EFFECT OF NON-STANDARD CURING AND CONSOLIDATION ON STRENGTH OF RC BEAMS

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ABSTRACT

Reduction in concrete compressive strength and flexural strength of reinforced concrete beams due to inadequate consolidation and curing was assessed in this study. The casting, consolidation, and curing were conducted outdoors during dry and hot summer of Riyadh (Saudi Arabia). Three groups of specimens were prepared using a good quality ready-mixed concrete. Each group comprised of ten reinforced concrete beams, five plain concrete beams for core testing and thirty standard cylinders.

All specimens were tested 28 days after casting. Reinforced beams were tested under two point loads, and cores and cylinders were tested under direct compression. The results indicated that although the inadequate consolidation and curing of concrete in hot and dry weather caused a significant reduction of more than 20 percent in the compressive strength, all the test beams maintained flexural strength at least 20 percent higher than the code nominal flexural strength.

INTRODUCTION

Based on local field survey [1] it was found that light rodding and inadequate water spraying were the commonly used methods for consolidation and curing of concrete in Saudi Arabia. Almost all local design offices assume conservative values of concrete compressive strength in their design regardless of the quality of concrete that will be used. The absence of local recommendations on this aspect was the other finding of the field survey. The survey indicated that in order to improve the local design practice and to formulate appropriate recommendations, there is a need for a wide range of research on variability in quality of locally produced concrete, effect of local weather conditions and commonly used curing and consolidation methods on concrete quality. The effects of these factors on the concrete quality should be studied in a systematic manner in order to reach economical, practical and efficient recommend-ations.

The validation and efficiency of recommendation of some international codes being practiced locally should be evaluated in the light of local conditions. The work described in this paper is part of a research being performed [1,2] to address these problems and to improve the reinforced concrete building design practice in the Kingdom of Saudi Arabia.

The American Concrete Institute Building Design Code, ACI-318, [3] recommendations limit the allowable reduction in the compressive strength of the field cured specimens by 15% of the companion laboratory-cured specimens. When this limit is exceeded, the Code requires core test or loading test. Both of these tests are usually undesirable, relatively expensive and difficult, and may adversely affect the member strength. The ACI-318 Code recognizes such problems and implies that the analytical investigation should be considered as the first option for the evaluation of load carrying capacity of the member. However, this option is presented in an ambiguous way. It is not clear if the analytical investigation should be performed using only the methods accepted by the Code or more refined methods available in the literature can be used. This study attempts to highlight this issue of analytical investigation.

SCOPE OF THIS INVESTIGATION

Reduction in concrete compressive strength and flexural strength of a reinforced concrete beam due to inadequate consolidation and curing were investigated in this study. The casting and curing were conducted outdoors during the dry and hot summer of Riyadh. The field temperature and humidity were 42 °C and 10% respectively. The scope of the study was limited to good quality concrete having relatively low temperature and improved workability. It was intended to simulate, as close as possible, the prevalent construction practice in Riyadh city.

RESEARCH SIGNIFICANCE

The study presents findings of an experimental work conducted outdoors in the hot and dry weather of Riyadh city. Consolidation of concrete in the forms and curing of the tested specimens were varied from poor to acceptable levels of practice. The test results were evaluated and compared to study the effect of these conditions on concrete strength and can be of interest to practicing engineers and building officials, specially in communities where proper regulatory control on construction of concrete structures is not available.

EXPERIMENTAL PROGRAM

Three different types of tests, shown in Fig. 1, were used to achieve the objectives of the study.

1. Group C (Compressive Strength Test of Standard Cylinders)

Compressive strength tests on standard cylinders (150x300 mm) comprised three subgroups (CS, CC, and CN) of thirty specimens each molded in accordance with ASTM C 31 [4]. The specimens of the subgroup CS were subjected to moist standard laboratory curing in accordance with ASTM C 31 [4]. The subgroup CC were cured in the field with burlap cover and water sprayed twice daily for seven days. The subgroup CN was cured in the field by sprinkling water only (twice daily and for seven days).

