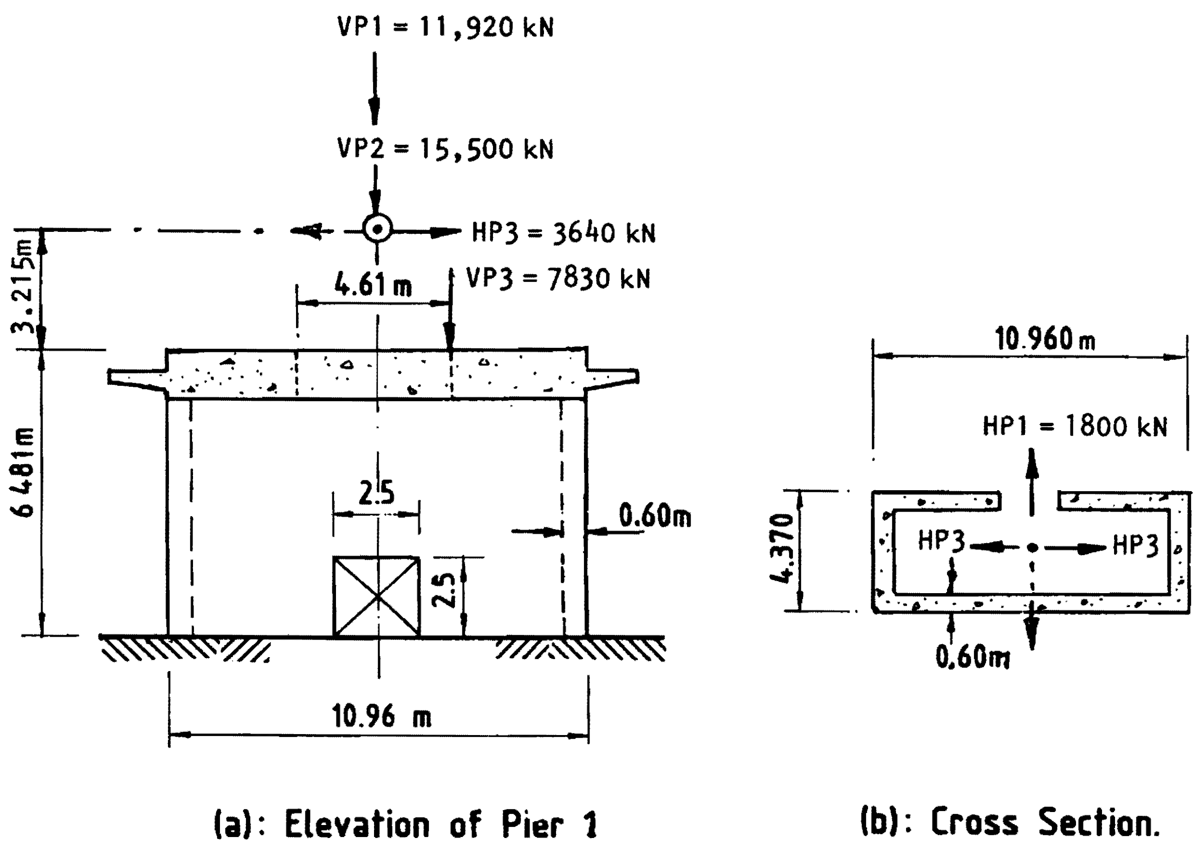

Figure 5. Forces Acting at Pier 1.

in Figure 2 was used. For torsional shear stresses in the walls, the cross section was treated as a thin-walled open section, due to the presence of a large hole of 2.5 m width in a longer side. The presence of this hole may not fully transform the closed-box section to an open one, because of the size, but this assumption is conservative, as it significantly reduces the torsional strength at this section. The cross section at the hole was considered to compute the maximum shear stress under the combined action of shear and torsion.

A pier is subjected to the combined action of axial compression and biaxial bending under normal condition. However, the consideration of some loss of contact of one tyre and the possible constructional discrepancy in the horizontal alignment produces torsion. This torsional moment proved to be the most critical, as the resulting combined shear stress from torsional moment and shear force far exceeded the permissible limit.

