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Evaluation and repair of Algiers new airport building

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Abstract

This paper reports on the assessment study conducted by the authors on the new Algiers airport building. The evaluation approach included visual inspection of the concrete, seismic parallel method for piles testing and non-destructive testing of concrete with Schmidt hammer, ultrasonic pulse velocity measurements, cores testing, carbonation tests and also ambient vibrations of the structure. The diagnostic confirmed that the concrete was of low strength and showed many shortcomings such as inappropriate mix design with respect to coarse aggregate size of concrete resulting in honeycombing, construction errors such as lack of cover and the use of low slump concrete, poor placement and inadequate vibration. The repair work involved the application of ready mixed cement based polymer modified sprayed mortar with and without fibers on more than ten thousand square meters of honeycomb concrete, the injection of about five hundred linear meters of cracks and the repair of about one hundred square meters of corrosion damaged concrete at a cost of more than three millions US dollars.

Keywords: Concrete; Low strength; Honeycomb; Non-destructive testing; Cores; Repair; Sprayed mortar

1. Introduction

The new Algiers airport terminal is a reinforced concrete building of more than 80 thousand square meters of floor slabs supported on more than 1700 foundation piles designed to handle about six million passengers a year. It is composed of two symmetrical modules in the form of a circular arc of an internal radius of 132 m and external radius of 175 m and is located near the actual Algiers international airport. The project involved the casting of more than 100 thousand cubic meters of concrete. It was designed by a German consultant and work started in 1986 but was interrupted in 1996 for financial difficulties as the structure was almost complete. The building was left unattended for more than three years without any waterproofing and in 1999, a new firm was appointed to complete the work and hence an evaluation of the existing structure was necessary in order to propose ways of completing it and repairing deficiencies.

The assessment study was done from September 1999 to march 2000 by a team led by the first author to evaluate the structure and propose remedial work. The evaluation approach included visual inspection of the concrete, seismic parallel method for piles testing, non-destructive testing of concrete with Schmidt hammer, ultrasonic pulse velocity measurements, laboratory testing of concrete cores to determine compressive strength, carbonation tests and also ambient vibrations of the structure. A review of the stress conditions of the structure, as it stands, was also made.

2. Assessment results and discussion

2.1. Visual inspection

After an international bid, and for political reasons a local firm without any previous proven experience in this type of buildings was appointed to construct the building. A lot of deficiencies were observed during the construction stage. These deficiencies resulted mainly in segregation and honeycombing. Typical honeycombs are shown in Figs. 1 and 2 whereas Fig. 3 shows corrosion in a circular column due to water infiltration from a leaking expansion joint on the roof. Some thermal and shrinkage cracks were also observed on all

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Fig. 1. Typical honeycombs in a reinforced concrete beam.

Fig. 2. Typical honeycomb at a column-beam junction.

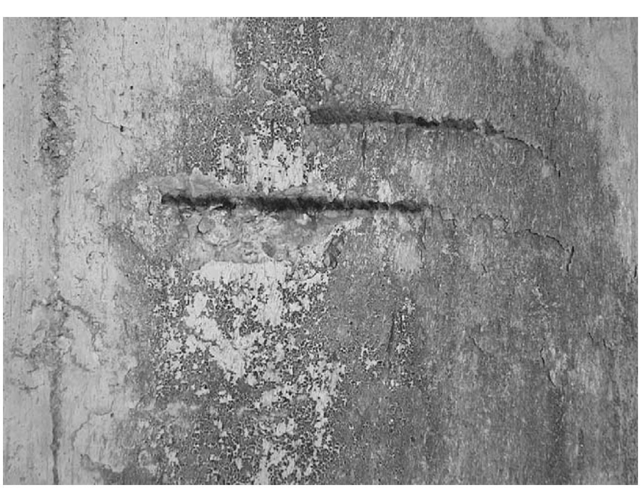

Fig. 3. Steel corrosion at the head of circular column.

