Condition assessment and renovation of an aged precast reinforced concrete multi-storey building

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Abstract. This paper assesses the condition of a 45 years old precast reinforced concrete multi-storey building and proposes innovative repair techniques for extending its service life. The residential building investigated consists of six floors, and each floor consists of 4 apartments with a total floor area of 900 m². Initially, visual inspection was conducted to identify the extent of deterioration in all parts of the building, followed by field non-destructive and destructive tests to determine the root causes of the damage. Moreover, analytical tools such as ETABS and SAFE design softwares were used by applying the same loading assumptions and material properties given by the designer, to check the compliance of the building with the safety requirements specified in ACI 318-14/SBC 304-18 building codes. The field and laboratory checks confirmed the occurrence of considerable degree of deterioration in some elements of the building due to reinforcement corrosion, alkali silica reaction (ASR), salt-scaling and leaching. The analytical checks revealed excessive deflections, due to a design error. Several advanced repair techniques including carbon fiber reinforced polymer sheets (CFRP), and steel jackets were implemented to restore the structural load carrying capacity of the columns and beams that suffered extensive deterioration. The excessive deflections were reduced below the code limit by supporting the deflected slabs using wide flanged I-section steel beams. Moreover, the Repair strategy proposed included, removal of the deteriorated concrete, coating the exposed steel bars with a protective anti-corrosive coating, and cathodic protection of the steel bars followed by injecting high strength cementitious grout.

Keywords: Durability, precast reinforced concrete, Repair, corrosion, strengthening.

1. Introduction
The general sector in the Kingdom of Saudi Arabia have witnessed a wide spread, and rapid development of precast concrete industry during the past 40 years. This could be due to its superior performance, lower construction time, high quality control and lower maintenance costs compared to normal concrete. Most of the precast concrete buildings located in Riyadh are subjected to one or different forms of deterioration during their service life. This is because of faulty construction workmanship, and arid weather conditions, where the temperature extremes reaches 52° C in summer and -1° C in winter, and the relative humidity ranges from 10 to 47%. Concrete defects occurs in different forms, ranging from fine to major cracks that may lead in some cases to serious damages or collapse. By studying those defects and their impact on the structure at an early age, they are more likely to be prevented.
Upgrading the design standards coupled with the damage caused by natural disasters, and aging of concrete structures necessitate the need for developing advanced constructions materials and innovative repair strategies for extending their service life at reasonable cost. Selecting an effective repair material that can enhance the performance of the structure would save the construction industry millions of Dollars. Rehabilitation of precast concrete constructed facilities results in economic benefits for the kingdom, and thus be in line with the knowledge based economy, on which the Saudi vision for the year 2030 is based. Recently, many researchers [1-8] have proposed several repair techniques for increasing
the life span of the constructed facilities. The most commonly used repair techniques proposed are: mortar and steel jacketing, ferrocement wire meshes, carbon/glass fiber-reinforced polymers (CFRP/GFRP) jacketing, FRP sheets and high-performance cementitious composites.

Tiago and Júlio [9] presented a case study on 20 years old reinforced concrete (RC), 16-storey building in Portugal. The first two floors of the building were severely damaged due to unusual floods in that area. Their repair strategy included constructing steel frames fixed rigidly to RC combined footing, to support the cantilever free area of the building. Brosens et al. [10] presented a case study on strengthening ribbed floor slabs of a 22 years old school building in Leuven, Belgium. Steel plates and carbon fiber reinforced polymer sheets (CFRP) were used to strengthen the slabs and increase their load bearing capacity from 3 kN/m² to 6 kN/m². As a result of the strengthening technique implemented, the school building was transformed into a city library with a considerable increase of load as a consequence. Nakamura et al [11] presented a new technology for strengthening and repairing bridges damaged by strong earthquakes in Japan. Some of the retrofitting methods developed include retrofitting road bridges by replacing hinged columns with hybrid towers and rubber bearings, and retrofitting long-span truss and cable-stayed bridges by installing dampers.

In the Kingdom of Saudi Arabia majority of the precast reinforced concrete structures are over 35 years old, and thus, there is a need for assessing their conditions for safety and economic reasons. The main objective of this investigation is to assess the damage of a typical precast reinforced concrete structure built in Eastern Riyadh in 1977, and develop innovative repair techniques for extending its life span. The multi-storey building investigated consists of six floors and each floor consists of 4 apartments with a total area of 900 m². Each apartment is constructed by using 18 precast solid slab parts as shown in Figure 1.

