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## STRUCTURAL STRENGTH EVALUATION BY NON-DESTRUCTIVE TESTS OF RC SLAB STRUCTURE AND ANALYSIS OF THE FAILURE CAUSES OF CONCRETE ROOF SLABS

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**Keywords:** concrete structures, field tests, load tests, core test, in situ tests, concrete slabs

### ABSTRACT

The main objective of this paper is to evaluate the safety and serviceability of an existing structural system by Structural Strength Evaluation by Non-Destructive Tests for Reinforced concrete (RC) Slabs Structure, and Analysis of the failure causes of (RC) roof slabs.

In this paper, three cases are studied on roof slabs (two-way slab, one-way joist slab system) of three villas in Kuwait. These villas were built in 2017, 2007 and 1997, respectively.

Weakness in the strength of the concrete and the deflection in the roofs is observed, as well as appearance of cracks in some of the slabs and in some of the beams.

The tests (load test, core test) are carried out according to ACI-318 so as to make known that they match the strength of the existing concrete and if it is safe or not.

In the end of the paper the tests gave good results to assess the safety of the structure. Two main reasons for the failure of two slabs are found, the first is poor implementation by the contracting company and non-compliance with the required technical specifications, and the second reason is fault in the design and weakness in the supervision executed by the engineering consulting.

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## 1. Introduction

The need to improve the ability of an existing building to withstand external loads arises usually from the evidence of damage and poor behavior during a finishing period or after.

It can be known also from calculations or by test or other methods what is the bearing of buildings for additional load that have been damaged in other places.

The method of repair and strengthening would naturally depend very largely on the actual forces in the building as well as the structural scheme and materials used for the construction of the building in the first instance, the technology that is visible to adopt quickly and on the amount of funds that can be assigned to the task, usually very limited.

Some methods like splints and bandages, wire mesh with gunite, epoxy injection, etc., have already been tried and applied in a few countries for repairing as well as strengthening damaged buildings. For that it is very important to evaluate the exact bearing strength for each structural member by Non-Destructive Tests, which is relevant for a variety of reasons like the poor implementation by the Contracting Company and non-compliance with the required technical specifications, as well as fault in design and weakness in supervision by the Engineering Consulting Office.

This paper describes an on-site evaluation of several roof slabs in three villas, the first of which is one-way joist slab and the second and third are two-way solid slabs, in order to introduce the principles and results of the method of load testing in the context of potential projects. These case studies represent an ideal test.

The first structure has weakness and deflection equal to 7.00 cm in the mid span of slab (12.00\*10.50 m) which happened after the removal of the scaffold before carrying any load (finish and live load), whereas the second and third structures present deflection in the center of slabs and diffuse cracks in the top and bottom surface of slabs and beams (6.00\*5.00 m) and another slabs dimensions (6.90\*5.25 m, 6.00\*6.20 m).

This paper focuses on determining the load level for each structure and finding out the causes of the failure. Special considerations are related to the design and the implementation of the conduct of this type of load test and core test is presented and discussed. Also, its focus is on the evaluation criteria and their significance, limitations and applicability.

## 2. Literature Review

Load testing of concrete structures in the United States is a century old tradition with one of the earliest well-documented cases found in the 1890s Birkmire 1894 [1].

The American Concrete Institute began formalizing load test procedures for concrete structures in 1920 [2] ACI 1920 [3]. At that time, the evaluation criteria for passing the load test focused on maximum deflection under sustained load combined with the recovery of deflection after the test load was removed. Subsequent Codes ACI 1936 defined the deflection evaluation criterion as a function of the span length squared and divided by the total depth of the member cross section [1]. This form of the deflection criterion is still in effect ACI 2005. Notable investigations into load testing of concrete structures documenting the practice of the last decades can be found in the literature of Fitz Simons and Longinow 1975, RILEM 1984, Bungey 1989 [1, 4 – 7].

The load test method is a relatively recent development and therefore only a limited number of reported case studies exist: Gold and Nanni 1998, Nanni and Gold 1998a,b, Mettemeyer and Nanni 1999, Galati et al. 2004, Casadei et al. 2005 [8, 9].

This method attempts to make use of advances in technology e.g. equipment, instrumentation and analytical tools to provide a safe and reliable procedure for structural evaluation consistent with contemporary construction and engineering practices and social needs.

### 3. Objectives of the Paper

The objectives of this paper are:

- To evaluate the structural efficiency of the building;
- To do development of in-situ evaluation methods for RC structures;
- To reach the real failure causes of the roof slabs and other structural elements;
- To verify the appropriate treatment work for the existing cracks and deflection;
- To collect the data that is necessary to perform the researches in the future;
- To provide analysis of the experimental results to professionals with evidence of the validity of in-situ assessment;
- To know the effect development of construction technology over a two-decade period in the building materials and methods of construction.

## 4. First Case: Villa in Mishref Suburb in 2017

### 4.1. Description of the Building

The first case was built in Kuwait city in 2016 (Fig. 1, 2, 3) on an area of 400 m<sup>2</sup> of land, consisting of four floors: basement, ground, first and second. The building was built of Reinforced concrete (RC). The structural system consists of columns and beams and slabs. There are two types of roof slabs used in the building, the first one is solid slabs and the second is *one-way joist slab*. That is usually used for wide spaces such as halls that are performed without columns in the middle of it. In this case we will focus on two slabs of the building that have clear crack and deflection problems; the first and second floor roofs are a *one-way joist slab system RC* concrete.

### 4.2. Preliminary Investigations

The following summarizes the preliminary assessment of the structure and the sources for the information used in designing the load tests.

#### 4.2.1. Structural Geometry and Material Characteristics

The structural geometry including columns locations and members sizes was determined from the engineering drawings attached, the slabs of the roof are two types, as we mentioned before. The second type of the slabs is one-way joist slab, the Dim. of the slab is (12.00\*10.50\*0.50 m) (*length x width x thickness*) and the section of design rib is shown in Fig. 2.

The material characteristics were provided by consulting the design office. The specifications indicated a compressive strength of concrete of 300 kg/cm<sup>2</sup>, and minimum yield strength for the mild reinforcement of the steel is 4000 kg/cm<sup>2</sup>.

#### 4.2.2. Initial Observation and Visual Inspection

A visual inspection was carried out to inspect the condition of the existing building structural elements, and then if needed to identify the method of loading and the slab to be loaded.

