

Experimental Time-Dependent Deflection of High Strength Concrete Panels

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Abstract This work presents an experimental study on both short-term and long-term deflections for a period more than seven months of reinforced high and normal strength reinforced concrete two-way flat plate panels. Four reinforced concrete panels with dimensions of (960×960×50) mm were investigated with simply supported edges. Concrete cube strength is about 75 MPa for HSC panels, and 30 MPa for the control panel. Sustain load kept along the entire test period and at the end, the loads is removed and the recoverable deflection was measured. It's been concluded that the rate of increase in HSC panels' long time deflection is less than that of NSC panels, also the long-term deflection is highly reduced by increasing the compressive strength of concrete and the long term deflection reduced about 20% when compressive strength increased from 25 to 65 MPa.

Keywords: concrete panels, high strength, Two-Way Slab, Long and Short -Term Deflection

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1. Introduction

Normal strength concrete by the American Concrete Institute (ACI) [1] definition is a concrete that has a cylinder compressive strength not exceeding 42 MPa (6000 psi). All other concretes are considered high-strength concretes (HSC). Concrete with a cylinder compressive strength of 140 MPa (20,000 psi) is used in high-rise structures in the United States and Europe [2]. In certain laboratories, exotic concretes of 315 MPa (45,000 psi) have also been produced [3]. Since concrete has to be reinforced for structural use, the level of ductility of the reinforcement becomes the limiting factor, since higher strength concrete tends to exhibit lower ductility.

This work, based on the experimental program conducted, discusses long-term deflection of HSC slabs. Experimental work for long-term deflection of HSC slabs is made.

2. Long Term Deflection

For a concrete structure, to be serviceable, cracking must be controlled and deflections must not be excessive. It must also not vibrate excessively. Concrete shrinkage plays a major role in each of these aspects at the service load behaviour of concrete structures.

Serviceability failures of concrete structures involving excessive cracking and/or excessive deflection are

relatively common. Numerous cases have been reported, in Australia [4] and elsewhere [5], of structures that complied with code requirements but still deflected or cracked excessively. In a large majority of these failures, shrinkage of concrete is primarily responsible. Clearly, the serviceability provisions embodied in our codes do not adequately model the in-service behaviour of structures and, in particular, fail to account adequately for shrinkage. In the case of HSC, the difficulty increased further.

The quest for serviceable concrete structures must involve the development of more reliable design procedures. It must also involve designers giving more attention to the specification of an appropriate concrete mix, particularly with regard to the creep and shrinkage characteristics of the mix, and sound engineering input is required in the construction procedures. High strength concrete structures require high standard of construction, involving suitably long stripping times, adequate propping, effective curing procedures and rigorous on-site supervision.

3. Experimental Investigation

Stoekl [6], reported the results of 15 years of sustained load study. He reported results for concrete with 28 days cylinder strength up to 50 MPa. Failures of specimens loaded to 70% to 75% of the short-term strength are reported even for 50 MPa concrete. He concluded that the sustained load strength was around 80% of the ultimate, regardless of the compressive strength and the eccentricity.

Ashour and Mahmood [7], presented an experimental and theoretical study on the influence of steel fibers and longitudinal tension and compression reinforcements on immediate and long-term deflections of high strength concrete beams of 85 MPa compressive strength. Test results of eighteen beams subjected to sustained load for 180 days show that the deflection behaviour depends on the longitudinal tension and compression reinforcement ratios and fiber content; excessive amount of compression reinforcement and fibers may have an unfavourable effect on the long-term deflections. The beams having the ACI Code's minimum longitudinal tension reinforcement showed much higher time-dependent deflection to immediate deflection ratio, when compared with that of the beams having about 50% of the balanced tension reinforcement. The results of theoretical analysis of tested beams and those of a parametric study show that the influence of steel fibers in increasing the moment of inertia of cracked transformed sections is most pronounced in beams having small amount of longitudinal tension reinforcement.

Ghali [8] in 1993 presented a proposed code changes for prediction of immediate and long-term deflections of reinforced concrete members. This study showed that the prediction of the immediate and long-term deflections of reinforced concrete members could be inaccurate for two reasons. First, the uncertainty of the material parameters such as modulus of elasticity, creep coefficient, shrinkage. The second is the use of an inadequate method of analysis.