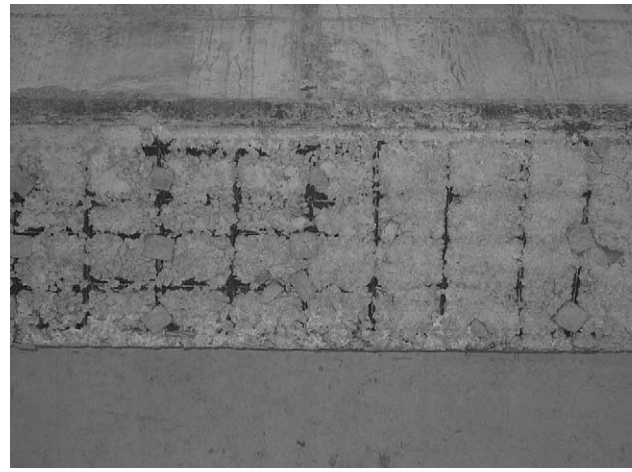

Fig. 4. Inadequate quality of concrete spacers.

25.7 long reinforced concrete beams with 2.0 m web depth. The quality of concrete spacers was inadequate (Fig. 4). The inspection of technical meetings reports, quality control of concrete strength files and records of tests on materials used confirmed that the concrete was of low strength. Many shortcomings were observed such as inappropriate mix design with respect to coarse aggregate size, construction errors such as lack of cover and the use of low slump concrete (80–100 mm) where plasticizer admixtures were rarely used, poor placement and inadequate compaction. The main cause of honeycomb concrete was severely congested reinforcing steel and inadequate spacing between parallel layers. Fig. 5 shows reinforcement congestion in a beam.

The concrete mix was not designed so that the largest aggregate size can pass between adjacent bars and between form and the reinforcement. ACI 318 [1] requires that the nominal maximum size of coarse aggregate shall

Fig. 5. Reinforcement congestion in a beam.

not be larger than 3/4 the minimum clear spacing between individual reinforcing bars and that clear distance between parallel bars in a layer shall be not less than the nominal diameter of the bar or 25 mm.

The drawings showed steel diameters not available locally and hence smaller diameters were used without amending reinforcement details resulting in a further decrease of bar spacing. In addition to that, workability was low and did not permit the concrete to be worked around reinforcement causing segregation particularly in shear walls and in the bottom of the sloped parts of the beams which seem not to be well vibrated by external form vibrators as clusters of coarse aggregates with little mortar were found (Figs. 6–8).

A typical example of honeycombing is that found on the bottom of 25.7 m long beams which are 0.5 m wide and with a web depth of 2 m and a 7.4 m long 55° sloped part. The beam drawings showed two to three rows of 28 mm bars both top and bottom and 10 mm stirrups spaced at 150 mm. The bar spacing at the bottom was between 55 and 85 mm with a cover of 45 mm. However, only 20 or 25 mm bar diameters were available on site and hence smaller bar spacing was provided as shown in Fig. 5 where some bars are in contact. The concrete mix design contained a 30 mm maximum aggregate size and a slump of 70–100 mm and hence concrete placement was difficult resulting in honeycombing. Honeycombing was also observed in the vicinity of wall penetrations. The honeycombs could have been avoided using appropriate mix design with lower aggregate size, appropriate workability and thorough consolidation.

It can be seen that forms were not well cleaned prior to concrete casting as steel tying wires and dust was seen on the bottom of beams (Figs. 9 and 10). Frequent changes in the source of constituent materials (mainly sand and gravel) were also noted.

Fig. 8. Clusters of coarse aggregates in a sloped part of a beam.

Fig. 6. Clusters of coarse aggregates in a shear wall.

2.2. Dimensional conformity

Linear dimensions of all structural elements were measured by an ordinary steel tape. Their relative positions, flatness of slabs and verticality and skewness of

Fig. 7. Clusters of coarse aggregates in a sloped part of a beam.

Fig. 9. Steel tying wires at the base of a beam due to non-cleaned forms.