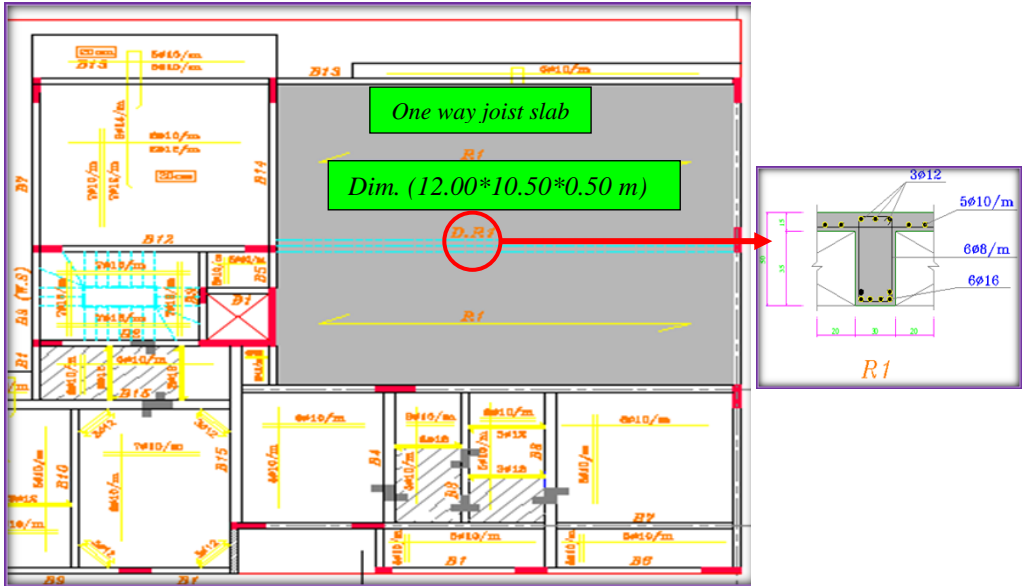


Figure 1. First and Second Floor Roof Slabs plan      Figure 2. Rib Section R1 (As Designed)



Figure 3. Elevation of building

From the visual inspection, clear cracks were found on the first and second roof slabs of the building. However, no significant cracks were found on other structural members.

At the roof of the basement and the ground floor there was no notice of a deflection of the slab or what is alleged to worry; on the contrary, in the roof slabs of the first and second floors deflection was apparent with the naked eye in the joist and edge beam and when measured was about 6 – 7 cm in the middle of the span. Also, cracks appeared in the joists and beam in addition to the vibration of the slab, therefore it was important to check the safety of the construction of the roof, which required carrying loading test for both slabs (Fig. 4) (One-way joist system with a 12.00 m length, 0.50 m thickness, static load test).

### 4.3. Description of the Load Test

#### 4.3.1 Load Test Configurations

The load tests were performed in a down load method; in particular, by distributing the load of the concrete blocks on the whole area of the slab in uniform manner with known weight to check the reaction on the slab.

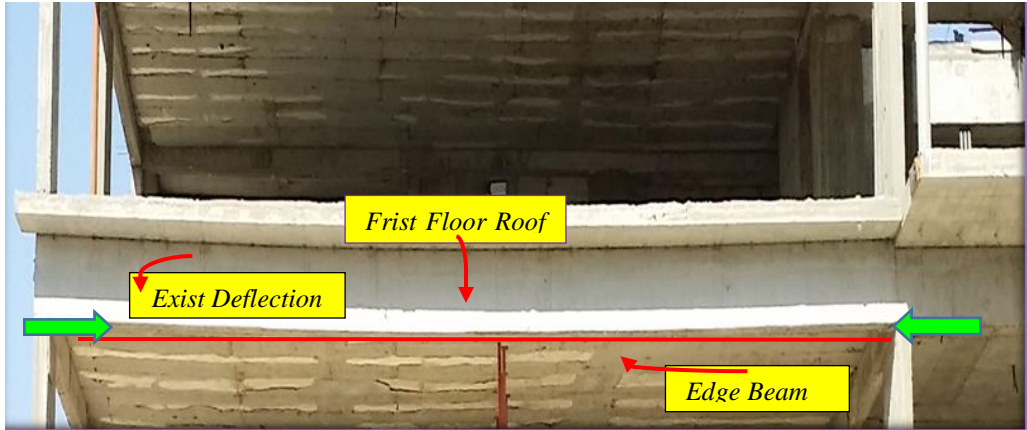


Figure 4. Slab and edge beam with 6 – 7 cm deflection in the middle of the span

#### 4.3.2. Deflection Measurement

Deflection measurement was taken in 5 different locations for by installing dial gauges, so a significant portion of the floor was monitored during the load test. Deflection measurements were taken with part from 100 of millimeter mounted on tripods on the level below the slab being tested. The five dial gauges were installed at the points G1 to G5 and the dial gauge no.1 (G1) is located at the middle of the slab as shown in Fig. 5.

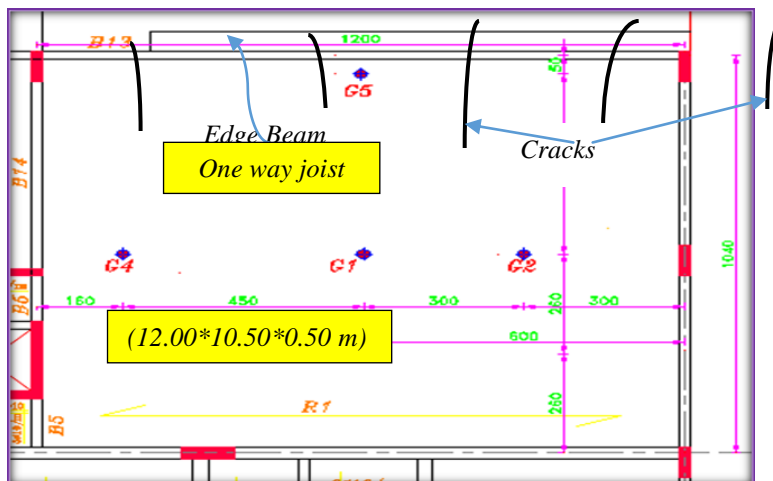


Figure 5. Location of Dial Gauges and crack of F.F.S

### 4.3.3. Simulation of Distributed Loading

The design loads were simulated by means of concrete blocks (30\*20\*15 cm) (length x width x height) and the sample weight equal to 15 kg for each block was relatively easy to install and control for the structure under investigation. For this purpose, this load is exactly similar to the finish and live load because it is uniformly distributed on the whole area of the slab equality.

The concrete blocks are distributed exactly like real load, the effects of the design load are similar to the load test. This gives the very good response of the slab system and allows using lighter equipment with low cost. The load test of the slabs was done in phases.

### 4.4. Testing Procedure

The next section shows the conceptual steps followed in order to:

- determine the value of the total test load magnitude during a preparatory phase;
- obtain the continuous structural assessment during the load test performance; and,
- obtain the real-time calibration of the test load according to the continuous assessment of the boundary conditions through the measurement of selected structural parameters.

#### 4.4.1. Load Test Protocols

From the American Concrete Institute (ACI) standard, two variables are considered for the principle evaluation and they are:

- 1) Dead load effect such as weight of slab and finish load;
- 2) Live load effect.

By this way, the total load (weight) that is applied on the tested deck slab can be calculated as suggested by ACI- 318/318R [11, 12].

