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Evaluation the Strength of one-way joist Concrete Slabs System by Load Test and Analysis the Causes of their Failure

Mohammad AbdulKader Al Zuhaili [1], Ali Mohammed Al Tuhoo [2]

[1] *M.Sc. Civil Engineering, Civil Engineering Department, Public Authority of Applied Education and Training, Kuwait.*

[2] *M.Sc. Civil Engineer, head of Civil Engineering Department, Public Authority of Applied Education and Training, Kuwait.*

الملخص:

تهدف الاختبارات الموقعية بشكل رئيسي الى تقييم سلامة وصلاحية النظام الانشائي القائم وديموميته (طول عمره) يصف هذا البحث الأساس المنطقي لاختبار تحميل لبلاطات خرسانية اعصاب هوردي باتجاه واحد وتطبيق النهج لتحديد مستوى التحميل وإجراءات التحميل ومتطلبات الأجهزة ومعايير التقييم والنتائج للمشاريع الميدانية. اضافة للبحث عن أسباب الفشل لهذا النوع من البلاطات.

دراسة الحالة لبلاطتي سقف الطابقين الأول والثاني من فيلا خاصة وهي بلاطات أسقف الخرسانية هوردي بأعصاب باتجاه واحد بدولة الكويت.

لوحظ ضعف في خرسانة بلاطات الأسقف، اضافة إلى هبوط واهتزاز وشقوق ظهرت في بعض البلاطات والجسور لذلك كان من الضروري التحقق من السلامة الانشائية لهذه العناصر الانشائية والبحث عن الأسباب

وكانت النتيجة بالنسبة لبلاطات السقف الأول والثاني غير آمنة والتي تتطلب هدما وتكسيرها، وإعادة التصميم والصب لهما وتدعيم بعض الأعمدة وإصلاح الجسور.

وسيتبين في نهاية البحث ان تجربة التحميل أعطت نتائج جيدة لتقييم سلامة المنشأ ، و ان اسباب فشل البلاطات يعود لسببين رئيسيين الأول سوء تنفيذ شركة المقاولات وعدم امتثالها للمواصفات الفنية المطلوبة والثاني قصور في التصميم والإشراف من قبل مكتب الاستشارات الهندسية المصمم والمشرف.

ABSTRACT

The main objective of in-situ testing is to evaluate the safety and serviceability of an existing structural system. This research describes the rationale and application of load test for *one way joist slab system.* The approaches are to determine the level of loading, loading procedures, hardware requirements, evaluation criteria and results for field projects. In addition to look for the possible causes for failure for this type of slabs. The case study was done for two concrete roof slabs on the first and second floors of a private villa is in *State of Kuwait*. There were observed weakness in the strength of concrete and deflection in the roofs and in addition, vibration and cracks appeared in some of slabs and beams. Therefore, it was necessary to check the structural safety of slabs and beams and look for causes of such failures.

The final result was for First and second Floor Roof Slabs were unsafe and the best solution to demolish the two slabs, and redesign and re-casting them and do additional supporting for some of the columns and some beams need to be repaired.

It was found at the end of the research that the loading test gave good results to assess the safety of the structure.it was found a main two reasons for failure of two slabs, were first is poor implementation from the contracting company and non-compliance with the required technical specifications, and the second reason is fault in design and weakness in supervision by the Engineering Consulting Office.

Keyword: *In situ test; Load tests; Concrete slabs; one way joist system; Concrete structures.*

1. Introduction

In-situ load testing is relevant for a variety of reasons including assessment of the effect of design and construction and deficiencies; novel strengthening and retrofit methods; capability of an existing structure to carry loads different from the original design; and, safety of structures that have experienced corrosion and degradation.

Presently, the default method for in-situ load testing of concrete structures is that prescribed in Chapter 20 of the Building Code published by the American Concrete Institute ACI committee 318-2005. This load test method and its evaluation criteria are widely referred to as the 24 h load test because the test load is held in place on the structure for a period of 24 h. we applied this method on this research.

This research describe the load test in-situ evaluation of a two roof slabs of private villa for first and second floor in order to introduce principles and outcomes of the load test method in the context of likely projects. These case studies represent an ideal test bed for the load test procedure.

The structural elements (First and second Floor Roof) were failure slabs because existing deflection equal to 6.00-7.00 cm in the mid span of slab (12.00m) happened after the removal the scaffold and before carries any load (finish and live load)

In This paper focuses on the determination of the load level and the loading procedure for each structure. Special considerations related to the design and conduct of this type of load test and core test are presented and critically discussed. The paper focuses on 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 to be found in the 1890s (Birkmire 1894).[1] The American Concrete Institute began formalizing load test procedures for concrete structures in 1920 (ACI 1920) [2], [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 Fitz Simons and Longinow 1975; RILEM 1984; Bungey 1989.[1],[4],[5],[6]

The load test method had some recent development and therefore only a limited number of reported case studies were done like (Gold and Nanni 1998; Nanni and Gold 1998a,b; Mettemeyer and Nanni 1999; Galati et al. 2004; Casadei et al. 2005) [7]. This method was attempting 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 societal needs.