Assessment

In assessing the strength, the insitu concrete strength as determined from a number of 75 mm dia cores extracted from the pier walls was considered. The average core strength was found to be lower than the specified strength of 30 MPa and it varied considerably from pier to pier. This observation led to the conclusion that the as-built structure had inferior strength and that quality control in mix design and construction was not strictly adhered to. For Pier 1, the average insitu concrete strength f_c' was estimated as 16.9 MPa, reflecting a poor concrete.

The permissible limits of stresses were taken as follows [3]: tensile stress = $0.50\sqrt{f_c'}$, compressive stress = 0.45 f_c', and torsional shear stress in concrete = $0.15\sqrt{f_c'}$ where f_c' is in MPa. The torsional shear stress was reduced taking into account the interaction of combined shear and torsion [3, 4].

It was observed from computation that the only structural problem facing the pier was the excessive torsional shear stress arising from a consideration of some loss of contact with one roller. Although the magnitude of the twisting action cannot be estimated with a sufficient degree of reliability and accuracy, the presence of a twisting action due to the dynamic action of the revolving tyre, which tends to deliver uneven contact forces to the rollers, was evidently clear from the site observation. It was therefore decided that a certain amount of the twisting action at the top of the pier should be included for a conservative assessment of the strength of piers. Accordingly, a twisting moment of 50% of the maximum possible due to total loss of support contact with one roller was considered in the assessment. The computed torsional shear stress in piers (0.81 MPa in Pier 1) exceeded the permissible limits determined in accordance with the insitu value of f_c'.

The calculation showed that the pier walls need to be thickened to 850 mm to withstand the calculated torsional moment, requiring a new concrete layer of 250 mm concrete thickness whose 28-day compressive strength should not be less than 30 MPa.

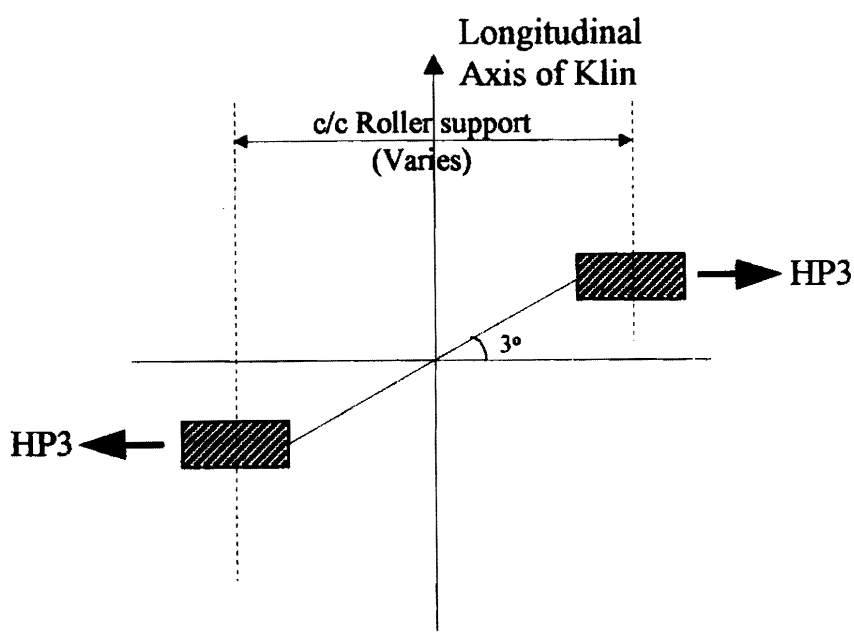

Figure 6. Possible Constructional Error in Horizontal Alignment.

THE REPAIR

The proposed repair which consisted of installing a new concrete jacket of 250 mm thickness around the exterior face of the box section was found to be the most suitable form of strengthening the piers under hot environmental condition. The details of the concrete jacket and the reinforcement are shown in Figure 7. The reinforcement provided in the wall, both vertical and horizontal exceeded the minimum requirement due to thermal and shrinkage actions.