Fig. 10. Steel tying wires at the base of a beam due to non-cleaned forms.

different elements were checked using a theodolite and a digital level. Dimensional tolerances as prescribed were found not to be respected. Some misalignment of beams were observed, flatness of slabs was not in conformity with specifications (up to a difference of 20 cm) and columns and walls were vertical but relative verticality between different levels was not respected (up 10 cm difference was observed). Nevertheless, dimensional non-conformities did not affect the structure as checked later but seems to affect the aesthetic and gives rises to difficulties in assembling manufactured elements.

2.3. Quality of concrete according to testing records

The specified concrete strength was either of a characteristic 200 mm cube strength (F'_c) of 25 MPa for most of the elements or 35 MPa for slender caisson deep beams. The cement used was either CEMI 32.5 or CEMII/A 32.5 with a content of either 350 or 400 kg/m^3. The French standards NF P 18-305 [2], requires that:

- no individual strength test (average of three cylinders) falls below F'_c by more than 1 MPa;
- average of n set of consecutive strength tests should equal or exceed $F'_c + 0.85\sigma$ for $F'_c \geqslant 30$ MPa and $F'_c + 1.2\sigma$ for $F'_c < 30$ MPa.

The analysis of more than 5000 cube test results available for piles gave an average compressive strength at 28 days of age of 31 MPa showing compliance with project specification requirements. However, the average standard deviation was 3.1 MPa showing inconsistency in the quality of concrete and irregular concrete production. More than 3300 cube test results of grade C25 concrete were analyzed and gave a satisfactory average compressive strength at 28 days of age of 30 MPa but a high standard deviation of 3.4 MPa showing a large variability of strength. It seems that concrete quality trend was not monitored to allow changes in concrete quality to be identified quickly. The average strength of about 180 test results of C35 concrete revealed an unsatisfactory average compressive strength of 30 MPa. Action should have been taken during construction stage to enhance the strength and quality of materials.

2.4. Concrete quality as assessed by non-destructive tests

In order to ascertain whether the in situ strength of concrete is acceptable for the designed loading system, 349 structural elements (beams, columns, walls or slabs) suspected, as low strength elements after the visual inspection, were tested by a combined method Schmidt hammer and ultrasonic pulse velocity. The use of the combined method was adopted because it is believed that it yields more reliable and closer results to the actual strength [3]. At least three sections in the most highly stressed zones were selected for each element under test for pulse velocity and at least nine measurements with rebound hammer test. More than 170 other elements around the suspected elements were also checked by Schmidt hammer.

The results confirmed the visual assessment of relatively low to medium strength of concrete on the elements tested. The estimated in situ strength based on both methods was comparable. The coefficient of variation for both concrete grades was high (15–20%). Pulse velocity measurements varied from 3300 to 4600 m/s indicating an irregular concrete production. More than forty elements showed an estimated in situ strength less than the characteristic strength and were checked by drilling cores on them.

2.5. Cores testing

126 cores were drilled from 42 elements suspected for low strength. The cores were 45×90 mm ones because of the reinforcement congestion. Cores were tested for compressive strength in dry conditions. The actual strength in the structure as well as the potential strength were calculated according to British Standards BS 6089 [4].

$$F'_c = 1.5/(1.2 \text{ estimated in situ cube strength}) \tag{1}$$

The cores testing results confirmed the low strength but most of the elements satisfied the ACI conditions [1] of structurally adequate as the average strength of three cores was at least 85% of F'_c and no single core was less than 75% of F'_c except five elements which were later demolished and rebuilt.

A good correlation was found with the non-destructive tests. This following linear relation developed by the first author for a similar grade concrete in construction projects of the same region based on more than 500 test results was found to best fit the in situ cube concrete strength:

$$F_c' = 0.39R + 11 \times 10^{-3}V - 26.2 \quad (2)$$

where F_c' is the compressive strength in MPa; R, the rebound hammer index; V, pulse velocity (m/s).

Fig. 11 showed the estimated concrete strength by this method is better than that proposed by other researchers [2,5].