2. Group P (Compressive Strength Test of Cores)

A total of 50 cores were taken from plain concrete beams in accordance with ASTM C 42 [5] and divided into two subgroups. Twenty five cores of subgroup PI were taken from five beams which were compacted mechanically and cured with damp burlap for seven days (as subgroup CC above), while the other 25 cores of the second subgroup PN were taken from five beams which were rodded in three layers and spray cured with water only (as subgroup CN above).

3. Group R (Flexural Test of RC Beams)

Under reinforced concrete beams for flexural test were subdivided into three subgroups of ten beams each. The subgroups RI and RA were cast and cured as subgroups PI and PN above, respectively. The beams of subgroup RP were cast in a single layer and compacted by ten tampings over the entire length of the beam. It was cured by sprinkling once a day for three days.

All specimens were tested 28 days after casting. RC beams were tested under two point loads, see Fig. 2, while cylinders and core specimens were tested under direct compression in accordance with ASTM C 39 [6]. Figure 2 also shows the dimensions and reinforcement details of the test beam. The longitudinal reinforcement ratio was 0.67 ρ_{max} [3] which is assumed to be a practical value. The concrete was obtained from a ready mix concrete plant in Riyadh city. The mix was designed to obtain 28-day cylinder nominal compressive strength of 33 MPa and average slump of 100 mm at an average temperature of 33 ^{o}C (at site). The mix proportions which was kept constant throughout the study are given in Table 1.

Nominal Moment Capacity

The moment capacity of the tested RC beam was calculated by different methods as follows:

1. According to the ACI-318 assumption employing the Whitney rectangular block [3] for modeling the concrete compressive stress-strain relationship, the nominal moment capacity is equal to 35.73 kN.m. The moment capacity is based on concrete nominal compressive strength, f'_c, of 33 MPa, and nominal steel yield strength, f_y, of 412 MPa (grade 60 Saudi rebar) and referred to in this paper as M_{ACI}.

2. A more refined prediction of the moment capacity was made using the following assumptions:

(a) The relationship between concrete stress and strain is assumed to follow Hognested et al model [7], shown in Fig. 3(a). Stress-strain distribu-tion in the compressive zone of the section was divided into two parts. The first one was assumed to be parabolic up to a strain value of 0.002. The second part was considered to be linear up to an ultimate strain, ε_{cu}, of 0.0035. The slope, Z, of the linear part depends upon the ratio of the confinement steel (stirrups). For the core of the RC beams considered in this study the slope was estimated to be 173 [8], while for the concrete cover a value of 250 was assumed.

(b) The properties of the reinforcing steel were determined from the results of tests conducted in KSU structural laboratory on ϕ14 mm Saudi reinforcing steel [1]. Based on the average of 20 samples tested, the following properties were adopted for the stress-strain behavior shown in Fig. 3 (b).

f_{ya} (average yield strength)	=	480 MPa
f_{sh} (average value)	=	515 MPa
f_{su} (average value)	=	623 MPa
Yield strain ε_y	=	0.0024
Strain hardening strain, ε_{sh}	=	0.004
Modulus of elasticity, E_s	=	200 GPa
Strain hardening slope, E_{sh}	=	58.86 GPa

Using a computer program, based upon the above refined model, the moment capacity, which is referred to as M_H, was calculated as 53.11 kN.m.

The theoretical effect of the reduction in the in-situ compressive strength of the concrete on M_H was calculated and presented, in Fig. 4, as a plot of f_c/f'_c versus M/M_H. The parameter f_c is an assumed variable which represents the in-situ compressive strength while

f'_c is the nominal compressive strength of 33 MPa. The parameters M and M_H are the variable bending moment and bending moment capacity corresponding to f_c and f'_c respectively.