In (2005)Masetti ,Casadei, et al. studied the behavior of one-way reinforced concrete slab.[1] The results were obtained from the analysis of the graph relationship between load and deflection. The maximum deflection should not be more than the allowable deflection from ACI 318 and the rebound (residual) deflection should not be less than the standard residual deflection that has followed ACI 318 as well.[8],[9],[10]

3. Objectives of the research:

The Objectives of this research were:

- To assess the structural adequacy of the building.
- To determine the level of imposed load that can be sustained;
- To do development on the in-situ evaluation methods for RC structures.
- To find the appropriate remedial work for the existing cracks and deflection.
- To collect the data that is necessary to perform the research and useful in the future

4. Description of the Building

Private villa was built in Kuwait city at 2016 (Fig. 1.) on an area of 400 square meters of land, consisting of four floors basement, ground, first and second, the building were built from Reinforced concrete.

The structural system is consisting from columns, beams and slabs. There are two kind of slabs used on the building the first one is solid slabs and the second is one way joist slab, that is it used usually for wide spaces such as halls that are not allowed columns

In This research we will focus our study on two slabs of the building that have clear cracks problem, first floors roof and second floors roof is a one way joist slab system RC concrete

Fig. 1.Elevation of Building

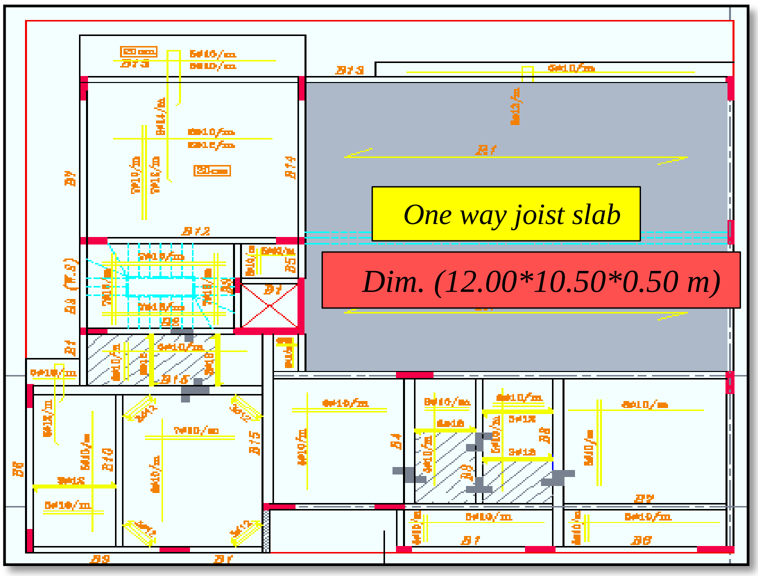

Fig. 2. Floor Roof Slabs plan

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

5.1. Structural Geometry
The structural geometry including columns locations and members sizes were determined from the engineering drawings attached, the slabs of the roof are two types, as we mentioned before. The second is one way joist slab which we focus our research it's Dimensions (12.00*10.50*0.50 m) (length x width x thickness) and section of design rib is shown in the (Fig.3)

5.2. Material Characteristics
The material characteristics were provided by consulting designing office. The specifications indicated a compressive strength of concrete of 300kg/cm2, and minimum yield strength for the mild reinforcement of the steel, is 4000kg/cm2

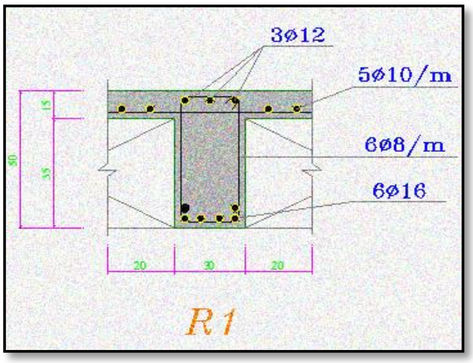

Fig. 3. Rib Section R1 (As Designed)

5.3. 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. From the visual inspection, a clear cracks were found on the first and second slab roof of the building. However, no significant crack was found on other structures members. At the roof of basement and ground did not see or notice a deflection the slab or what is alleged to worry; on the contrary in the roof slabs of the first and second floor deflection was apparent in the naked eye in the joist and edge beam and when measured was about 6-7 cm in the middle of the span, And cracks appeared in the joists and beam in addition to the vibration of the slab, therefore it was important check the safety of construction of the roof. And which required carrying loading test for both slabs. (Fig.4) On the basis of the visual inspection, the two slabs (one way joist system) with 12 m length the thickness of the slab was 50cm, was selected for a static load test.