In order that the new layer of concrete acts integrally with the old concrete with a no-slip interface, dowels of 16 mm diameter bars were installed at a horizontal spacing of 200 mm and a vertical spacing of 300 mm. This stitching of the old and new layer together by dowels ensures adequate bonding of the two layers.

The L-shaped dowels were inserted into the wall by drilling holes slightly larger in diameter to a depth of 150 mm and then anchoring the dowels within the hole by epoxy- grout. The detail of the dowels is shown in Figure 7. The new jacket was also anchored into the foundation slab by inserting dowels of 20 mm diameter into the drilled holes on the top of the foundation slab and then filling-in the holes with epoxy grout.

In order to minimize the shrinkage cracking and to provide a better bond with the old concrete, shotcreting was favored for the vertical construction of jacket, obviating the need of any formwork. The chosen method of construction also had the advantage of rapid concreting, as the shutdown period was limited to about only 25 days.

Repair Procedure

The repair procedure incorporated quality-controlled step-by-step procedure to ensure a sound construction. The pier walls were first sandblasted to a depth of about 6 mm to remove surface laitance and expose good surface. All visible, well-formed major cracks were then sealed by epoxy injection [5] and all defective and damaged areas were exposed by removing damaged concrete. Wherever corrosion damage existed, the exposed rebars were sandblasted to remove all rusts and following this they were coated with a coat of zinc rich primer.

The holes for dowels were drilled into the walls and the foundation slab by using hand-held percussion drills, avoiding the reinforcement in the walls. To avoid oozing out of the grout from the hole, the holes in the walls were drilled with a small upward inclination. The holes were cleared of dust by blowing air and then each hole was filled up with heat resistant epoxy grout following which the dowels were inserted into the grout-filled holes (Figure 8). The oozing out of grout ensured that the pocket with the dowel inside was completed filled with grout.

In order to determine the strength of the anchorage design and the effectiveness of epoxy grout, insitu trial pullout tests were conducted apriori for both horizontal and vertical dowels. Tests confirmed that design and construction were satisfactory to guarantee a minimum pullout force of 80% of the yield strength of the dowels.

Following the installation of all dowels, the reinforcement for the new jacket consisting of horizontal and vertical reinforcements (Figure 7) was caged around the pier, using dowels as their support (Figure 9). The new concreting was then accomplished by shotcrete using a dry mix. The mix proportions for shotcrete were determined by trial mixes and trial tests conducted on panels. Initial mix contemplated the use of small amount of coarse aggregates of 9.5 mm maximum size to reduce shrinkage characteristics of the mix proportion. However, as the loss of coarser aggregate from rebound was excessive, the maximum size of coarse aggregates was kept at 4.75 mm. The finally adopted mix proportion in a batch was as follows: 245 kg of coarse aggregate (maximum size of 4.75 mm) and 105 kg of fine aggregate, representing a proportion of 70% coarse, and 30% fine for the total aggregate and 100 kg of ordinary Portland cement (Type I). Water–cement ratio was maintained approximately at 0.38, by controlling the water flow at the nozzle. Experienced nozzle men were selected for the work by reviewing their workmanship and skill through a series of prequalification tests [6]. The shotcreting procedure essentially followed the recommendations of ACI 506 R [6].

Moist curing by sprayed water was activated soon after shotcreting and was continued for seven days. The specification required extraction of 75 mm core samples from the shotcreted jacket for verification of the workmanship and, more importantly, the required 7-day strength, which was specified to be a minimum of 25 MPa. The exterior faces were coated finally with a protective coating to provide additional protection against a chemical attack.

The most challenging task of this construction was the very limited time of about 25 days that were made available to the contractor during a shutdown of a kiln to carryout the repair work for the piers for this kiln. In total, the strengthening work was carried out for seven piers in four kilns. As the owner was reluctant to extend the shutdown of a kiln to avoid financial losses from nonoperation of the kiln, the contractor had to deal with an extremely tight schedule demanding a hectic operation.