#### 4.4.2. Load Calculation

Following the Building Code Requirements for Structural Concrete ACI 318/318 [12]

The Value test load shall be calculated.

$$\text{Total Load} = 0.85*(1.4*\text{Dead load} + 1.7*\text{Live load})$$

*Test load*

$$W = 0.85*(1.4DL+1.7LL) \quad (1)$$

*D.L i. e. Dead load Contain 1. Finishes. + 2. Services and ceiling*

*L.L i.e. Live Load expected.*

D. L. Calculation, Finishes Floor Layers (tile 2.5cm + mortar 2.5cm + fill 12cm)

*Tile 2.5cm U.W.=2.2T/m<sup>3</sup>) Mortar 2.5cm (U.W. =2.2 T/m Fill 12cm Th. (U.W.=1.7T/m<sup>3</sup>)*

**\*For Two Slabs First and Second Floor Roof:**

*1- Finishes Floor (tile 2.5cm + mortar 2.5cm + fill 12cm)*

$$= 0.025*2200+0.025*2200+.10*1700 = 0.280 \text{ Ton/m}^2$$

*2- Services and ceiling = 0.050 + 0.50 = 0.100 Ton/m<sup>2</sup>*

$$\text{Total dead load} = 0.280 + 0.100 = \mathbf{0.380 \text{ Ton/m}^2}$$

$$\text{Live Load} = 0.2 \times 1 = \mathbf{0.200 \text{ Ton/m}^2}$$

$$\text{Total Test load } W = 0.85*(1.4*\mathbf{0.380} + 1.7*\mathbf{0.200}) = \mathbf{0.740 \text{ Ton/m}^2}$$

#### 4.4.3. Load Configuration

The load was applied to the slab uniformly distributed on the whole area. The intensity of the applied load as it was determined in three layers of blocks, and the total load of these three layers equals the exactly calculated test load so the same effect in terms of negative moment resulting from the factored, uniformly-distributed load

One square meter contains 16.5 block \*15 kg = 247 kg/m<sup>2</sup>/layer.

Three layers 3\*247 kg/m<sup>2</sup> = 741 kg/m<sup>2</sup> # 0.740 Ton/m<sup>2</sup>.

The ACI requirements and standards for the structural using condition must be considered and limited by two variables that are [10, 11, 12]:

- 1) Maximum Deflection and
- 2) Rebound Deflection or Residual Deflection.

According to ACI 318/318R, the maximum deflection and the rebound deflection are

$$\Delta_{\max} \leq L^2 / 20000h \quad (2)$$

$$\Delta_{\text{rebound}} \leq \Delta_{\max} / 4, \quad (3)$$

where:  $\Delta_{\max}$  is the Maximum Deflection;

$\Delta_{\text{rebound}}$  is the Rebound Deflection or Residual Deflection;

$L$  is length of slab on the short side, and  $h$  is thickness of slab.

#### 4.4.4. Load Testing Procedure

Procedure for load testing

1. Install the dial gauges no.1 to 5 ( $G1 - G5$ ) onto the slab structure for five points that are located as shown in Fig. 5 and the dial gage no.3 ( $G1$ ) is installed at the middle of the slab. The dial gage installation uses the magnetic base (Fig. 6).
2. Record all initial deflections and the temperature prior to the testing.
3. Increase the load (Concrete Block Weight) step by step from 0%, 25%, 50%, 75%, and 100% of the maximum test load and each load step is held for 1 hour (for this deck slab structure, the design maximum live load equals 200 kg/m<sup>2</sup>).
4. Except the maximum test load (100%) that has to maintain 24 hours.
5. After 24 hours, the test load is decreased step by step from 0%, 50%, 75% and 100% of the maximum test and each released load step is held for 1 hour.
6. After releasing all test loads, it is maintained for 24 hours.

## 5. First Floor Roof Slab



Figure 6. Dial Gauges Installation in F.F.S and Magnetic Holder

### 5.1. Calculation of Maximum Allowable Deflection:

Slab Dimension = 12.0 m x 10.5 m x 0.5 m (thickness)

Criteria I: Max Allowable Deflection: According to ACI 318

Note: Slab Span = 10.50 m, Thickness = 0.50 m

$$\Delta_{max} = L^2 / (20000 * h) = (10500)^2 / (2000 * 500) = 11.025 \text{ mm.} \quad (2)$$

Maximum Measured Deflection  $\Delta_{meas.} = 20.66 \text{ mm.}$

$\Delta_{meas.} = 20.66 \text{ mm} > \Delta_{max} = 11.025 \text{ mm.}$

Criteria II: Rebound Recovery Allowable Deflection:  $\Delta_r \text{ max} \leq \Delta \text{ max}/4$  Acc. to ACI 318.

### 5.2. Testing Results

The results from the load test are shown by tables and charts as in Table 1 respectively.

Table 1. Dial Gauges Readings

	<i>Dial Gauge Reading (mm)</i>	<i>G1</i>	<i>G2</i>	<i>G3</i>	<i>G4</i>	<i>G5</i>
1	Load 0%	4.5	1.16	5.84	2.59	6.07
2	Load 25% (Stage 1)	7.5	3.32	10.5	6.3	8.24
3	Load 50% (Stage 2)	13.85	5.31	17.5	12.9	15.36
4	Load 75% (Stage 3)	24.22	9.3	26.5	21.5	24.48
5	<b>Max. Measurement dif.</b>	<b>19.72</b>	<b>8.14</b>	<b>20.66</b>	<b>18.91</b>	<b>18.41</b>

*During the 3rd stage of loading the excessive deflection was measured, cracks appeared in the slab and edge beam, it became wider for that we stopped the increase of the load (Fig. 7)*

	<i>Dial Gauge Reading (mm)</i>	<i>Gr1</i>	<i>Gr2</i>	<i>Gr3</i>	<i>Gr4</i>	<i>Gr5</i>
1	Released Load 0%	24.22	9.3	26.5	22	24.48
2	Released Load 25%	17.37	6.61	19.75	14	19.11
3	Released Load 50%	13.74	5	15.24	10	15.89
4	Released Load 75%	9.7	2.85	10.35	6	11.6
Net Deflection after Released Load		<b>14.52</b>	<b>6.45</b>	<b>16.15</b>	<b>16</b>	<b>12.88</b>
Dial Gauge Recovery Reading %		<b>74%</b>	<b>79%</b>	<b>78%</b>	<b>82%</b>	<b>70%</b>

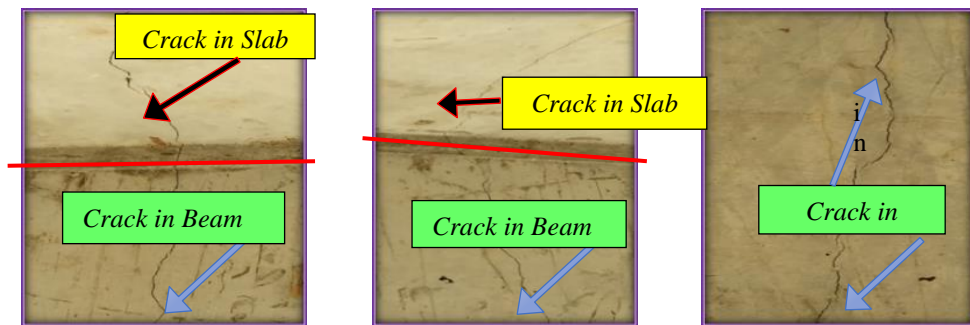
**Table 2. Testing Deflection ( $\Delta_{max}$  and  $\Delta_{rebound}$ ) and Allowable Rebound Deflections**

<i>Dial Gauge No.</i>	$\Delta_{max}$	$\Delta_{rmax}$	$\Delta_{max}/4$	<i>Result</i>
<b>G1</b>	19.72	5.20	4.93	<b>Unverified</b>
<b>G2</b>	8.14	1.69	2.04	<b>Verified</b>
<b>G3</b>	20.66	4.51	5.165	<b>Verified</b>
<b>G4</b>	18.91	3.38	4.73	<b>Verified</b>
<b>G5</b>	18.41	5.53	4.60	<b>Unverified</b>

Referring to tables 1 and 2, it is clear that gauges no.**G1** and **G5** are unverified

**Note:** 1. All maximum deflections ( $\Delta_{max}$ ) from the testing must be less than the calculated deflection that equals to **11.025 mm**.