2.6. Carbonation depth

Carbonation depth was measured on all cores by spraying with phenolphthalein solution on the freshly taken drilled cores which when carbonated will remain colorless as compared to uncarbonated concrete which turns purple–red. The average depth was compared to the steel cover which was measured by an electromagnetic cover-meter. An average carbonation depth of 10–25 mm was found compared to an average cover of 5–30 mm which showed clearly that most of the concrete cover is carbonated and action should be taken to passivate the reinforcing steel to preserve the structure life. It should be noted that there were many cases where reinforcement was seen at the bottom of the slabs and beams with no cover at all. As the carbonation depth (d) for a given set of environmental factors depend on the age of exposure (t) and the permeability of concrete (k):

$$d = kt^{1/2} \quad (3)$$

The permeability constant obtained from this equation varied from 5 to 10 mm/year$^{1/2}$, for an average age of 5–10 year, is in accordance with the findings of Wong et al. [6] for low strength concrete (20 MPa) who found k varying from 6.0 to 8.1 mm/year$^{1/2}$. Inadequate compaction and improper curing seems to be the cause of the carbonation and carbonation induced corrosion problems which started to occur in this relatively short time (5–10 years).

2.7. Quality of concrete spacers

Although the number of spacers used was very large, these were site made and of low quality. BS 8110 [7] prohibits the use of site made spacers and the concrete society recommends that concrete spacers be made of a minimum of grade C50 concrete [8]. These spacers are likely to be critical as they affect the quality of the concrete cover. Absorption tests were conducted on some of these spacers. The units were first dried to a constant weight (W_d) and then immersed in water at 20 °C until a constant weight (W_s). The water absorption (W_a) is given by:

$$W_a(\%) = 100((W_s - W_d)/W_d) \quad (4)$$

The results of tests on six spacers of varying dimensions from different locations gave a water absorption of 10–13% indicating a higher porosity and a higher open pores volume and hence a lower durability. It was proposed to take out all apparent spacers and apply a repair mortar on the beam bases to enhance concrete appearance and durability.

2.8. Piles testing

The building foundation consisted of 1730 reinforced concrete piles of 1.20 m diameter and of a depth which

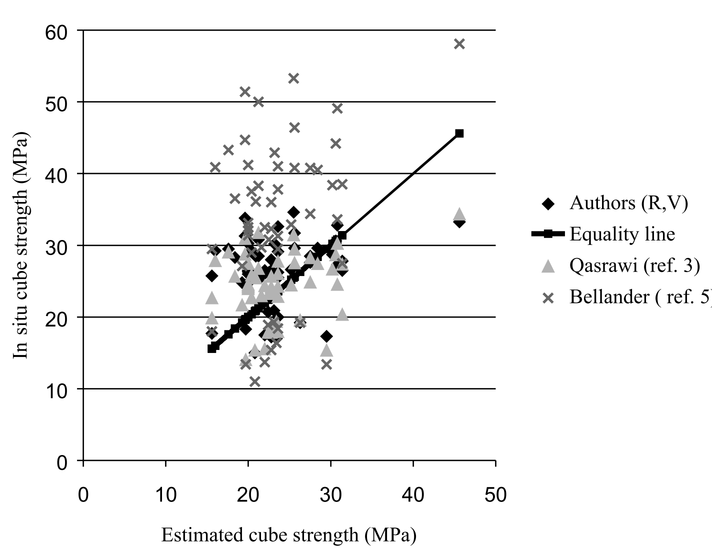

Fig. 11. Predicted in situ concrete strength using various methods.

varies from 14 to 28 m giving a total length of 31 492 linear meters. For access and cost difficulties, only eleven outer piles were tested by the parallel seismic method. Six bores of 24 m depth and five of 33 m depth were drilled, tubed and filled with water. The method consists of boring a 40–60 mm bore holes parallel to the pile to be tested at a distance not exceeding 0.5–1.5 m and to a depth slightly higher than the depth of the pile. The method is based on the measurement of the speed of waves through concrete and soil. A hammer is struck on top of the pile and time, for the impulse wave to reach a receiver which is moved progressively at a step of 0.5 or 1.0 m inside the hole, is monitored. Hence, the depth of the pile is measured and pulse velocity in both the concrete and the adjacent soil is found. Fig. 12 shows a typical depth–time curve for a tested pile. All piles tested were found to be cast to a depth exceeding the theoretical depth by 0.7–1.0 m. The ultrasonic pulse velocity in concrete varied from 4200 to 4800 m/s and hence the quality of concrete could be classified as good to excellent [9]. No defect or voids were observed in the tested pile.