He concluded also that long-term deflection cannot be predicted accurately by the use of the ACI Code multiplier (λ) because the equation does not include several parameters that influence the deflection.

Hall and Ghali [9] in 2000 presents the results of an experimental investigation of the long-term deflection behaviour of concrete shallow beams reinforced with glass fibre reinforced polymer (GFRP) bars. The long-term deflections of the GFRP-reinforced beams are compared to deflections of identical beams reinforced with steel bars. All beams were under sustained loading for approximately 8 months.

The variables were the level of sustained loading and the reinforcement materials: steel or GFRP. The experimental immediate and long-term deflections of both the steel and the GFRP-reinforced beams were compared to calculated deflections using the CEB-FIP Model Code 1990, and the ACI 318-95 code using the recommendations of ACI Committee 209; these references are for steel reinforced concrete members.

The test results indicate that under similar loading conditions and the same reinforcement ratio, the GFRP-reinforced beams had long-term deflections, due to creep and shrinkage, 1.7 times greater than those of the steel-reinforced beams. A comparison of the theoretical and experimental immediate and long-term deflections indicates that the CEB-FIP Model Code 1990 gives reasonable predictions for all beams, and that the ACI 318-95 code, using the ACI Committee 209 recommendations, overestimates the deflections due to the combined effects of creep and shrinkage.

Jawad [10] in 2000 presented a theoretical and field investigation of the long-term deflections concrete two-way slab systems. Extensive field measured two-way slab deflections were made in this work. A multi-storey

building with 96 panels was investigated. Measurements were taken using standard levelling technique. The collected data were used to furnish three proposals of a long-term multiplier value suitable for Iraqi conditions. In comparison with various multipliers recommended elsewhere, the second proposal is found to be the best acceptable one according to the local field measurements. A proposed model for calculating the long-term deflections of two-way slabs is presented to be suitable for Iraqi conditions.

4. Materials and Experimental Program

4.1. General

To produce high strength concretes, several parameters have to be optimized in addition to mix design, although the design of the concrete mixture is a major factor in achieving the desired strength. Several methods can be applied to achieve high strength concrete. In general, high-strength concrete contains strong aggregates, a higher Portland cement content, and a low water/cement or water/cementations ratio. The addition of water-reducing admixtures, superplasticizers, polymers, blast furnace slag or silica fume is common today [2].

4.2. Materials

4.2.1. Cement

Ordinary Portland cements (OPC) was used in the experimental program. It is produced by Al-Sabe'a Lebanon factory. The cement was kept in closed plastic containers throughout the experimental work to keep the cement in a good condition and to minimize the effect of humidity. Its properties are conformed to the Iraqi specifications No.5/1984 [11].

4.2.2. Fine Aggregate (sand)

Al-sela natural sand with maximum size of 4.75 mm was used throughout this work. The grading of the sand was conformed to the Iraqi specification No. 45/1984 [12].

4.2.3. Coarse Aggregate

Crushed gravel from AL-Nibaey region was used throughout this work. According to the recommendations of ACI 211.4R-93 [13] for mix selection of high performance concrete, the maximum size of 10 mm (3/8 in.) for the crushed gravel was selected. The crushed river gravel coarse aggregate were washed, then stored in air to dry the surface, then stored in containers in a saturated surface dry condition before using.

4.2.4. Superplasticizer

High range water-reducing admixture called SP-1 was used throughout the experimental work. The superplasticizer was produced by (Al-AZRAK Company, Jordan) and it is complied with ASTM C494 type A&F as described in the manual of the product.

4.2.5. Mixing Water

Tap water was used for casting and curing all the specimens.

4.2.6. Steel Reinforcing Mesh

One size of normal strength steel wires was used. Wires of size ($\phi 2.5\text{mm}$) used as a bottom mesh reinforcement with 5 mm concrete cover. Yield strength of the wires was determined by tensile test. Results of test showed that the yield strength of the wires of ($\phi 2.5\text{mm}$) equal to 420 MPa. The number of wires was (21) in each direction at the bottom face.