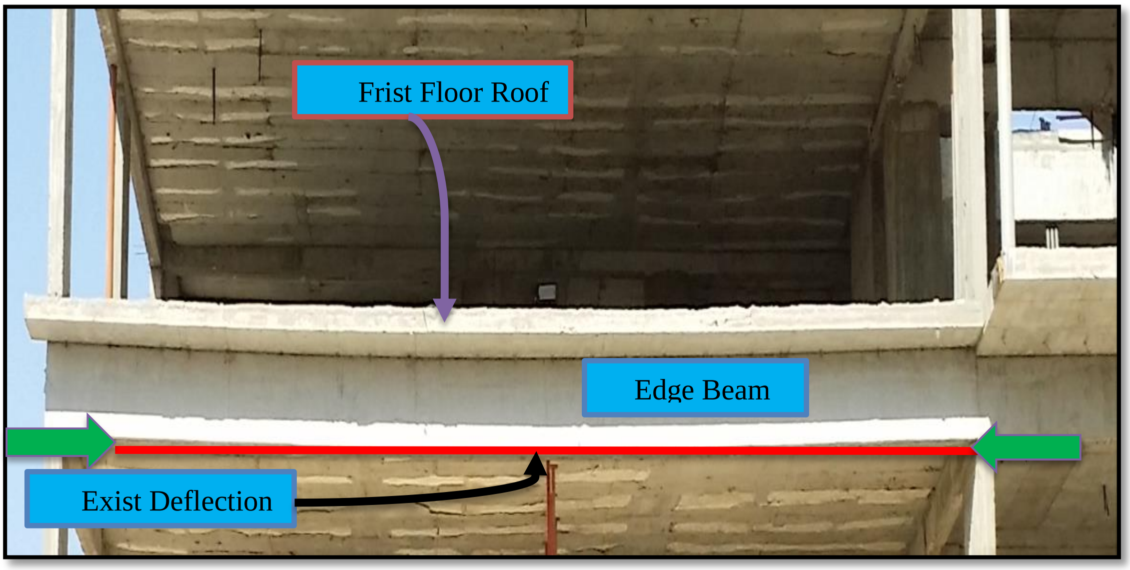

Fig. 4. Slab and edge beam with 6-7 cm deflection in the middle of the span

6. Description of the Load Test

6.1 Load Test Configurations

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

6.2 Deflection Measurement

Deflection measurement were taken in 5 different locations for installed the 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)

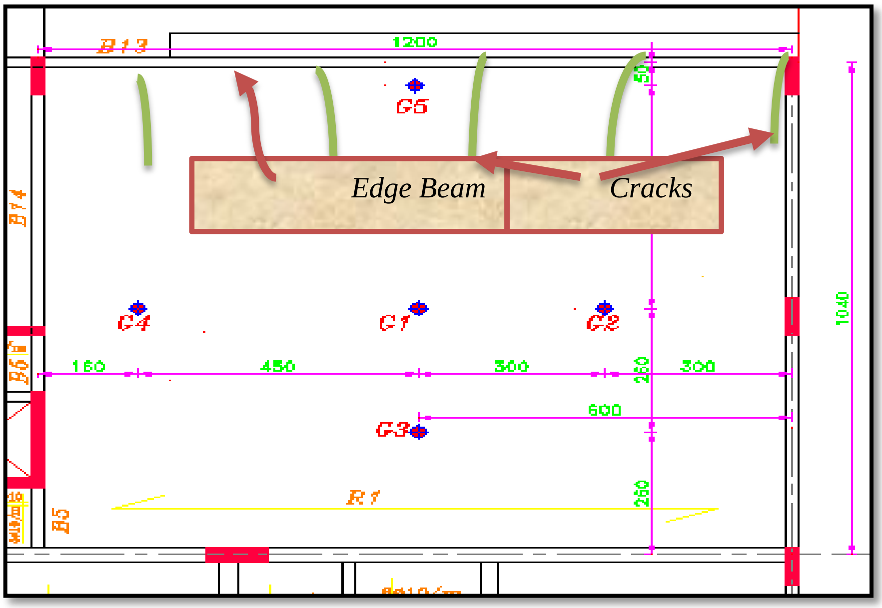

Fig. 5: Location of Dial Gauges.and crack of F.F.S

6.3. Simulation of Distributed Loading

The design loads were simulated by means of concrete blocks (30*20*15 cm) (length x width x height) and sample weight equal 15 kg for each block are relatively easy to install and control. For the structure under investigation. And For this purpose, this load is exactly similar of finish and live load because is uniformly distribution on all area of slab equality.(Fig. 6) and since the load from the concrete blocks distributed like exactly real load, the effects of the design load is Similar to the load test. This give the very good response of the slab system and allowed using lighter equipment with low cost. the load test of slabs was done in phases.

Fig.6 Weight of unit concrete block equal 15 kg/one

7. 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.

7.1. Load Intensity

Four load intensity levels were used. The recently published ACI 437.1R-07 [ACI 437, 2007] .Recommends that the load intensity as provided in Chapter 20 of 318-05 [ACI, 2005] be redefined as follows.