![Figure 1. A floor layout of the building investigated](image_url)

### 2. Condition Assessment

The condition of the structure was assessed by checking the design and as-built drawings, visual inspection and field tests [12-15] to identify the causes of structural and durability defects, for proposing the appropriate repair strategy. Concrete defects observed are classified into two main categories: structural and durability defects. The structural defects identified are the large deflections of some slabs and beams, whereas the durability defects identified are characterized by the occurrence of severe,
moderate, and fine cracks. The durability defects observed consist mainly of reinforcement corrosion, alkali aggregate reaction, salt scaling, and leaching.

2.1 Checking the structural design and as-built drawings

Slabs: The original design of the structure was checked following ACI318-14/SBC304-18 codes [16,17], using the same loading assumptions and material properties given by the designer. A 3D model was performed using the ETABS software to check the safety of all the structural members as shown in Figs. 2 and 3. The results showed that the design of slabs was not safe for the long-term deflection which is in agreement with the deflection measurements in the field. However, the safety of the slab against long term deflection was double checked using SAFE design software for each precast slab part. Slab parts with and without balcony were checked as shown in Figs. 4 and 5 and Figs. 6 and 7, respectively. The results of the long-term deflection calculated by the model are presented in Table 1 and compared with the field measurements results and the allowable values specified in SBC 304-18. The measured thickness of the designed slab was found 160 mm, which is less than the allowable value of 200 mm determined as per the code. The field deflection measurements of all the slabs in the building were performed using laser device. It is observed from Table 1 that the measured deflection of 75% of the slabs exceeded the deflections limits specified in the code. These results are in agreement with expected values based on the design check using ETABS and SAFE software.

Beams: the field deflection measurements using the laser device indicated that limited number of beams exhibited excessive deflection as shown in Table 2. Most of these values are in compliance with the deflections values predicted using ETABS and the values specified in ACI 318, and SBC 304.

Columns: the field inspection indicated that most of the columns were in satisfactory condition, and met the design requirements of the ACI/SBC codes. However, the rebound numbers and core tests showed low compressive strength of the concrete for some columns compared to the strength values required by the designers.

![Figure 2. Three dimensional model of the designed building using ETABS software](image1)

![Figure 3. Floor deflection results of the original design using ETABS Software](image2)
Figure 4. Design check of the slabs with balcony (S3,4,7,8,9,10,11,12,13,16,17,18) using SAFE.

Figure 5. Deflection results of slabs with balcony (S3,4,7,8,9,10,11,12,13,16,17,18) using SAFE.

Figure 6. Design check of the slabs without balcony (S1,2,5,6,14,15) using SAFE.

Figure 7. Deflection results of slabs without balcony (S1,2,5,6,14,15) using SAFE.
Table 1. Deflection results of the slabs.

| Element | ETBAS long term deflection (mm) | ACI/SBC Limit (mm) | Field Measurement of long-term deflection (mm) |
|---------|---------------------------------|--------------------|-----------------------------------------------|
|         |                                 |                    | Min   | Average | STD*  | Max   |
| S01     | 25                              | 20                 | 11    | 19      | 4.2   | 27    |
| S02     | 25                              | 20                 | 11    | 21      | 4.3   | 28    |
| S03     | 26                              | 20                 | 14    | 22      | 5.8   | 34    |
| S04     | 24                              | 20                 | 7     | 14      | 4.4   | 22    |
| S05     | 23                              | 20                 | 18    | 23      | 2.9   | 29    |
| S06     | 25                              | 20                 | 16    | 23      | 3.3   | 29    |
| S07     | 30                              | 20                 | 11    | 23      | 5.9   | 33    |
| S08     | 33                              | 20                 | 18    | 27      | 4.3   | 36    |
| S09     | 46                              | 20                 | 19    | 25      | 4.5   | 40    |
| S10     | 32                              | 20                 | 6     | 17      | 5.9   | 26    |
| S11     | 32                              | 20                 | 0     | 20      | 6.1   | 31    |
| S12     | 32                              | 20                 | 15    | 22      | 3.1   | 27    |
| S13     | 32                              | 20                 | 4     | 23      | 5.7   | 33    |
| S14     | 30                              | 20                 | 5     | 18      | 5.3   | 28    |
| S15     | 29                              | 20                 | 12    | 19      | 4.5   | 29    |
| S16     | 37                              | 20                 | 10    | 16      | 3.4   | 24    |
| S17     | 37                              | 20                 | 11    | 19      | 4.2   | 26    |
| S18     | 34                              | 20                 | 16    | 22      | 4.1   | 30    |