5. Experimental Program

Table 1. Details of the concrete mixes used in slabs used for Long Term Deflection

Index	Slab ID	Cement Kg/m ³	Aggregate content		Water content kg/m ³	S.P% by wt. of Cement	W/C Ratio	Vebe time sec.
			Sand kg/m ³	Gravel kg/m ³				
High strength	L1	550	665	990	170	1.4	0.32	8
	L2	550	665	990	169	1.4	0.32	14
	L3	550	665	990	178	1.45	0.336	13
Normal Strength	L4	350	525	1050	175	--	0.5	6

5.2. Mixing, Casting and Curing Procedure

Four slabs of (960×960×50) mm were cast corresponding to the different types of concrete mixes. These slabs marked (L1, L2, L3 and L4) as shown with the details in Table 1 above. For each slab, three (100 mm cube) of concrete were cast. The same mix procedure used in the first part of experimental work was adopted in this part of experimental work. Each slab reinforced with one bottom steel mesh of (21 $\phi 2.5$ Each Way). Figure 1 shows details of the slabs.

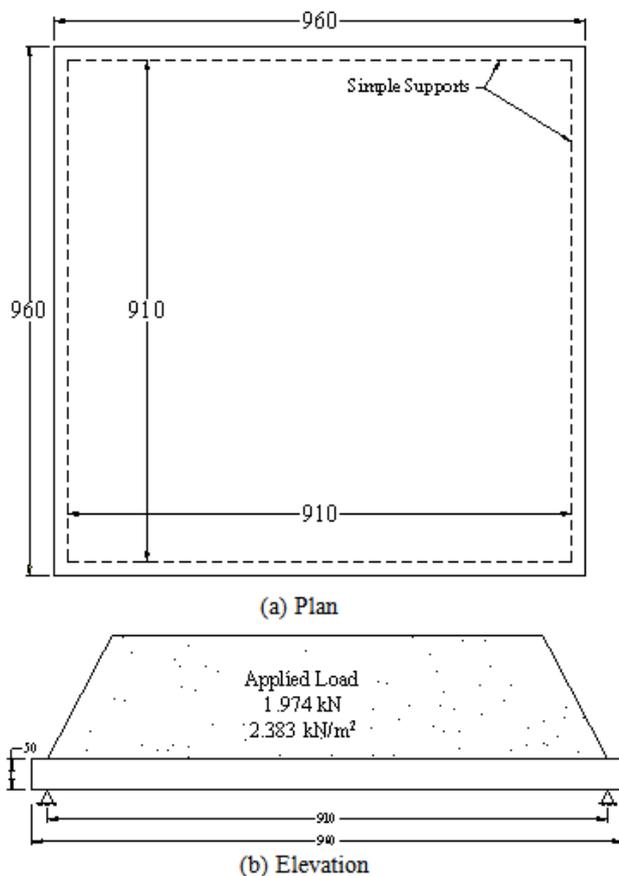


Figure 1. Details of slabs for long time deflection test, all dimensions in (mm)

5.1. Mix Design

According to the recommendations of the ACI 211.4R 93 [13] several trial mixes were made. Reference concrete mixture was designed to give a 28-day characteristic compressive strength of 65 MPa. The cement content was 550 kg/m³ (and 350 kg/m³ for normal concrete), water/cement ratio was 0.32, (1.4%) superplasticizer by weight of cement, and proportions of mix was found to be [1:1.21:1.8] by weight. Table 1 shows the details of the different types of the concrete mix used in this work.

5.3. Testing Specimens

All slabs were tested at 28 days age. Before the day of testing, the slabs were taken out from the container of curing, cleaned and painted. All slabs were tested on simple supports and in pure bending by uniform load (sustained load produced by sub-base layer placed on the panels) as shown in Figure 2.

One dial gauge (with an accuracy of 0.01 mm) was placed at the center of the bottom face of each panel for reading the deflection after different periods. Nylon sheets were placed on top of slabs to separate concrete from the sub-base load layer. The load of sub-base layer and slab load are (2.383 kN/m²).