2.9. Overall behavior of the structure

The concrete compressive strength and modulus of elasticity were fed into a numerical model of the structure and stress conditions on all elements were checked. The structural behavior of the building was checked in some specific points under ambient vibrations from an external source. The measured frequencies were approximately equal to those calculated by the numerical model with an accuracy of 3–17% (an average of 10%). The results confirm the rigidity and structural adequacy of the structure.

The most stressed structural elements were localized on the numerical model and stresses in concrete and reinforcing steel checked using the actual in situ strength of concrete in the elements as measured by non-destructive testing and/or cores. Also, all elements with either a low concrete strength or a dimensional non-conformity were checked. Most of these elements were found to be structurally adequate for supporting all combinations of forces and the load carrying capacity of the structure was not jeopardized. However, some low strength caisson deep beams failed to support the shear stresses at the supports and hence it was proposed the reduction of the weight of the initial reinforced concrete roof shell which was not built yet by at least 70% or strengthening the beams. The earlier solution was chosen by the owner and alternative solutions with different materials are under investigation. Two columns, one slab, one beam and a shear wall were demolished and rebuilt due to the very low strength of the concrete (less than 16 MPa).

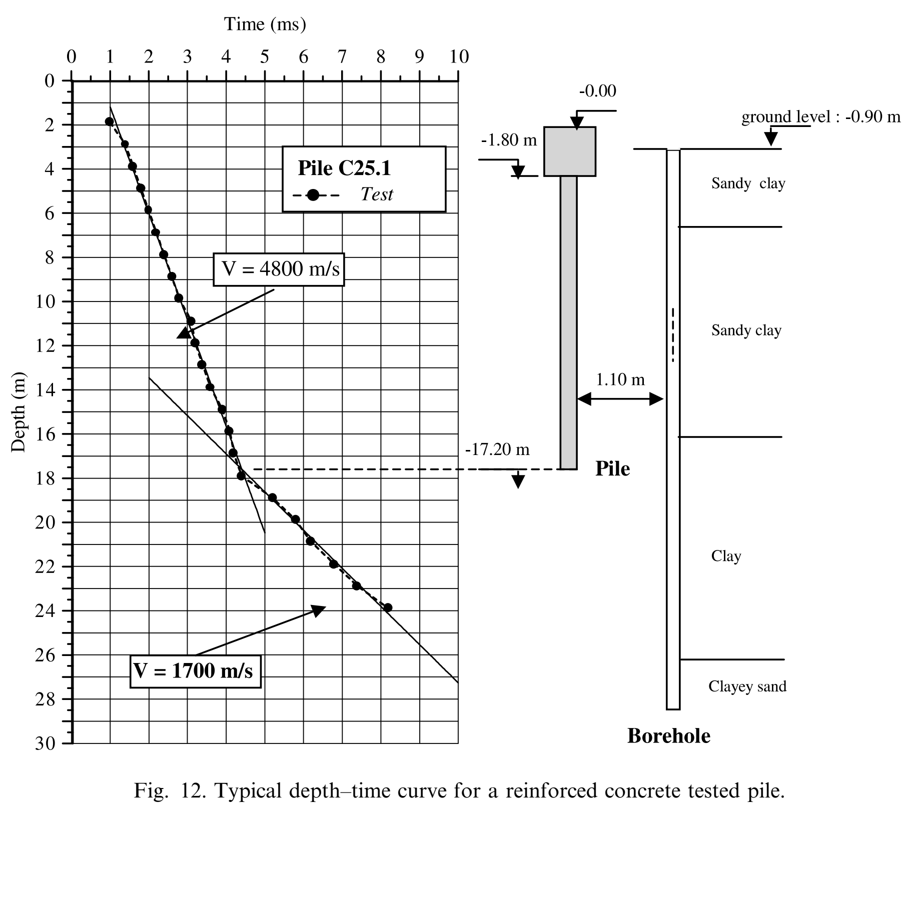

Fig. 12. Typical depth–time curve for a reinforced concrete tested pile.