7.2. 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 [10],[11]

7.3. Load Calculation.

Following the Building Code Requirements for Structural Concrete ACI 318/318R - 300 Chapter 20.[11]

The Value test load shall be calculated.

Total Load = 0.85(1.4*Dead load + 1.7*Live load)*

$$\text{Test load } W = 0.85 * (1.4\ D + 1.7 * L) \qquad (1)$$

D.L i.e. Dead load Contain 1 - Finishes.

2 - Services and ceiling

L.L i.e. Live Load expected.

Typical One way Joist Slab:

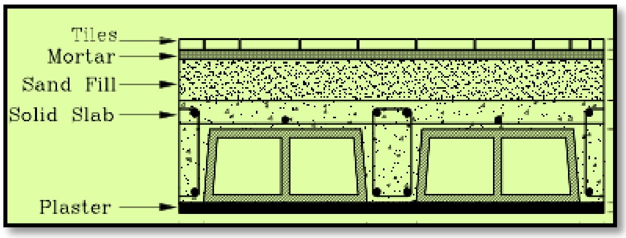

Fig. 7 Typical Cross-Section D. L. Calculation, Finishes

Floor Layers (tile2.5cm + mortar2.5cm +fill 12cm)
Tile 2.5cm U.W.=2.2T/m^3)
Mortar 2.5cm (U.W. =2.2 T/m
Fill 12cm Th. (U.W.=1.7T/m^3)

For Two Slabs First and Second Floor Roof:

1- Finishes Floor (tile2.5cm + mortar2.5cm +fill 12cm) =
*= 0.025*2200+0.025*2200+.10*1700= 0.280 Ton/m^2*
2 -Services and ceiling = 0.050 +0.50 = 0.100 Ton/m^2
*Total dead load = 0.280 + 0.100 = **0.380** Ton/m^2*
*Live Load = 0.2 x 1 = **0.200** Ton/m^2*
Test load = 0.85 (1 . 4 D + 1.7 *L)*
Total Test load W *= 0.85* (1 . 4 * **0.380** + 1.7 ***0.200**) = 0.740 Ton/m^2*

7.4. Load Configuration

The load was applied at of slab uniformly distributed at all area as shown in (Fig.8)
The intensity of the applied load at 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 contain 16.5 block *15 kg = 247 kg / m^2/layer

*Three layers *247 kg / m^2= 741 kg/ m^2 # **0.740** Ton/m^2*

The ACI requirements and standards for the structural using condition must be considered and limited by two variables that are:

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

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

$\Delta\ max \leq L^2/20000h$ (2)

$\Delta\ rebound \leq \Delta\ max /4$ (3)

Where: *Δ max* is the maximum deflection
Δ rebound is the Rebound deflection or Residual Deflection
L is length of slab on the short side, and h is thickness of slab

7.5. 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. (*G1*) is installed at the middle of the slab. The dial gage installation is used the magnetic base (Fig.9) and (Fig. 5)
2. Record all initial deflections and the temperature prior 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/m2).
4. except the maximum test load (100%) that has to maintain 24 hours (Fig. 4)
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 release all test load, it is maintained for 24 hours.

8-First Floor Roof Slab:

Fig. 8. Loading by Concrete Blocks uniformly distributed load

Fig. 9. Dial Gauges installation.in F.F.S

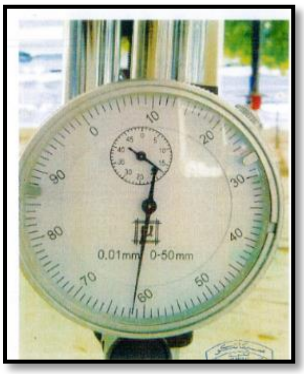

Fig**. 10**. Dial Gauge

8.1. Calculation of Maximum Allowable Deflection:

Slab Dimension = 12.0m x 10.5m x O.5m (thickness

Criteria I: Max Allowable Deflection: According to ACI 318

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

$\Delta max = L^2 / (20000*h) = (10500)^2 / (2000*500=$ <u>**11.025 mm**</u> (2)

Maximum Measured Deflection $\Delta meas. = 20.66$ *mm.*

$\Delta meas. = 20.66$ *mm* $> \Delta max = 11.025$ *mm*

Criteria II: Rebound Recovery Allowable Deflection:

According to ACI 318 $\Delta rmax \leq \Delta max/4$

8.2. Testing Results

The results from the load test are shown by the table 1 and graph (Fig. 5) respectively.

	Dial Gauge Reading (mm)	*G1*	*G2*	*G3*	*G4*	*G5*
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	Max. measurement dif.	***19.72***	***8.14***	***20.66***	***18.91***	***18.41***
During 3rd stage of loading the excessive deflection was measured, cracks appears in the slab and edge beam. its becomes wider for that we Stopped increase of the load						
	Dial Gauge Reading (mm)	*Gr1*	*Gr2*	*Gr3*	*Gr4*	*Gr5*
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		*14.52*	*6.45*	*16.15*	*16*	*12.88*
Dial Gauge Recovery Reading %		***74%***	***79%***	***78%***	***82%***	***70%***

Table 1: Dial Gauges Readings.