*STD: Standard deviation

Table 2. Deflection results of the beams.

| Element | ETBAS long term deflection (mm) | ACI/SBC Limit (mm) | Field Measurement of long-term deflection (mm) |
|---------|---------------------------------|--------------------|-----------------------------------------------|
|         |                                 |                    | Min   | Average | STD  | Max  |
| B01     | 17                              | 21                 | 1     | 4.9     | 2.1  | 9    |
| B02     | 18                              | 21                 | 0     | 5.5     | 3.6  | 14   |
| B03     | 6                               | 15                 | 0     | 2.8     | 2.2  | 7    |
| B04     | 7                               | 15                 | 0     | 4.0     | 3.2  | 10   |
| B05     | 14                              | 20                 | 0     | 4.8     | 2.6  | 9    |
| B06     | 16                              | 20                 | 0     | 5.2     | 4.8  | 21   |
| B07     | 6                               | 20                 | 0     | 3.2     | 2.7  | 10   |
| CB07    | 22                              | 10                 | 6     | 10.9    | 3.3  | 18   |
| B08     | 6                               | 20                 | 0     | 3.0     | 2.5  | 10   |
| CB08    | 21                              | 10                 | 6     | 13.7    | 3.7  | 21   |
| B09     | 18                              | 25                 | 0     | 3.9     | 3.0  | 10   |
| CB09    | 10                              | 10                 | 2     | 7.9     | 4.1  | 15   |
| B10     | 16                              | 20                 | 0     | 5.0     | 3.2  | 11   |
| B11     | 40                              | 28                 | 14    | 18.9    | 2.7  | 25   |

2.2 Concrete durability defects
The major concrete durability defects observed in the building included defects due to reinforcement corrosion, defects due to alkali aggregate reaction, defects due to salt scaling and leaching, and minor defects due to drying shrinkage.
2.2.1 Defects due to reinforcement corrosion: moderate to severe corrosion cracks were observed in some locations such as slabs, corners of columns, and tension side of beams as shown in Figs. 8 through 11. The extent of the corrosion damage shown in the photos demonstrated a lack of maintenance for the building. The visual inspection indicated that the main cause of corrosion was the leakage of water from the pipes in the bathrooms. Moreover, the field tests indicated high concentration of chlorides in the concrete. Some of the beams and columns exhibited major longitudinal cracks along the main tension reinforcement as shown in Fig. 12 and 13. This is caused by the volume expansion induced by the corrosion products of the main steel bars.

Figure 8. Steel corrosion at the middle of slab
Figure 9. Steel corrosion at the corner of slab
Figure 10. Steel corrosion at the tension side of beam
Figure 11. Steel corrosion at the corner of a column
2.2.2 Defects due to alkali silica reaction, salt scaling and leaching: Fine to moderate map cracks were observed in the bathrooms areas and on the surfaces of some slabs as shown in Fig. 14. This is caused by the reactive silica present in some aggregate particles in the concrete mix. Some slabs in the bathrooms exhibited scaling and lime leaching as shown in Fig. 15, because of the hot and humid environment in such area.

3. Repairing Strategy
The repair strategy proposed in this investigation focused on reducing the excessive deflections of the deflected slabs, and repairing the concrete durability defects appearing on the surfaces of various members of the building.