Figure 2. Slabs for long-time deflection test

6. Panel Specimens Test Results

6.1. Short-Term Deflections

Table 2. Measured Short-Term Deflection

Panel No.	Age of Panel at Loading (Days)	f_{cu})28 MPa *	f'_{c})28 MPa **	Measured (Δ s)mid. (mm)
L1	10	74.7	63.5	0.932
L2	10	76.2	64.77	0.904
L3	10	76.6	65.1	0.974
L4	10	29.1	24.74	1.105

* Cube Compressive Strength.

** Equivalent cylinder comp. strength as per reference 16.

Deflections are recorded immediately after the panels are loaded in place on their supports. Table 2 summarizes the panels tested, giving actual compressive strength of concrete, age of concrete panel when loaded and measured short-term deflections. The instantaneous deflections are measured for all panels at an age of 28 days [14,15].

6.2. Long-Term Deflections

To evaluate the time-dependent deflection behavior, it is necessary to load the panels for a long period of time.

Deflection measurements were taken over a seven-month period under the predetermined level of sustained load permit assessment of this effect.

Plots of the measured midspan deflection versus time for the four panels are shown in Figure 3. It can be seen that the deflection increases with time due to the effects of creep and shrinkage. However, the rate of increase of deflection decreases with time. Also, it is clear that the increase in concrete strength reduces the long-term deflections.

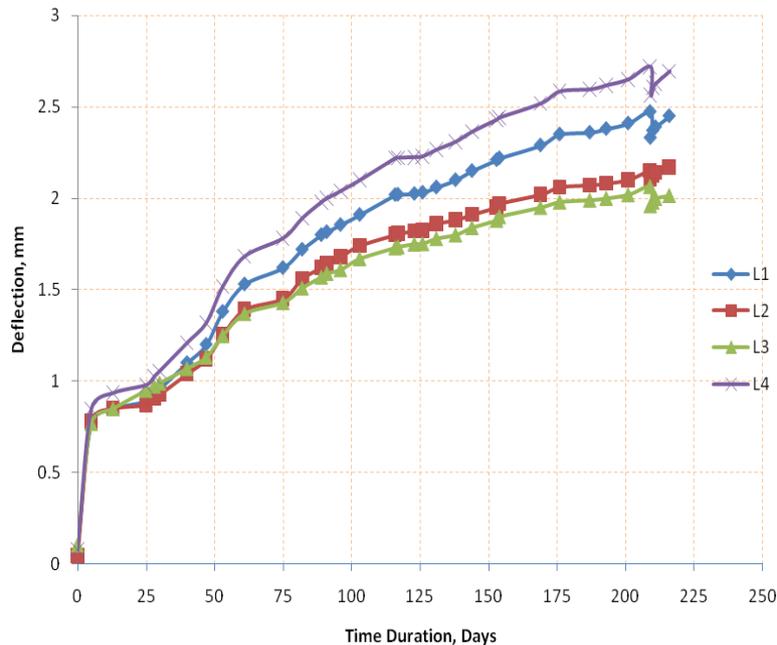


Figure 3. Measured mid-span deflection versus time for the panel L1 to L4

A large increase in deflection is noticed during the first three months after the load is applied. After that, the rate of increase became smaller as time progressed, i.e., in all panels there is a substantial increase of deflection followed by a period where the deflection increases are

minimal. Table 3 summarize the measured long-term deflections for each loaded panel. It can be seen that the rate of increase in HSC panels' long time deflection is less than that of NSC panels.