2. The rebound deflections must be less than the calculated rebound deflections that are shown in the last column of Table 2.



**Figure 7. Cracks at the edge beam and F.F.S**

### 5.3. Analysis of Load Test Results

The graph in Fig. 8 shows the relationships between the maximum deflection of the slab deflection for the dial gauge No.1 (G1) is 19.72 mm and the rebound deflection is 5.20 mm. For the dial gauge No.5 (G5), the maximum slab deflection is 18.41 mm and the rebound deflection is 5.35 mm. From the load test results, all maximum deflections ( $\Delta_{max}$ ) from the testing must be less than the calculated deflection that is 11.025 mm (calculated from Equation (2)) and the rebound deflections must be less than the calculated rebound deflections that are shown in the last column of Table 2 as well.

This slab has been suggested for demolition because the maximum deflection is more than the allowable as per equation (2) (11.025 mm) and also before applying all of the load as well as many cracks have appeared.

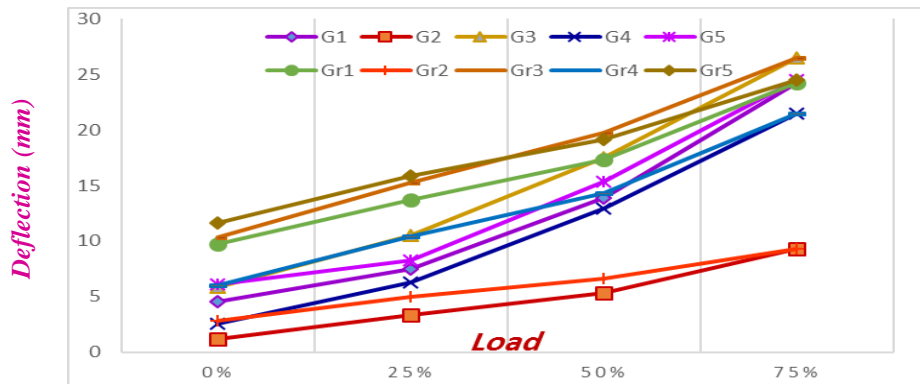


Figure 8. Relationships between Defl.(G), Rebound Defl.(Gr) and Max. Load Percent G1-G5  
*“Structural member tested (First Floor Roof Slab) does not satisfy Criteria I or Criteria II”*

## 6. Second Floor Roof Slab:

Fig. 9 is showing the location of the Dial Gauges on the plan of the second floor slab.

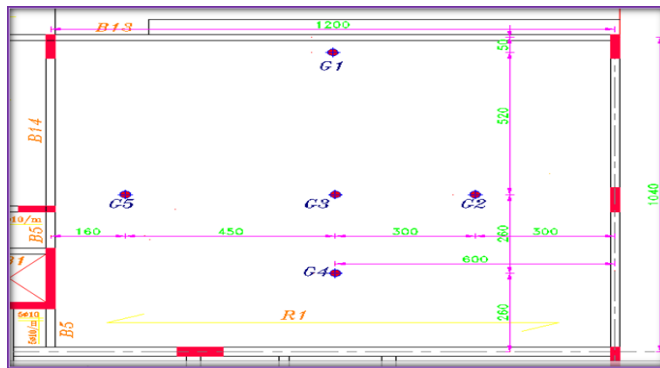


Figure 9. Location of the Dial Gauges on the plan of SFR

### 6.1. Load Calculation

In this case the same procedures: test load in the first floor roof slab (4.3), (4.4):

$$\text{Total Test load } W = 0.740 \text{ Ton/m}^2$$

$$\text{Three layers } 3 \times 247 \text{ kg/m}^2 = 741 \text{ kg/m}^2 \# 0.740 \text{ Ton/m}^2$$

### 6.2. Calculation of Maximum Allowable Deflection

$$\text{Same of (5.1): } \Delta_{max} = L^2 / (20000 \cdot h) = (10500)^2 / (2000 \cdot 500) = \underline{11.025 \text{ mm}}$$

$$\text{Maximum Measured Deflection } \Delta_{meas.} = 20.66 \text{ mm.}$$

$$\Delta_{meas.} = 20.66 \text{ mm} > \Delta_{max} = 11.025 \text{ mm}$$

### 6.3. Testing Results

**A) Maximum Allowable Deflection:** The results from the testing (both the maximum and rebound deflections) must be compared with the allowable maximum and rebound deflections that are calculate from Equation (2) and (3), respectively, as shown in Table 3.

**Table 3. Dial Gauges Readings**

<i>Dial Gauge Reading (mm)</i>		<i>G1</i>	<i>G2</i>	<i>G3</i>	<i>G4</i>	<i>G5</i>
1	LLoad 0%	4.25	7.92	8.43	8.02	0.44
2	Load 25% (Stage 1)	10.82	17.61	18.4	11.45	3.02
3	Load 50% (Stage 2)	17.3	25.4	25.94	15.04	5.78
4	Load 75% (Stage 3)	20.81	29.71	29.91	17.86	6.26
5	Load 100% (Stage 4)	20.81	30.00	30.55	18.31	6.53
6	Max. Measurement difl.	<b>16.56</b>	<b>22.08</b>	<b>22.12</b>	<b>10.29</b>	<b>6.09</b>
	<b>Result</b>	<b>Unverified</b>	<b>Unverified</b>	<b>Unverified</b>	<b>Verified</b>	<b>Verified</b>

**Table 4. Testing Deflection ( $\Delta_{max}$  and  $\Delta_{rebound}$ ) and Allowable Rebound Defl.**

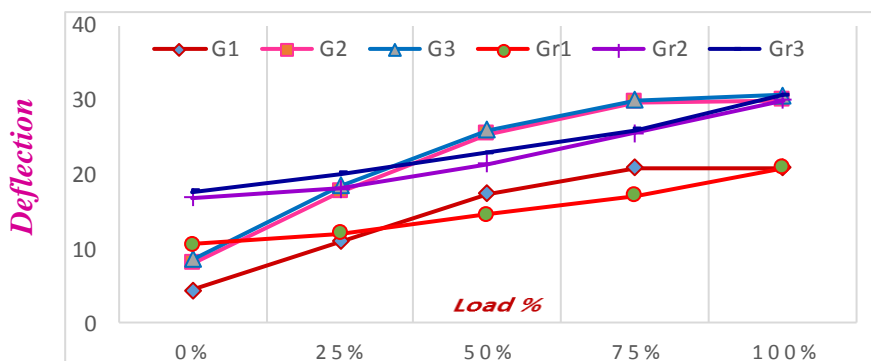
<i>Dial Gauge Reading (mm)</i>		<i>Gr1</i>	<i>Gr2</i>	<i>Gr3</i>	<i>Gr4</i>	<i>Gr5</i>
1	Load 100% held for 24 hours	20.81	30	30.55	<i>No Need</i>	<i>No Need</i>
2	Released Load 25%	17	25.5	25.9		
3	Released Load 50%	14.5	21.4	22.8		
4	Released Load 75%	12	18	20		
5	Released Load 100% held for 24h	10.44	16.75	17.62		
Net Deflection after Released Load		<b>10.36</b>	<b>13.25</b>	<b>12.93</b>		
Dial Gauge Recovery Reading%		<b>63%</b>	<b>60%</b>	<b>58%</b>		

**B) Rebound Recovery Allowable Deflection:**

Referring to Tables 4 and 5 it is clear that gauges No. *G1 – G3* are unverified.