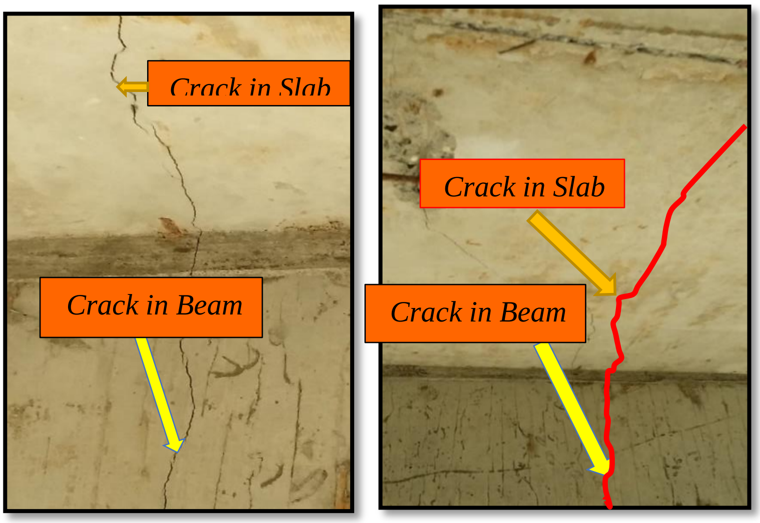

Fig. 11 Cracks at the edge beam and F.F.S

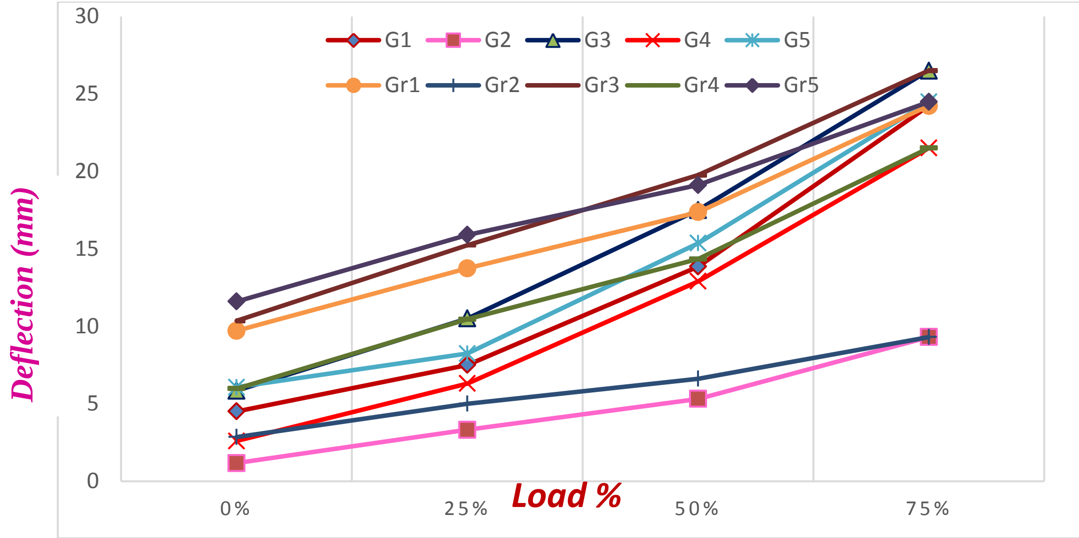

Fig. 12: Defl. (G), Rebound Defl. (Gr) and Max. Load Percentage for Dial Gauge

Refer to table 1 it clear that gage no. ***G1***, and ***G5*** Unverified

Dial Gauge No.	*Δmax*	*Δrmax*	*Δmax/4*	*Result*
G1	19.72	5.20	4.93	***Unverified***
G2	8.14	1.69	2.04	***Verified***
G3	20.66	4.51	5.165	***Verified***
G4	18.91	3.38	4.73	***Verified***
G5	18.41	5.53	4.60	***Unverified***

Table 2 Testing Defl. (Δ_{max} and $\Delta_{rebound}$) and Allowable Rebound Defl. ACI

Note: *1. all maximum deflections (Δ max) from testing must be less than the calculated deflection that equals **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.