Table 3. Measured Long-Term Deflection for all panels

T in days	L1		L2		L3		L4	
	meas (ΔL) mm	ratio ΔL / ΔS	meas (ΔL) mm	ratio ΔL / ΔS	meas (ΔL)mm	ratio ΔL / ΔS	meas (ΔL) mm	ratio ΔL / ΔS
28	0.932	1.00	0.904	1.00	0.974	1.00	1.056	1.00
30	0.96	1.03	0.927	1.03	0.99	1.02	1.058	1.00
40	1.1	1.18	1.04	1.15	1.07	1.10	1.074	1.02
50	1.29	1.38	1.187	1.31	1.191	1.22	1.197	1.13
60	1.511	1.62	1.373	1.52	1.355	1.39	1.359	1.29
70	1.588	1.70	1.429	1.58	1.409	1.45	1.51	1.43
80	1.691	1.81	1.529	1.69	1.487	1.53	1.515	1.43
90	1.808	1.94	1.631	1.80	1.58	1.62	1.585	1.50
100	1.886	2.02	1.714	1.90	1.644	1.69	1.648	1.56
110	1.969	2.11	1.772	1.96	1.73	1.78	1.738	1.65
120	2.023	2.17	1.813	2.01	1.743	1.79	1.777	1.68
130	2.054	2.20	1.852	2.05	1.774	1.82	1.879	1.78
140	2.117	2.27	1.89	2.09	1.813	1.86	1.957	1.85
150	2.19	2.35	1.937	2.14	1.867	1.92	2.107	2.00
160	2.248	2.41	1.99	2.20	1.922	1.97	2.287	2.17
170	2.299	2.47	2.026	2.24	1.954	2.01	2.314	2.19
180	2.354	2.53	2.064	2.28	1.984	2.04	2.451	2.32
190	2.37	2.54	2.075	2.30	1.995	2.05	2.574	2.44
200	2.406	2.58	2.098	2.32	2.018	2.07	2.652	2.51
210	2.48	2.66	2.15	2.38	2.07	2.13	2.725	2.58

The ratio of deflection increments between the 28-days and 7-months periods vary from 27 to 34 % for HSC panels in average. The corresponding ratios for NSC panels vary from 16 to 25 % (10, 17, 18, 19, 20).

At the end of testing period of seven months, the load is removed and the rate of deflection recovery is monitored. Results at the end of seven months testing are summarized in Table 4, where (Δ_s), (Δ_i), (Δ_r), and (Δ_{ir}) are short term deflection at the center of the panel, experimental deflection increment, recoverable deflection, and irrecoverable deflection, respectively.

Table 4. Test Results of Sustained Loading

Panel No.	short term deflection Δ_s (mm)	Exp Def. increment Δ_i (mm)	Recoverable deflection Δ_r (mm)	Irrecoverable deflection Δ_{ir} (mm)
L1	0.932	1.548	0.11	2.37
L2	0.904	1.246	0.11	2.04
L3	0.974	1.096	0.11	1.96
L4	1.056	1.669	0.14	2.585

7. Conclusions

Time dependent deflection up to 210 days was measured for three HS concrete panels (~65 MPa) and one control panel with (24 MPa).

The following conclusions can be drawn based on the results of this work;

1. The overall trend of deflection-time curves is similar. The high strength panels showed similar or little less values at 1st week and clearly less values at later days.
2. The 210-days center panel deflection for control panel was (2.725) mm, while it was (2.48, 2.15, and 2.05) mm for high strength panels. Ratio of $\Delta_{HS} / \Delta_{control}$ are (0.91, .79, and 0.75), indicating an overall better performance of HS concrete panels in long term deflection.
3. The 210-days ratio of long term to short term deflection (Δ_L/Δ_S) for control panel was (2.58), the corresponding ratios for HS panels are (2.66, 2.38, and 2.13) with an average of (2.39). They are less than the control panels, however, the values of ACI ratio (2.0) for beams (used for slabs) seems doesn't apply. A values of 2.5 as suggested by Branson [17] sound suitable for normal and HS concrete slabs.
4. A large increase in deflection is noticed during the first three months after the load is applied. At the last three months, the rate of increase became smaller as time progressed.
5. Deflection recovery, when panels were unloaded, had been detected and found to be about 15%, 18%, 20% and 40% for panels with cube compressive strength 74.7, 76.2, 76.6 and 29.1 MPa respectively.

Abbreviations

HSC	: High Strength Concrete
NSC	: Normal Strength Concrete
S.P	: Supper Plasticizer
Meas	: Measured
W/C	: Water / Cement Ratio
f_{cu}	: Cube Concrete Compressive Strength
f_c'	: Cylinder Concrete Compressive Strength
Δ_S	: Short Term Deflection
Δ_L	: Long Term Deflection
Δ_i	: Exp Def. increment
Δ_r	: Recoverable deflection
Δ_{ir}	: Irrecoverable deflection

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