**Table 5. Testing Deflection ( $\Delta_{max}$  and  $\Delta_{rebound}$ ) and Allowable Rebound Deflections ACI**

<i>Dial Gauge No.</i>	$\Delta_{max}$	$\Delta_{rmax}$	$\Delta_{max}/4$	<i>Result</i>
<b>G1</b>	16.56	6.19	4.14	<b>Unverified</b>
<b>G2</b>	22.08	8.83	5.52	<b>Unverified</b>
<b>G3</b>	22.12	9.19	5.53	<b>Unverified</b>
<b>G4</b>	<i>No need because it is verified from the first criteria (A)</i>			
<b>G5</b>	<i>No need because it is verified from the first criteria(A)</i>			



**Figure 10. Relationship between Defl. (G), Rebound Defl. (Gr) and Max. Load Percentage**

#### 6.4. Analysis of Load Test Results

The graph in Fig. 10 shows the relationships between the maximum deflection of the slab, the dial gauges No.1 and 3 (G1, G3), which is 16.56 – 22 mm, and the rebound deflection, which is 6.19 – 9.19 mm. But there is no need for G4 and G5 to check the rebound deflection because it is verified from the first criteria.

From the load test results, all maximum deflections ( $\Delta_{max}$ ) from the testing must be less than the calculated deflection that is 11.025 mm (calculated from Equation (2)) and the rebound deflections must be less than the calculated rebound deflections that are shown in the last column of Table 4 as well. This slab has been suggested for demolition because the maximum deflections are more than allowable as per equation 2 (11.025 mm) and the rebound deflections still exist in the slab structure as shown in Table 4.

### 7. Cutting, Demolition and Removal of Slabs

After the experimental results that appeared in the load test the slabs were unsafe, according to the first and second procedures of the ACI-318, so the Engineering Consulting Office decided to demolish and remove both slabs, roof I and roof II.

The demolition and removal work was carried out. The slabs and adjacent concrete slabs were supported to ensure that the adjacent construction elements were not affected by the cracking and demolition process.

A plan has been made to cut the concrete of the slab, taking into account the distance of the surrounding slab, to remove it calmly and carefully, and to maintain the existing reinforcing steel to bond with the new reinforcement steel length of 1.30 meters as shown in Fig. 11.

Upon completion of the demolition and removal, important things were found out:

- \* The total thickness of the executed slab ranges from 43 to 45 cm.
- \* The thickness of the slab is variable of the slab and varies from one place to other.
- \* The thickness of the concrete slab above the ribs ranges from 8 to 12 cm.
- \* The reinforcement is complying with the design which is 6 # 16 per rib.
- \* Concrete is weak in some places and positions of the slab.
- \* Distribution of ribs, molds, and interstitial spaces between ribs is irregular and unequal.



Figure 11. Reinforced concrete slabs after cutting

## 8. Design Checking

Reference to the ACI 318-05 codes for the design of the one-way joist slab system of the rib in one direction shows that the thickness limits for this type of slabs is not less than  $L / 16$  i.e. for the case study the required thickness is equal to  $1200/16 = 75$  cm at least and thus the design is not identical to the ACI 318-05 codes (Tab. 6).



Figure 12.  
Measuring of thickness of  
the rib (R1) slab

ACI 318-08 table 9.5(a): minimum thickness( $h_{min}$ )

member	simple	One end continuous	cantilever	Two end continuous
One way ribbed slab and beam	$L/16$	$L/18.5$	$L/8$	$L/21$
one way solid slab	$L/20$	$L/24$	$L/10$	$L/28$

Table 6. Min. thickness of the rib (R1)

## 9. Verification of implementation

The measurement of thickness of concrete slabs after cutting showed it is 43 cm (Fig. 12) less than the thickness required as according to ACI 318, which equals  $L/16 = 75$  cm (Tab. 6).

Moreover, it does not match the thickness of the original design which is done by consulting the engineering office and is equal to 50 cm. Also, the thickness of the concrete slab above the ribs does not correspond to the thickness of the design, which is 15 cm as shown in Fig. 2, 12. Furthermore, poor implementation can be seen in the removal of the scaffold before the specified time or not tying the supporting stent to it. In addition, the summer temperature increase generates cracks in the upper slab resulting from shrinking and creep [7].

## 10. Study and Analysis

From the mentioned above, it is found out:

- 1) Weakness of structural design carried out by the Engineering Consulting Office in terms of the dimensions of the rib section that does not comply with the ACI-318 requirements.
- 2) Defect in the implementation as the rib section of the performance and its dimensions does not match the rib design section.
- 3) Bad distribution of ribs and empty molds as well as unequal distance between them.
- 4) Change in the thickness of the slab from one place to another.
- 5) Contractor's Failure to apply the technical conditions and specifications for the execution of the concrete works, e.g. there are gaps and voids in the concrete.
- 6) The strength of the reinforcement bars is not tested.
- 7) The concrete was not tested and may be weak because the results showed weakness in some places during the cutting and removal.

## 11. Second Case: Villa in Suburb Bayan in 2007

### 11.1. General Description of the Building

This study is carried out to assess the integrity of the structural system of the completed building in the Bayan area. The information and data available are architectural and structural drawings, as well as the results of concrete core tests. The building consists of a basement, a ground floor, the first and the second floors. The building was built of reinforced concrete, and the walls were built of bricks (Fig. 13, 14).

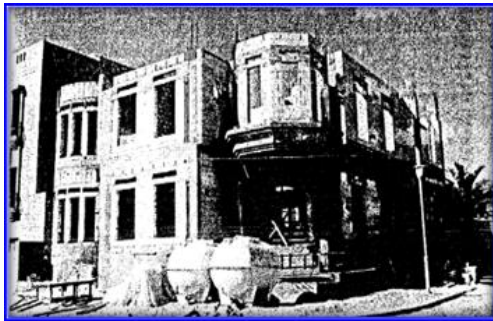


Figure 13. General view of the building

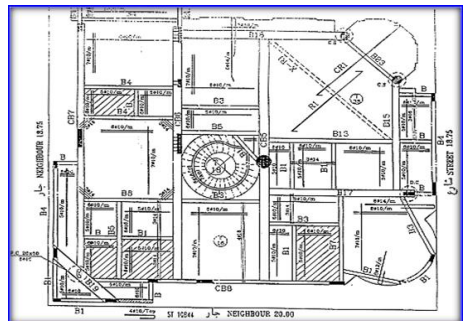


Figure 14. Ground floor roof slab

### 11.2. Optical inspection results

The cracks were distributed at the positive and negative moment regions. The deflection was in the center of the slabs. The testing (core test) was of slabs and beam to assess the strength of the existing concrete and look for reasons if it fails. A detailed optical inspection was carried out to identify the apparent structural defects and to compare the dimensions of the structural elements executed on the site as well as what is required by the structural design drawings for some elements of the building:

- 1) There were inclined cracks in some of the beams, B17, CB1 and CB5 in the ground roof.
- 2) There were cracks at the bottom at some of the slabs in the ground roof (Fig. 15, 16).
- 3) There were changes in the color of the concrete slab roof in different places.
- 4) There are vertical cracks in the B3 beam (Fig. 15, 16).
- 5) Concrete core samples were taken from different parts of the building. The results show that the compressive strength of the concrete of the beam CB5 is  $130 \text{ kg/cm}^2$  and  $140 \text{ kg/cm}^2$  for the B18 and for the ground floor roof is  $168 \text{ kg/cm}^2$ . The results indicate a weak concrete.