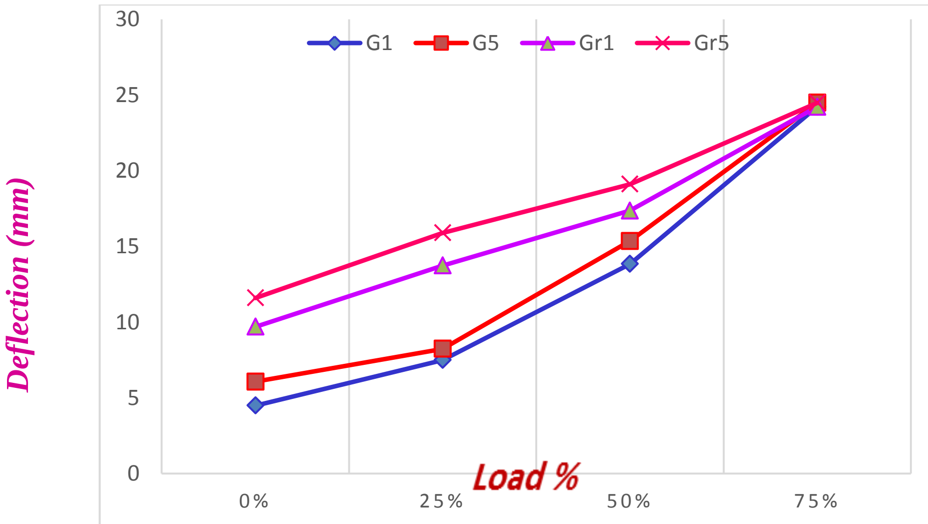

Fig. 13: Deflection and Maximum Load Percentage for No.G1, G5

8.3. Analysis of Load Test Results

The graph, that is shown in (Fig. 12), show 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 to 5.20mm. For the dial gauge No.5 (G5), the maximum slab deflection is 18.41 mm and the rebound deflection is 5.35 mm as shown in (Fig. 13). From the load test results, all maximum deflections (Δ 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 maximum deflections is more than allowable as equation (2) (11.025 mm) also before applying all of the load many cracks were appeared

“Structural member tested (First Floor Roof Slab) does not satisfy Criteria I or Criteria II”

9. Second Floor Roof Slab:

9.1. Load Calculation.

In this case the same Procedures test load in the first floor roof slab

Total Test load W *= 0.85* (1 . 4 * **0.380** + 1.7 ***0.200**) = 0.740 Ton/m^2*

Slab Dimension = 12.0m x 10.5m x O.5m (thickness)

*One Square meter contain 16.5 block *15 kg = 247 kg / m^2/layer*

*Three layers *247 kg / m^2= 741 kg/ m^2 # **0.740** Ton/m^2*

9.2. Calculation of Maximum Allowable Deflection:

Slab Dimension = 12.0m x 10.5m x O.5m (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 mm** (2)

Maximum Measured Deflection Δmeas. = 20.66 mm.

Δmeas. = 20.66 mm >Δmax= 11.025 mm

Criteria II: ***Rebound Recovery Allowable Deflection:***

$\Delta rmax \leq \Delta max/4$ ***According to ACI 318***

Fig. 14: Location of Dial Gauges.of S.F.R

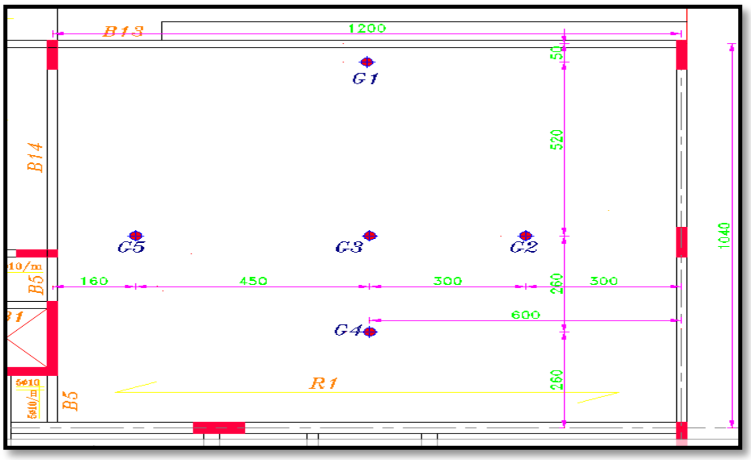

Fig. 15 Location of Dial Gauges on the plan of SFR

Fig. 16: Loading by Concrete Blocks uniformly distributed load

9.3. 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.

	Dial Gauge Reading (mm)	*G1*	*G2*	*G3*	*G4*	*G5*
1	Load 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
	Max. measurement dif.	***16.56***	***22.08***	***22.12***	***10.29***	***6.09***
	Result	***Unverified***	***Unverified***	***Unverified***	***Verified***	***Verified***
	Dial Gauge Reading (mm)	*Gr1*	*Gr2*	*Gr3*	*Gr4*	*Gr5*
1	Load 100% held for 24 h	20.81	30	30.55	***No Need***	***No Need***
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 24h	10.44	16.75	17.62		
	Net Deflection after Released Load	10.36	13.25	12.93		
	Dial Gauge Recovery Reading	***63%***	***60%***	***58%***		

Table 3: Dial Gauges Readings.