EVALUATION OF RESULTS

Nominal Compressive Strength

The average compressive strength, $\overline{f}_{cs}$, of the thirty specimens from the standard laboratory tests (subgroup CS) was 34.34 MPa with standard deviation of 1.43 MPa. According to Section 5.3 of the ACI-318, these values maintain a specified compressive strength of 32.4 MPa which is close to the value of 33 MPa assumed at the mix design stage. Also, it was found that the requirements of Section 5.6.2 of the ACI-318 Code for acceptance of concrete under the laboratory conditions were satisfied.

Reduction in Compressive Strength

The average compressive strength obtained from the standard cured cylinders, CS, was 34.38 MPa. This value was considered as a reference for the evaluation of reduction in the compressive strength obtained in both field cured cylinders and cores. The mean value of subgroups CC and CN were 29.21 and 27.82 with reduction of 15 and 19 percent when compared with results obtained from CS subgroup. These reductions in strength were mainly attributed to the field curing because both subgroups were compacted by the same method.

The ACI committee 305 (Hot Weather Concreting) [9] reports that the test specimens molded and cured in air at a temperature of 38 ^{o}C and 25 percent relative humidity produced only 62 percent strength of that obtained for standard specimens moist-cured at 23 ^{o}C for 28 days. The strength produced in the test program under more hot weather condition, however, was significantly higher (81 percent of the standard specimens). The limitted reduction in the compressive strengths of test specimens may be attributed to the improved workability and relatively low temprature concrete used in the study.

The ACI-318 Code, Section 5.6.3, requires that procedures for curing concrete shall be improved if strength of field-cured cylinders at test age designated for determination of f'_c is less than 85% of the companion laboratory-cured cylinders; the 85% may be waived if field cured strength exceeds f'_c by more than 3.5 MPa. Therefore, the field cured specimens of subgroup CC are accepted as they fullfilled the requirements of the ACI-318 Code mentioned above. On the other hand, subgroup CN specimens are not accepted.

The ACI-318 Section 5.6.4 recommends core tests if tests of field-cured cylinders indicate deficiencies in curing. This requirement was achieved in this study through cores subgroups PI and PN. The mean values of compressive strength of core specimens in subgroups PI and PN respectively were 29.0 and 27.0 MPa, as shown in Table 2, with average reductions of 15% and 21% compared with the average compressive strength of subgroup CS.

The provisions of the ACI-318 Code consider the concrete to be structurally adequate if the average strength of three cores is not less than 85% of f'_C and if no single core is less than 75% of f'_C. It can be seen from Table 2 that in the case of PI specimens the reduction in the compressive strength observed is within the limit provided by the ACI-318 Code, but for the subgroup PN reduction is higher than the prescribed limit.

Reduction in Flexural Strength

The moment capacity based on the experimental load measured in the test is summarized in Table 3 and compared with the design moment capacity. The mean values of flexural capacity of subgroups RI, RA and RP were 51.13, 47.5 and 45.3 kN.m, respectively. The corresponding values of coefficient of variation were respectively 2.8, 3.67 and 4.18 percent. Fig. 5 presents standard deviation error bars for the three subgroups.

As shown in Table 3, the average reductions in moment capacity of subgroups RA and RP were 7% and 12% respectively when compared with the average capacity of subgroup RI. Fig. 4 indicates that these reductions theoretically correspond to 16% and 27% reductions in the in-situ compressive strengths which are more than the ones observed from the field cured cylinders discussed earlier. This indicates that improper consolidation and curing of reinforced concrete members may cause a reduction in compressive strength in the member higher than that indicated by field specimens cured in a manner similar to the member. The reduction in compressive strength of field cured specimens reflects only the effect of curing while the reduction in the in-situ compressive strength reflects the effect of curing and consolidation. It can be said, however, that the reduction in the concrete compressive strength has a little effect on the flexural capacity of under-reinforced concrete beams. Such a conclusion was reported also by Soroushian et al [10], but, at the same time, they found that the strength of RC column is very sensitive to reduction in the concrete compressive strength. It was reported that changing the compressive strength by ± 30 percent results in about ±3 percent and ±25 percent variations, respectively, at the flexural strengths and compressive strengths of RC sections. It is well documented [11,12] that the poor construction practice has a harmful effect on the durability of RC members.