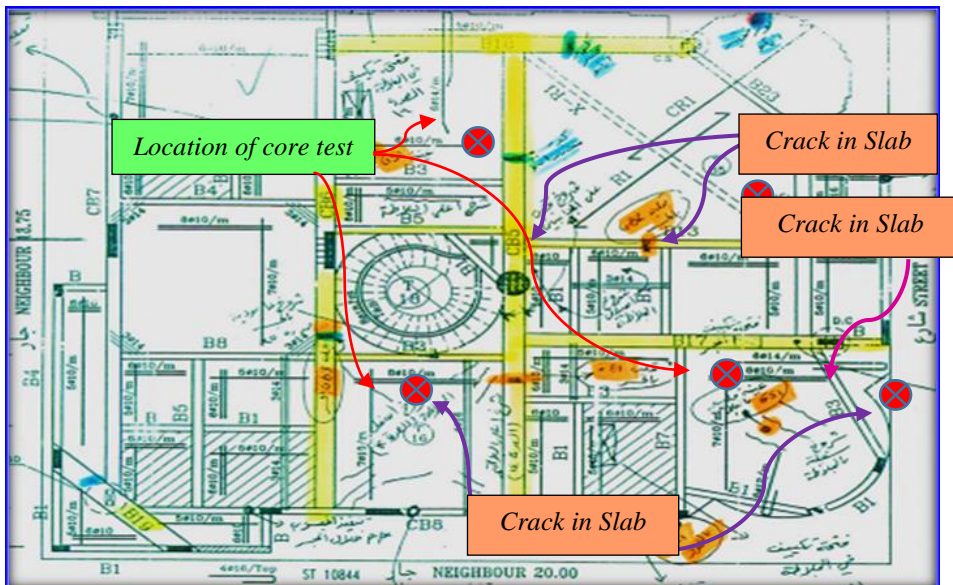


Figure 15. Ground floor roof slab (location of cracks and core test sample)

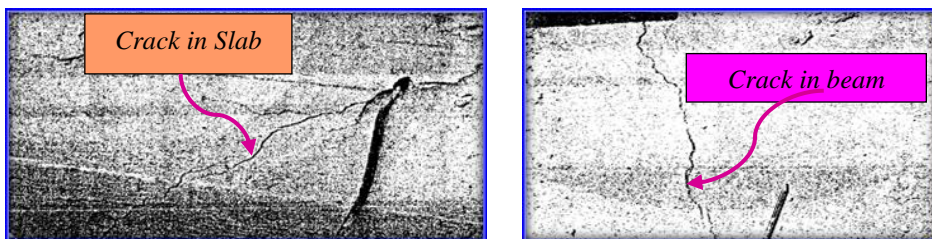


Figure 16. Cracks at the beam and slab

### 11.3. Concrete tests

Concrete core samples were taken from various places in the building (five samples: three from the beams and two from the ground roof slab) to determine the ability of the concrete to bear the compressive strength of the concrete and its quality, as well as to measure

the actual strength of the concrete. They were examined 24 hours after the extraction. The following table shows the detailed results of the concrete samples (Fig. 17, Tab. 7).

#### 11.4. Concrete strength: concrete core tests

The specifications of the building stipulate that the compressive strength of the concrete for the slabs and beams should be not less than 250 kg/cm<sup>2</sup> and for the columns 300 kg/cm<sup>2</sup>.

The results were 208, 130, 205 kg/cm<sup>2</sup> in the beams and 168, 170 kg/cm<sup>2</sup> in the slab.

The knowledge that the two samples CB3 and GS1 contain bars of reinforcement, which significantly affect the impact on the result, can be taken into consideration when using the results of the tests and the structural analysis. The strength decrease of the GB2 beam reaches 130 kg/cm<sup>2</sup>, about 52% of the required strength (Tab. 7, 8). The results show that the average strength of the beam and slabs samples is 188 kg/cm<sup>2</sup>, which is equivalent to 75% of the imposed design strength of 250 kg/cm<sup>2</sup>. It is also noted that three samples are less than 75% of the design strength imposed (Tab. 7, 8).

**Table 7. Results of the core test**

<i>No</i>	<i>Specimen</i>	<i>GB1</i>	<i>GB2</i>	<i>GB3</i>	<i>GS-1</i>	<i>GS-2</i>
1	- Identification of the specimen:	<i>Beam</i>	<i>Beam</i>	<i>Beam</i>	<i>Slab</i>	<i>Slab</i>
2	- Location: Axes:	E/6	G/7	E/5	CIS	H/S-6
3	- Average diameter (mm):	99.3	99.3	99.3	99.3	99.3
4	- Core Weight (gm) <sup>1</sup>	2380	2312	2435	1693	1707
5	- Density of specimen (kg/m <sup>2</sup> ):	2190	2144	2240	2197	2157
6	- Maximum load of failure (KN):	133.3	82.1	130.0	114.7	115.8
7	- Cross-sectional area (mm):	7744	7744	7744	7744	7744
8	- Measured compressive strength (N/mm <sup>2</sup> ):	17.2	10.6	16.8	114.8	115.8
9	- Correction factor:	0.95	0.96	0.96	0.89	0.89
10	Compressive strength (N/mm <sup>2</sup> ):	16.4	10.2	16.1	13.2	13.4
11	- Estimated insitu cube strength (N/mm <sup>2</sup> ):	20.4	12.7	20.1	16.5	16.7
12	<b>- Estimated in-situ cube strength (kg/cm<sup>2</sup>):</b>	<b>208</b>	<b>130</b>	<b>205</b>	<b>168</b>	<b>170</b>

According to para. 5.6.5.4 of ACI-318 the specifications are as follows: concrete is considered constructively valid if the core test samples are provided with the following two conditions:

1) The strength average of three samples is not less than 85% of the design strength of the concrete;

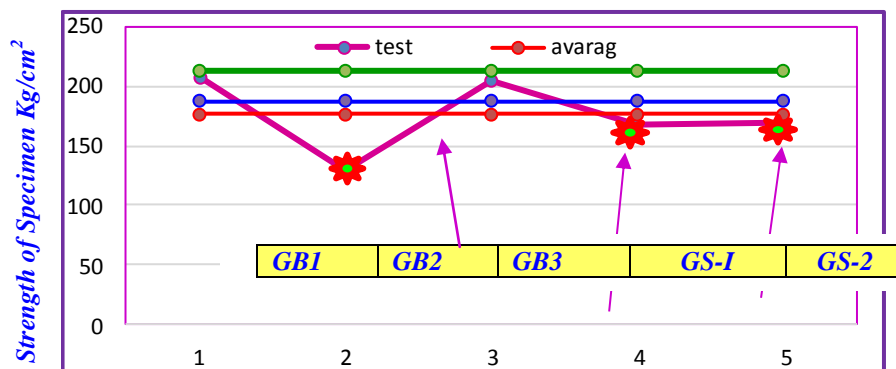
2) There are no samples whose strength is less than 75% of the design strength of the concrete.