3. Repair method and materials

3.1. Method and materials

The repair work proposed involved the application of sprayed mortar on more than ten thousand square meters of honeycomb concrete, the injection of about five hundred linear meters of cracks and the repair of about one hundred square meters of corrosion damaged concrete, more than 1500 linear meters of expansion joints and waterproofing in basement. The selection of the appropriate repair material was based on its intrinsic properties as well as its compatibility between the repair material and the existing concrete substrate. Hence, cement based repair materials were chosen for their low cost and compatibility. Recommendations for the application of the repair materials given by French standards NF P 95-101 [10] were followed. Due to the lack of experienced firms in the repair field, one of the major tasks was to train contractor's personnel to carry out the job exactly in the manner specified to ensure the repairs are durable.

3.2. Honeycomb and steel corrosion repair

The first step of repairing of honeycombs and segregations was surface preparation by removing laitance and loose concrete by either pneumatically driven or hand-operated lightweight jack hammers and rust from the reinforcement bars by sand blasting and final cleaning by water jet. Simple geometrical prepared surface shapes were used to avoid differential drying shrinkage. Following the cleaning of the reinforcement, the bars were treated with a cement based coating containing a corrosion inhibiting admixture. Old–new concrete interface are often critical spots and optimum roughened old concrete surfaces is believed to be the most important step in a successful repair work and hence was inspected thoroughly and feather-edging avoided [11,12]. No bonding agent was applied. Final substrate cleaning was carried out immediately before repair work to prevent contamination of the prepared surfaces and to get a saturated surface-dry surface. An imported commercial prepackaged cement based polymer modified mortar was mixed and cast according to the manufacturer's instructions and used for the repair of honeycombs. For deep honeycombs, a similar mortar was used but incorporating 7% by weight of cement of silica fume and also polypropylene fibers in order to achieve higher thickness in one layer of sprayed mortar. Mechanical and physical properties of these two materials are summarized in Table 1.

As shrinkage deformation is believed to be significantly affected by the curing environment especially under hot climate [13], curing was started immediately by sprinkling water for a minimum of three days. No shrinkage cracks were observed. In the case where shallow concrete cover was encountered with no apparent segregation or honeycombs and in order to reduce the undesirable noise and dust of concrete removal, a corrosion inhibitor was applied by a roller on the entire surface. The material is based on an organic and inorganic, film forming, blended amino compound that allows the inhibitor to diffuse through the concrete and form an adsorbed layer on the surface of reinforcement that displaces any hydroxides on the steel surface. A further coating was applied to concrete which will not be clad or rendered at a later stage to keep carbon dioxide from permeating into concrete and gives a better appearance.

3.3. Crack injection

Vertical parallel cracks spaced at 0.5–1.0 m with a width of 0.3–0.5 mm were observed along long deep beams coupled with larger cracks starting from the

Table 1
Summary of polymer modified cement mortars used characteristics

		Fiber reinforced cement based silica fumed polymer mortar	Cement based polymer modified mortar
Compressive strength (MPa)	3 days	16	20
	7 days	28	25
	28 days	40	36
Flexural strength (MPa)	3 days	3.6	3.0
	7 days	4.0	3.8
	28 days	6.5	6.0
Workability	Slump (mm)	100	80
	LCPC flow time (s)	5–10	5–10
Pullout strength (MPa)		3.0	2.9
Modulus of elasticity (GPa)		32.0	30.0
Shrinkage at 28 days (μ)		520	590

angles of large openings. These cracks are most likely due to early thermal stresses as concrete was cast onto a previously hardened layer. Normal drying shrinkage may have widened these cracks.