The moment capacity predicted using Hognestad Model, M_H of 53.11 kN.m, was in a good agreement with the results obtained in case of subgroup RI, 51.13 kN.m. Therefore, one can say that the actual moment capacity of a reinforced concrete member may be predicted precisely if the actual properties of steel and concrete are appropriately incorporated in the calculations.

Figures 6, 7 and 8 show the relationship between the total applied load and the strain in the bottom reinforcing steel measured at mid-span during the test for beam subgroups RI, RA and RP respectively. It can be noted that, the plastic deformation (deformation beyond the yield point) of reinforcing steel was the largest for the beams of subgroup RI, the beams of low reduction in in-situ concrete compressive strength, and the least for the beams of subgroup RP, the beams of large reduction in the in-situ compressive strength of concrete. This indicates that the reduction in concrete compressive strength could increase the tendency of brittle failure of RC beams in flexure. This effect is expected to be more critical in the case of beams having higher values of reinforcement ratios.

The comparison shown in Table 3 between the measured moments and M_{ACI} indicates that all the tested beams sustained flexural strength higher than the design values. The mean to nominal ratios for subgroups RI, RA and RP were 1.40, 1.30 and 1.23 respectively. It can be seen that the measured average moment capacity in subgroup PI was 1.4 times the nominal moment capacity defined by the ACI-318 although the field cured specimens and the core tests showed an average reduction of 15% in the concrete compressive strength for similar type of curing and consolidation used. In the case of beams of subgroup RP the mean to nominal ratio was 1.23, i.e. the average measured nominal strength was greater than M_{ACI} by 23%.

For relative comparison, Allen [13] used the variabilities of basic strength parameters, concrete compressive strength, steel yield strength and cross-sectional dimensions, to estimate the statistical characteristics of the flexural behavior of reinforced concrete beams. He reported that, for reinforced concrete elements, the mean to nominal ratio was in the range of 1.06 to 1.25 and the coefficient of variation was in the range of 0.09 to 0.21, the higher values being for shallow members and poor workmanship.

The increase in the moment capacities of test beams was mainly due to the high observed value of the mean to nominal ratio of yield strength of the Saudi steel which was equal to 1.18 and the effect of the steel strain-hardening which is dependent on the reinforcement ratios [14]. The effect of these two factors were clearly demonstrated in the calculation of M_H, which was very close to the measured moments, when the two factors were considered in the calculation.

CONCLUSIONS

Based on the results of the present investigation and noting that a good quality of concrete with improved workability was used, the following conclusions can be drawn:

1. There was a reduction of up to 15% in the compressive strength of concrete cast in hot and dry climate although an adequate procedures for curing and consolidation were used. A reduction of up to 21% was observed when improper curing and consolidation were used.

2. Improper consolidation and curing in the field may cause a reduction in compressive strength of

concrete in the member more than that allowed in the ACI 318-89 Code.

3. The mean value of moment capacities measured experimentally was noticeably higher than the nominal value calculated under provisions of the ACI-318 Code. This was noted even in the case of field cured beams which showed a significant reduction in the concrete compressive strength. Therefore, a reduction in the compressive strength of field cured cylinders and cores in general, is not indicative of the flexural strength of under reinforced beams in existing structure.

4. Although the reduction in concrete compressive strength had a little effect on the moment capacity of under reinforced beams, it could increase their tendency to brittle failure in flexure. It should also be mentioned that the reduction in the concrete compressive strength due to poor construction practice may have an adverse effect on the durability and serviceability of RC members.

5. The theoretical estimation of the flexural strength of a RC beam was found to be close to the experimental value when the actual properties of Saudi rebars (including strain hardening) and Hognestad model were considered in the calculation.

This demonstrates that the adequacy of flexural strength of the member may be proven by precise analytical investigation which may reduce the need for conducting load tests.