The results of the tests indicate that these conditions are not met in the ground roof slab.

The cause of the concrete weakness was due to the design of the concrete mixture also by the company; supplying the concrete by increasing the water or due to bad curing of the concrete by the contractor company. It is necessary to verify the slabs in other ways, such as a load test (Fig. 17).

**Table 8. Analysis of core test results**

No	Strength of core test (kg/cm <sup>2</sup> )	GB1	GB2	GB3	GS1	GS2
1	Strength design value	250				
2	Min. Computing = $0.85 * F_c$	212.5				
3	Average Strength	181.83				
4	Strength Not Less Than $0.75 * F_c$	187.5				
5	Test Reading (insitu)	208	130	205	168	170
6	<b>Result</b>	Verified	Unverified	Verified	Unverified	Unverified



**Figure 17. Strength of core test, Average, Min., Not less**

### 11.5. Study and Analysis

It is clear from the study that the thickness of the slabs is sufficient as well as the reinforcing steel of the slabs in both directions and the absence of an upper reinforcement steel, resulting in cracks around the slabs and the sides of the beams as there is no reinforcing in the corners of the slabs causing additional cracks. The reduction of the cross-sectional reinforcing steel in the B11 and the CB5 beam was also shown to be unsafe due to the design and the weakness of the concrete, resulting in clear cracks. It is noted from the design that the slabs are designed as simple support and should be designed as continuous slabs with the knowledge that the reinforcing steel is properly positioned and properly designed.

It was observed in the beams that ACI-318 code was not followed when the upper reinforcing steel was not extended to 22% of the length of the larger beam in the adjacent beam.

### 11.6. Conclusions and Recommendations

The study shows that the structural elements in the building need supporting and strengthening to enable the building to carry out the structural role required to ensure the structural safety required for the building. The repair process should be designed for defective structural elements by a consulting engineering firm specializing in repairs.

Beams and slabs in each landing should be repaired by a specialized company and engineering supervision.

## 12. Third Case: Villa in Suburb Salmiya in 1997

### 12.1. General description of the building

The study was conducted for an existing building (villa) in the Salmiya area which was built in 1997. The information and data available are architectural and structural drawings, as well as the results of concrete core tests and load test. The building consists of a basement, a ground floor, a first and second floor, and a roof with two rooms. The building was completed with the construction of the structure (Fig. 18). From the observation and visual inspection of the building, there are many damages and defects in the structural elements:

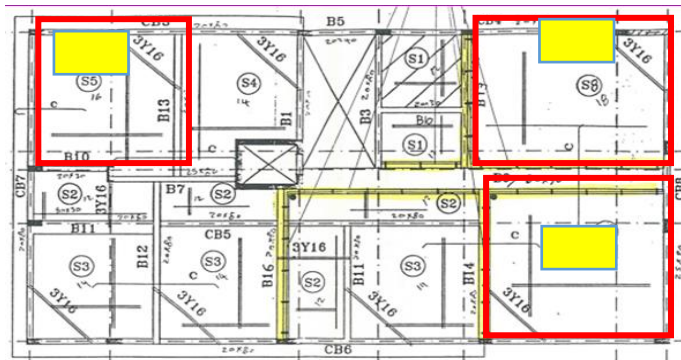


Figure 18. Ground Floor Roof Slabs plan

### 12.2. Optical inspection results

#### Beams:

- 1 – The air conditioning openings are near to the beams support points (at the high shear points), causing fractures and cracks in these places (Fig. 19).
- 2 – The concrete cover of the steel reinforcing of the beams is insufficient as the stirrups steel reinforcing have appeared from the bottom of the beam.
- 3 – There are cracks around most of the air conditioning openings in the beams.
- 4 – Nesting and voids (honeycomb) in concrete.
- 5 – There are inclined cracks in some beams and lack of straightness (Fig. 19).
- 6 – Adhesion of the bars of steel reinforcing in the some beams, which prevents the passage of concrete, causing a lack of cohesion between concrete and steel.

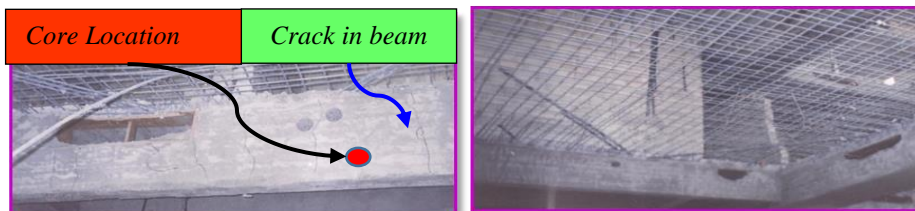


Figure 19. Cracks in these beams

### **Slabs:**

1 – The presence of water in many areas in most of the slabs, indicating the presence of cracks and deep fractures on the whole thickness of the slab.

2 – There are cracks and cracks up in some places to display 2 mm.

3 – There is a large deflection in the floors of the first floor (roof of the ground floor) up to 5 cm in slabs S5, S8 and S9 in addition to the presence of deep and numerous cracks on the perimeter of the slab, which sometimes reach the full thickness of the slab.

Thirteen concrete core test samples were taken from all parts of the building to verify the quality and safety of the concrete in all floors, taking into consideration that they are distributed so that each floor and all structural elements are represented and the following Tab. 9 shows this:

**Table 9: Results of the core test**

<i>No.</i>	<i>Place of Sample</i>	<i>Type &amp; Axis</i>	<i>Number</i>	<i>Cubic Strength (kg/cm<sup>2</sup>)</i>
1	Basement slab roof	Beam-C	B15	<b>153</b>
2	Basement slab roof	Beam-D	B15	<u>128</u>
3	Basement slab roof	Beam-3-4	B16	<b>161</b>
4	Basement slab roof	Slab-E-F,6-7	S4	<u>140</u>
5	Ground slab roof	Beam-D	B9	<b>162</b>
6	Ground slab roof	Beam-6	B14	<b>179</b>
7	Ground slab roof	Slab-B-C,5-6	S3	<b>187</b>
8	Ground slab roof	Slab-B-D,6-8	S9	<b>175</b>
9	Ground slab roof	Slab-D-F,2-3	S5	<b>172</b>
10	First slab roof	Beam-D	B9	<b>233</b>
11	First slab roof	Beam-8	CB8	<b>175</b>
12	First slab roof	Slab-D-F,6-8	S8	<u>142</u>
13	First slab roof	Slab-D-F,2-3	S5	<u>113</u>

### **12.3. Concrete strength: concrete core tests**

As mentioned above in the previous case and para. 5.6.5.4 of ACI-318 specifications conditions of core test concrete, from the results of the tests we conclude the following:

1. The average concrete strength of the basement roof slab is 67.2% of the design value and the least value is equivalent to 59% of the design value, so the concrete is considered failure.

2. The average concrete strength of the ground roof slab is less than 82% of the design value, so the concrete is considered failure.

3. The average concrete strength for the first roof slab is equivalent to 76.4% of the design value and the smaller value is 52% of the design value, so the concrete is considered failure.

The thickness of the roof slabs was measured in the concrete core test samples and compared with the design thickness according to the drawings and it was found less than 10%.