Prior to repair, ultrasonic pulse velocity measurements were taken and later compared to the data after the repair to provide an evaluation of the effectiveness of the repair. The cracks were repaired with resin injection using conventional hand guns. Holes were drilled at close intervals along the cracks, injection nipples were fixed at these intervals, and the surface of the crack between the nipples sealed. Resin was then injected under pressure from lower nipples to higher ones.

3.4. Quality control and cost

The high cost incurred in repairing the structure makes it essential to conduct a thorough quality control program on materials and workmanship. Poor on-site practices and indifferences to quality control during the repair installation, often produce a final product of dubious quality [14]. The compressive and flexural strength results of repair mortars are presented in Fig. 13 showing high variability of in situ repair mortars compressive strength from different batches though the strength was up to standards.

Bond test were conducted on site and in the laboratory according to the French standards NF P 18-852 [15]. The procedure calls for a partial depth core to be drilled in the test area to a depth extending into the original concrete. A circular steel plate with a threaded insert is then bonded with a fast-setting epoxy to the top of the unbroken core. Then the test is performed using the pullout instrument to apply a tensile force until failure occurs. The failure line is noted and the bond strength calculated. NF P 18-840 [16] stipulates that a minimum pullout strength of 1.5 and 2.0–3.0 MPa must be developed for non-structural and structural repairs respectively. The pullout strength (Table 2) varied from as low as 0.65 MPa to as high as 2.1 MPa but failure occurred most of the time in the concrete and rarely on the bond line or on the repair mortar. This led to a more frequent quality control tests on site for both materials and workmanship.

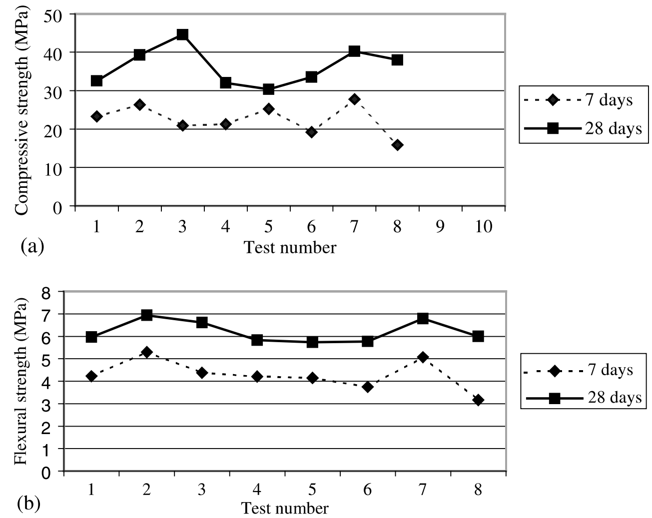

Fig. 13. In situ compressive and flexural strengths of the repair mortar used: (a) compressive strength; (b) flexural strength.

The repair was done by a local firm at a cost of more than three millions US dollars. Difficulties were encountered in doing the repair work with unskilled labor and inexperienced firm. The cost of repair was quite high and this shows that designers and contractors should take necessary steps to ensure durable structures.

4. Conclusion

The assessment study of this building showed many shortcomings on concrete strength and concrete quality. The main deficiencies observed were honeycombs and segregations. The evaluation of the building structure proved that the design of structural elements without precautions concerning concrete mix design and without

Table 2
Summary of some pullout (bond) strength results

Test number	Laboratory tests		In situ tests	
	Pullout strength (MPa)	Type of failure	Pullout strength (MPa)	Type of failure
1	1.27	Concrete	1.16	Concrete
2	1.38	Concrete	0.67	Mortar
3	1.63	Concrete	0.65	Partial
4	1.78	Concrete	0.97	Concrete
5	0.92	Interface	1.39	Interface
6	1.27	Interface	2.01	Mortar
7	1.22	Mortar	2.17	Concrete
8	1.43	Mortar	1.24	Interface
Average pullout strength (MPa)	1.36		1.28	

respecting standards recommendations concerning reinforcement details and in particular aggregate size, actual steel spacing and consolidation techniques can lead to costly repairs and delay. The repair work conducted showed that choosing a well established firm with proven experience is necessary and good quality materials and good quality control on site are important.

Acknowledgements

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