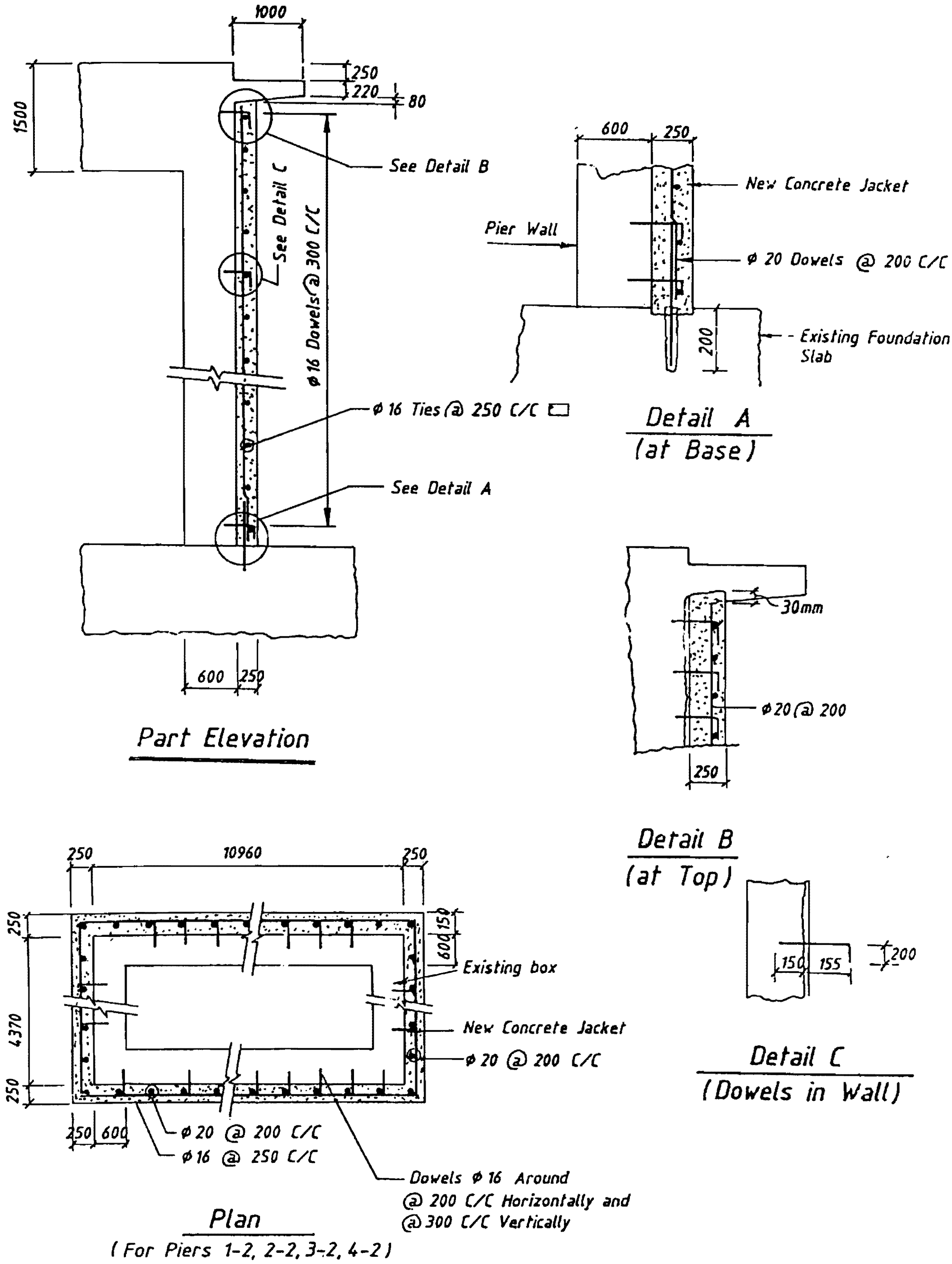

Figure 7. Detail of the Concrete Jacket.

Figure 8. Embedded Dowels in a Pier Wall.