6. Further research similar to this study are recommended to include different qualities of concrete and ratios of tension reinforcing steel than those considered in this study.

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Table 1: Mix Design of Concrete

Materials	Weight kg/m^3
Cement	350
Sand	720
Agregate (10 mm size)	400
Aggregate (20 mm size)	750
Water	174
Retarder (Conplast P509)	0.35

Plasticizer (Cormix SP4)	0.70

Table 2. Compressive strength for core specimens

Beam No.	f_c MPa	f_c/f'_c %	$f_c/\overline{f}_{cs}$ %
Subgroup PI			
PI1	27.86	84	
PI2	27.10	82	
PI3	31.60	96	
PI4	28.65	87	
PI5	29.80	90	
Overall Average	29.00	88	85
Subgroup PN			
PN1	24.62	75	
PN2	24.42	74	
PN3	31.78	96	
PN4	27.37	83	
PN5	26.87	81	
overall Average	27.00	82	79

Note:

f_c = Average strength of five cores taken from individual beam

f'_c = Nominal compressive strength, 33 MPa

$\overline{f}_{cs}$ = Average strength of subgroup CS, 34.34 Mpa

Table 3 Comparison Between Experimental and Nominal Moment Capacities

Beam	Experimental Moment Capacity (kN.m.)		
	Subgroup RI	Subgroup RA	Subgroup RP
1	49.10	45.00	45.80
2	50.70	49.10	42.50
3	50.70	45.80	45.80
4	52.70	47.40	45.80
5	53.10	49.10	42.50
6	52.30	49.10	47.40
7	52.30	45.80	44.10
8	49.10	45.80	47.40
9	50.50	49.10	47.40
10	50.90	49.10	44.10
M_{av} (1)	51.13	47.50	45.30
COV %	2.80	3.67	4.18
M_{av}/M_{ACI} (2)	1.40	1.30	1.23
Reduction % (3)	-	7	12

Note:

(1) M_{av}: Average moment capacity of 10 beams.

(2) M_{ACI}: Moment capacity under the provisions of the ACI-318 Code which equals to 35.73 kN.m.

(3) Reduction % is referred to the subgroup RI.

APPENDIX-NOTATION

f'_c : Nominal concrete compressive strength

f_y : Nominal yield strength

f_{ya} : Average yield strength

f_{sh} : steel strength at strain hardening

E_s : Modulus of elasticity of steel

E_{sh} : Strain hardening slope

ε_{cu} : Ulimate concrete strain

ε_{sh} : Strain hardening strain

ε_y : Yield Strain

Z : Slope of the linear part (Fig.2)

ρ_{max}: Maximum tension reinforcement ratio

Fig.1 : Scheme of the Experimental Work

Fig.2 : Reinforced Concrete Test Beam

Fig.3 : Stress - Strain Relationship for
(a) Concrete and (b) Steel

Fig.4 : Theoritical Relationship Between In-situ Compressive Strength Reduction and Moment Capacity Reduction by Using Hognestad Model

Fig.5 : Standard Deviation Error Bars of Moment Capacity for Subgroups RI, RA and RP.

Fig.6 : Load - Steel Strain Observations for Subgroup RI

Fig.7 : Load - Steel Strain Observations for Subgroup RA

Fig.8 : Load - Steel Strain Observations for Subgroup RP

Region AB

$f_c = f'{}_{,c} \{(2\ \varepsilon_c/\varepsilon_o) - (\varepsilon_c/\varepsilon_o)^2\ \}$

Region BC

$f_c = f'{}_{,c} \{(1 - Z(\varepsilon_c - \varepsilon_o)\}$

Region AB

$f_s = E_s\ .\ \varepsilon_s$

Region BC

$f_s = f_y + (\varepsilon_s - \varepsilon_y)$ (**خطأ!**)

Region CD

$f_s = f_{su} - (f_{su} - f_{sh})$ **خطأ!**