## 12.4. Study and Analysis

A structural analysis of the core test results and comparison with the structural design provided by the Engineering Consulting Office shows:

1 – Beams: The reinforcement ratio of some beams is greater than the permissible.

Reinforcement ratio according to ACI-318 makes the method of failure surprising (Brittle mode of failure) in the case of increasing the stresses of the structural elements (Tab. 10).

2 – Slabs: In the ground and first floor the slabs S5, S8 and S9 are structurally unsafe (Tab. 11).

**Table 10: Analysis of the core test results**

<i>Strength of core test (kg/cm<sup>2</sup>)</i>		<i>B15-C</i>	<i>B15-D</i>	<i>B16</i>	<i>S5</i>	<i>S9</i>
1	Min. Computing = $0.85 * F_c$	<b>0.85*250 = 212.5</b>				
2	Average Strength	<b>(153+128+161) / 3=147.33 &lt; 212.5</b>			<b>(187+175+172)/3=178</b>	
3	Strength Not Less Than $0.75 * F_c$	<b>0.75*250 = 187.5</b>				
4	Test Reading (in-situ)	<b>153</b>	<b>128</b>	<b>161</b>	<b>172</b>	<b>175</b>
5	<b>Result</b>	<b>Unverified</b>	<b>Unverified</b>	<b>Unverified</b>	<b>Unverified</b>	<b>Unverified</b>

## 12.5. Load test

The loading test for the roof slabs of the ground floor was carried out by the Government Inspection Center of the Ministry of Public Works – Kuwait, 20/10/1997. The results are as follows:

**Table 11. Analysis of the load test results**

<i>No</i>	<i>Statement</i>	<i>Slab1</i>	<i>Slab2</i>	<i>Slab3</i>
1	Identification of the specimen & Location	S5	S8	S9
2	Dimensions (m):	6.05*5.8	5.25*4.6	4.50*4.0
3	Thickness (cm):	18	14	14
4	Dead Load: 1) Own Weight (Kg/m <sup>2</sup> )	450	350	350
5	2) Covering Load (Finishing) (Kg/m <sup>2</sup> )	200		
6	Total Dead Load (TDL) (Kg/m <sup>2</sup> )	650	550	550
7	Live Load (Kg/m <sup>2</sup> )	250		
8	Total Load (Kg/m <sup>2</sup> ) = $0.85 * (1.4D.L + 1.7L.L)$	1134.75	1015.75	1015.75
9	Test Load = Total Load – TDL (Kg/m <sup>2</sup> )	484.75	465.75	465.75
10	<b>Max. Allowable Deflection = <math>L^2/20000 * T</math> (mm)</b>	<b>9.34</b>	<b>5.7</b>	<b>7.56</b>
11	<b>Max. Measured Deflection (mm)</b>	<b>8.27</b>	<b>5.05</b>	<b>6.95</b>
12	<b>Remarks: This slab as required by ACI-318</b>	<b>Satisfy</b>	<b>Satisfy</b>	<b>Satisfy</b>

Cracks have been developed in one side at the test and small cracks also at opposite side.

It was noted that the value of the deflections that occurred during the tests are acceptable because the ratio of the existing reinforcing steel is greater than the maximum percentage according to the ACI-318, which means that the concrete strength will reach its maximum value before the steel reaches the yield point.

## **12.6. Conclusions and Recommendations and Identification of Damage Causes**

With reference to the damage and its shape after the study of the design of the drawing and the work of the core tests and load tests for the concrete, we can summarize the causes of damage as follows:

- 1 – Concrete is non-conforming to the required specifications in the roof slabs.
- 2 – Poor implementation of the roof slabs where the thickness of many slabs does not match the drawings, causing the occurrence of distortions and large cracks in many of the slabs.
- 3 – Poor implementation in the installation of steel bars where the required distances between the bars were not considered, which led to the occurrence of voids in the concrete, which leads to a weak link between the concrete and steel.
- 4 – The lack of supervision of the required level and lack of care and quality control of implementation, especially in the observation of damage in the roofs and beams, as well as poor implementation of pouring concrete add to the installation of air-conditioning openings in the high cut points of beams.

## **13. Methods of treatment and repair**

- 1 – Due to the weakness of the concrete and the numerous cracks in the slabs of the ground and first roof, it is broken and re-poured after proper reinforcement work and the continuity of adjacent slabs.
- 2 – Strengthening of some of the beams, identified in the site by appropriate sectors of steel with the use of the necessary epoxy materials.
- 3 – Reinforcing some columns with steel sections.
- 4 – Applying epoxy layer to the old concrete faces where they are bonded with the new concrete before casting directly [9].

## **14. General Conclusions and Recommendations**

Based on the results obtained from the tests, we can summarize the results as follows:

1. The Non-Destructive Tests in-situ proved that the ACI-318 code gives logical results and expressive status and strength of the structural component.
2. The causes of weak slabs and beams resulted from weak of design and defect in implementation since they are not complying with the international codes.
3. Weak application of the requirements and specifications of the international codes appeared in the structural design of the buildings and the structural elements by the engineering consulting offices and the absence of control and audit of these designs.

4. Weak supervision and checking on the construction contracting companies to apply the technical conditions appeared in design and strict compliance with the drawings and technical specifications of the construction and buildings.
5. The repairing works of the structures are not useful because existing cracks and deflections exceeded the allowable limit during the test. In addition, the appearance of cracks and deflections in the slabs and the presence of vibrations was dangerous in some of the slabs and some of the beams.
6. We recommend that the contractors selected for a project should be classified professionally and registered in the committee of construction industry.
7. The results showed that two decades later the development of the construction technology from design or implementation was little.

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# ОЦЕНКА НА НОСИМОСПОСОБНОСТТА НА СТОМАНОБЕТОННИ ПЛОЧИ ПО БЕЗРАЗРУШИТЕЛНИ МЕТОДИ И АНАЛИЗ НА ПРИЧИНИТЕ ЗА ТЯХНОТО РАЗРУШЕНИЕ

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*Ключови думи:* стоманобетонни конструкции, полеви тестове, изпитвания на натоварването, ядрен тест, *in situ* тестове, бетонни плочи

## РЕЗЮМЕ

Основната цел на настоящата статия е да се направи оценка на безопасността и експлоатационните качества на съществуващата структурна система чрез структурна оценка на якостта чрез безразрушителни тестове за структурата на плочи от стоманобетонни конструкции и анализ на причините за отказ на покривните плочи (RC).

В тази статия са разгледани три случая на покривни плочи (двупосочна плоча, еднопосочна плоча) от три вили в Кувейт. Тези вили са построени през 2017, 2007 и 1997 г., съответно.

Наблюдава се слабост в якостта на бетона и отклонението на покривите, а в някои от плочите и някои от гредите се появяват пукнатини.

Тестовите (изпитване на натоварване, ядрен тест) бяха проведени съгласно АСІ-318, за да се знае, че съответстват на якостта на съществуващия бетон и дали е безопасно или не.

В края на статията тестовите дадоха добри резултати, за да се оцени носимоспособността на стоманобетонната конструкция. Съществуват две основни причини за дефекти на две плочи: първо – некачествено изпълнение от страна на възложителя и несъответствието с изискваните технически спецификации, а втората причина са изчислителни грешки и недобър контрол от страна на строителния надзор.

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