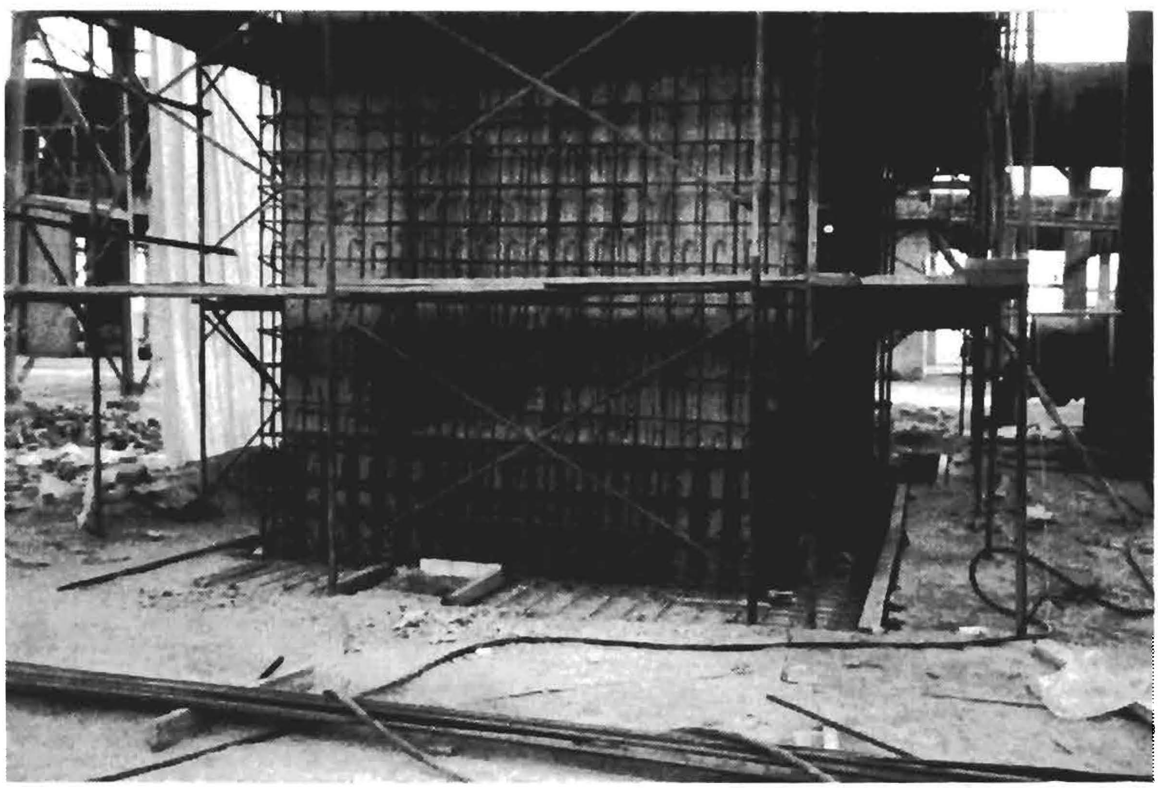

Figure 9. Placement of Reinforcement Around the Pier.

SUMMARY AND CONCLUSIONS

An assessment of the structural distress of kiln supporting piers in a cement plant has been covered in this paper, to highlight the adverse implication of possible constructional discrepancies and the operational loads on the strength. For sensitive structures where a small constructional anomaly may significantly impair the structural behavior, design criteria should prudently include such a provision.

The proposed repair scheme is designed to strengthen the pier to sustain torsional moments that may arise from the worst possible loading. The strengthening is achieved by adding a new concrete jacket of 250 mm thickness to the exterior faces of the box-shaped pier to increase the wall thickness. The jacket was stitched to the wall and foundation with numerous epoxy grouted dowels to ensure full composite action with the old concrete. The concrete was placed by shotcrete using a dry mix of cement and aggregates. The preference for shotcrete was dictated by the demand of a fast construction, good bond with the old concrete, and reduced shrinkage cracking.

ACKNOWLEDGEMENT

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