3.1 Structural repair of deflected slabs
The excessive deflections of the slabs and balconies were controlled by using wide fanged I-section steel beams of size HE240A and HE160A respectively as supports on the tension side as shown in Fig. 16. The safety of the strengthened slabs was double checked using the SAFE software as shown in Figures 17 and 18. Moreover, the effectiveness of the strengthening schemes was confirmed by comparing the deflections of the slabs listed in Table 3 before and after installing the steel beam supports.
3.2 Structural repair due to concrete durability defects

Most of the concrete deteriorations occurred in the building were mainly caused by chloride corrosion. Several techniques are proposed in this investigation to counteract the effects of reinforcement corrosion [18-20]. To gain access to the reinforcing steel, mechanical means such as pneumatic hammers or milling are used to remove the concrete cover. Removing dust by sand blasting and wire brush; coating the exposed steel bars with a protective anti-corrosive coating; applying a high strength cementitious batch to ensure that the steel is back in a high PH alkaline environment. To ensure a perfect prevention against the ingress of chloride ions into concrete, cathodic protection of steel bars should be conducted. In cathodic protection, the corroding areas of steel are made cathodic by the supply of electrons from an anode applied either to the concrete surface or embedded. At areas where the reinforcing bar has corroded beyond 20% of the original size, additional bar shall be spliced in to guarantee structural integrity.

![Figure 16. Architectural plan for a repaired unit of the building](image)

Table 3. Deflection results of the beams.

| Slab                          | Before Repair | After Repair |
|-------------------------------|---------------|--------------|
|                              | Span (m) | Deflection using SAFE (mm) | Span (m) | Deflection using SAFE (mm) |
| Slabs without balcony         |          |             |          |                          |
| S3, S4, S7, S8, S9, S10, S11, S12, S13, S16, S17, S18 | 4.79 | 64 mm | 2.40 | 10 mm |
| Slabs with balcony            |          |             |          |                          |
| S1, S2, S5, S6, S14, S15      | 4.69     | 48 mm | 2.35 | 4 mm  |


The structural members that suffered extensive concrete damage due to reinforcement corrosion were strengthened further using advanced composite materials. The damaged columns were strengthened by wrapping 2 layers of unidirectional carbon fiber sheets around the column sides. Moreover, some of the columns were strengthened using 10 mm thickness high strength steel jackets. The beams that suffered severe deterioration were also strengthened by installing 2 layers of U shaped unidirectional CFRP jackets as shown in Fig. 19. The initial stage of repairing part of the slab subjected to corrosion damage is shown in Fig. 20.

3.3. Structural repair of slabs against ASR, and salt scaling damage

The slabs exhibiting extensive ASR map cracks, and salt scaling, are repaired as follows: a) identifying the parts of the slabs subjected to damage b) chipping away the cracked concrete using saw-cutting
hammers c) removing the dust by sand blasting and wire brush d) saturating the surface with water e) using a bonding agent for promoting adhesion between the repair overlay and the original concrete f) covering the cut area of concrete with appropriate wooden forms g) pouring a high strength cementitious mixture to create an impervious water tight overlay and curing it for one week. For the slabs that are severely damaged by ASR and salt scaling, external strengthening of the repaired concrete defects with CFRP sheets is recommended.

4. Conclusions
A study was carried out on a multi-storey, 45 years old, precast reinforced concrete residential building in Riyadh. The main objective of the study was to renovate the building, and extend its life span. The following conclusions can be drawn from this study:

1) The preliminary checks and visual inspection of the building indicated that some of its parts were severely deteriorated due to chloride corrosion, alkali aggregate reaction, and salt scaling.
2) The destructive and non-destructive field tests confirmed the occurrence of high level of chlorides and reactive siliceous aggregates in the concrete, low concrete compressive strength, and poor workmanship during construction.
3) The leakage of water through some of the water pipes embedded in the slabs of the bathrooms coupled with the existence of chlorides in concrete, induced moderate to severe chloride corrosion of steel reinforcement in some parts of the building.
4) The safety of the structure was double checked using modern design softwares. The analytical investigation showed large deflections of some slabs because of design error.
5) The repair strategy included the following steps:
   a) controlling the deflections of the slabs using wide flanged I-section steel beams,
   b) strengthening the severely damaged members using CFRP composite sheets, and steel jackets,
   c) removing the concrete cover, removing dust by sand blasting and wire brush,
   d) coating the exposed steel bars with a protective anti-corrosive coating,
   e) preventing chloride corrosion from spreading to the nearby bars through cathodic protection,
   f) pouring high strength cementitious mixture over the damaged areas, followed by sufficient curing.

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