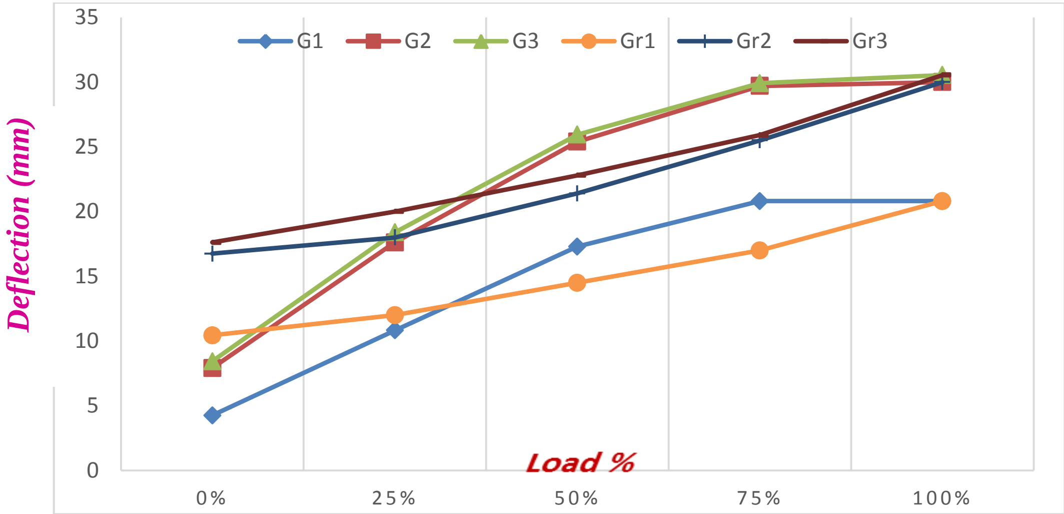

Fig. 17: Deflection (G), Rebound Deflection (Gr) and Maximum Load Percentage

b) **Rebound Recovery Allowable Deflection**

Refer to table 3it clear that gage No. *G1, G5 Unverified*

Dial Gauge No.	*Δmax*	*Δrmax*	*Δmax/4*	*Result*
G1	16.56	6.19	4.14	***Unverified***
G2	22.08	8.83	5.52	***Unverified***
G3	22.12	9.19	5.53	***Unverified***
G4	***No need because it verified from first criteria***			
G5	***No need because it verified from first criteria***			

Table 4: Testing Deflection (Δ_{max} and $\Delta_{rebound}$) and Allowable Rebound Deflections

Note 1. All maximum deflections (Δ_{max}) from testing must be less than the calculated deflection that equals 11.025 mm. (calculated from Equation (2)).
2. The rebound deflections must be less than the calculated rebound deflections that are shown in the four column of Table ***According to ACI 318*** $\Delta r_{max} \leq \Delta_{max}/4$

9.4. Analysis of Load Test Results

The graph, that is shown in (Fig. 17), show the relationships between the maximum deflection of the slab deflection for the dial gauge No.1, 3 (G1, G3) is (16.56-22.mm) and the rebound deflection is to (6.19-9.19 mm). Shown in (Fig. 17). But for G4, G5 no need to check rebound deflection because it's verified from first criteria.

From the load test results, all maximum deflections (Δ 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 maximum deflections is more than allowable as equation (2) (11.025 mm) and rebound deflections still exist in the slab structure as shown in Table 4.

"Structural member tested (First Floor Roof Slab) does not satisfy Criteria I or II"

10. Cutting, Demolition and Removal of Slabs:

After the experimental results that appeared in the loading test and the slabs were unsafe, according to the first and second procedures of the ACI-318, 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. (Fig. 18)

Fig. 18. Supporting the adjacent concrete slabs

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 m (Fig.19)

Fig.19. length of 1.30 meters

Fig.20. Reinforced concrete slabs after cutting

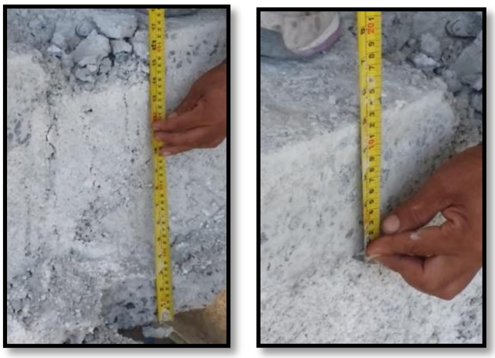

Fig.21.measuring of thickness of the rib and top slab

Fig.22. reinforced concrete slabs after cutting

Upon completion of the demolition and removal, important things were appeared:
* The total thickness of the executed slab ranges from 43 to 45 cm and does not match the structural design, which is 50 cm (Fig. 20, 21).
* The slab thickness is unequal on entire of area slab, its varies from place to other
* The thickness of the concrete slab above the ribs ranges from 8-12 cm and also does not correspond to the thickness of the design which is 15 cm
* The reinforcement is identical to the design, which is 6 # 16 per rib
* Concrete is weak in some places and positions of the slab
* Distribution of ribs, and spaces between ribs is irregular and unequal

11. Design checking:

After reference to the ACI38-05codes 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 not less than L / 20, 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

Minimum thickness h				
	Simply supported	One and continuous	Both ends continuous	Cantilever
Member	Members not supporting or attached to partitions of other construction likely to e damaged by large deflections			
One-way slabs	L/20	L/24	L/28	L/10
Beams or ribbed one-way slabs	L/16	L/158.	L/21	L/8

Table 5: Minimum thickness of one-way slab ACI

12. Verification of implementation:

measurement of thickness of concrete slabs executed after cutting showed 43 cm its less than thickness required as mentioned above in ACI 318 requirement which equal L/16=75 cm Additional does not match the thickness original design which done by consulting engineering office is equal to 50 cm as shown

13. Analysis and study:

From the mention above, and back to the loading experiments, the defects is found in:

1. Weakness of structural design carried out by the Engineering Consulting Office in terms of the dimensions of the rib section that do not comply with the ACI -318 requirements or international codes.
2. Defect in the implementation as the rib section of the performing and its dimensions do not match with the rib design section.

3. Bad distribution of ribs and space and distance between them, which are unequal
4. Change the thickness of the slab from place to another.
5. Contractor's Failure to apply the technical conditions and specifications for the execution of the concrete works for example there are gaps and empty spaces in the concrete
6. The strength of the reinforcement bars not tested.
7. as well as the concrete was not tested and may be weak because the results showed and during the cutting and removal is weakness in some places

14. Conclusions and recommendations:

Based on the study, discussion and analysis mentioned above, the following can be inferred:

1. Demonstrated that the loading experiments in Chapter 20 of ACI-318 provide logical results, expressive state and strength of the studied structural component.
2. The loading test showed that the concrete slabs were more elastic because the deflection was relatively large compared to the initial deflection, because the reinforcing steel was acceptable
3. During the third phase of loading the slab in the roof of the first floor cracks appeared at the free end of the slab and did not appear on the other side despite the same design and this indicates the contribution of a top slab link above the ribs with adjacent slabs in carrying structural work loads
4. The causes of cracks and weak slabs resulted from weak design and defect in implementation and lack of supervision because they are not complying with the international codes
5. Lack of application of the requirements and specifications of international codes in the structural design of buildings and structural elements by the consultant engineering office and no control and audit of these designs.
6. Lack of application of the conditions and specifications stipulated in international codes in the implementation of buildings.
7. Lack of supervision and quality control by the supervisor engineering office and lack of application of technical conditions for the design and strict compliance with drawings and technical specifications for construction and buildings.
8. Inadequate implementation by construction contractors to apply technical conditions for the design and strict compliance with drawings and technical specifications of building.
9. We recommend that the contractors selected for a project should classified, professional and registered in the committee of construction industry.
10. Slabs were unsafe to withstand the uniformly distributed maximum load because the permissible deflection exceeded allowable.
11. This work achieves structural strength through the load test of the slab structure, and this work performs both a non-destructive assessment (NDT) and a load test in order to know the strength of the slabs.
12. The surrounding member structures that support the slab were insufficient to support the load design because they showed no signs of damage and cracking during the test. But there is not enough force to prevent deflection.
13. The existing cracks exceeded the spread during the test.

For the previous causes, the repair work is not useful.

References

[1]. Galati, N., et al., In-situ evaluation of two concrete slab systems. I: Load determination and loading procedure. Journal of Performance of Constructed Facilities, 2008. **22**(4): p. 207-216.

[2]. Goble, G.G., Geotechnical related development and implementation of load and resistance factor design (LRFD) methods. Vol. 276. 1999: Transportation Research Board.

[3]. Galati, N. and T. Alkhrdaji, In Situ Evaluation of Structures Using Load Testing, in Forensic Engineering 2009: Pathology of the Built Environment. 2010. p. 657-667.

[4]. Mettemeyer, M. and A. Nanni, CENTER FOR INFRASTRUCTURE ENGINEERING STUDIES.

[5]. Luping, T. and L.-O. Nilsson, Rapid determination of the chloride diffusivity in concrete by applying an electric field. Materials Journal, 1993. 89(1): p. 49-53.

[6]. Ridge, A.R. and P.H. Ziehl, Evaluation of strengthened reinforced concrete beams: cyclic load test and acoustic emission methods. ACI Structural Journal,2006. **103**(6)

[7]. Illidge, F.A.B., Acoustic emission techniques and cyclic load testing for integrity evaluation of self-compacting normal and self-compacting lightweight prestressed concrete girders. 2010, University of South Carolina.

[8]. Code, A.B., 318,“. Reguirememfor Structural Concrete and Commentary,” American Concrete Institute, 1995: p. 37-38.

[9]. Committee, A., A.C. Institute, and I.O.f. Standardization. Building code requirements for structural concrete (ACI 318-08) and commentary. 2008. American Concrete Institute.

[10]. Comm ittee, A., Building code requirements for structural concrete (ACI 318-14) and commentary. ACI, Farmington Hills, United States, 2014.