STRENGTH OF CONCRETE AND R/C BEAMS AS AFFECTED BY CURING AND CONSOLIDATION PRACTICE IN HOT WEATHER

By

M. Al-Haddad[1], A. Arafah[1] and A. El-Hindi[2]

ABSTRACT

Reductions in concrete compressive strength and flexural strength of reinforced concrete beams due to inadequate consolidation and curing were tested in this study. The casting, consolidation, and curing were conducted outdoors during very dry and severe 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 plan concrete beams for core testing and thirty standard cylinders. Specimens of the same group were subjected to the same method of consolidation and curing.

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 higher 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.

The reduction in the compressive strength of field cured specimens, in general, is not indicative of reduction in flexural strength of beam members.

---

[1] Assistant Professor, Department of Civil Engineering, College of Engineering, Riyadh, Saudi Arabia.

[2] Graduate Student, Department of Civil Engineering, College of Engineering, Riyadh, Saudi Arabia.

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 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 scientific research on variability in quality of locally produced concrete, effect of local weather conditions and commonly used curing and consolidation methods on concrete quality, and the local level of quality control. The effects of these factors on the concrete quality should be studied in a systematic manner in order to reach economical, practical and efficient recommendation on this aspect.

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 a part of a research being performed to address these problems and improve the reinforced concrete building design practice in the Kingdom of Saudi Arabia.

The American Concrete Institute Building Design Code, ACI 318-89, [2] recommendations limit the allowable reduction in the compressive strength of the concrete in the member by 15%. When this limit is exceeded, the code requires core test or loading test. Both of these tests are usually undesirable, expensive and difficult, and may adversely affect the member strength. The 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, 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 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 very dry and severe 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 with improved workability and at relatively low temperature. It was intended to simulate, as close as possible, the prevalent construction practice in Riyadh city.

RESEARCH SIGNIFICANCE

The study presents findings of experimental work conducted outdoors in sever hot and dry weather of city of Riyadh. Consolidation of concrete in the forms and curing of the tested specimens were varied from poor to acceptable levels of practice. The test results evaluate 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 building is not available.

EXPERIMENTAL PROGRAM

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

1. Group C (Compression Test of Standard Cylinders)

Compressive tests of standard cylinders of three subgroups (CS, CC, and CN) of thirty specimens each. The specimens of the subgroup CS were subjected to standard laboratory curing, the subgroup CC were cured in the field with burlap cover and twice daily water spray, the subgroup CN was cured in the field by sprinkling water only.

2. Group P (Compression Test of Cores)

A total of 50 cores were taken from plain concrete, PC, beams are divided into two subgroups. Twenty five cores of subgroup PI were taken from five PC beams which were compacted mechanically and cured with damp burlap, while the other 25 cores of the second subgroup PN were taken from five PC beams which were rodded and spray cured with water only.

3. Group R (Flexure Test of RC Beams)

RC beams for flexural test were subdivided into three subgroups of ten beams each. The subgroups RI and RA were cast and cured as the first and second subgroups of PC beams. The subgroup RP was very lightly rodded and poorly cured by sprinkling water once a day for three days.

All specimens were tested 28 days after curing. RC beams were tested under two point loads, see Fig. 2, and cylinders and core specimens were tested under direct compression. Figure 2 also show the dimensions and reinforcing details of the test beams. The longitudinal reinforcement ratio was 0.67 ρ_{max} which is assumed to be the most practical value. Concrete was ordered from a ready mix concrete plant in Riyadh city. The mix was designed to obtain 28-day cylinder compressive strength of 33 MPa and average slump of 100 mm at an average temperature of 33^{o}c (at site). The mix design was kept constant throughout the study.

Nominal Moment Capacity

Nominal moment capacity was calculated by different methods as follows:

1. According to ACI assumption employing the Whitney rectangular block [2] for compressive stress in concrete, the nominal moment capacity, M_n, is equal to 35.73 KN.m. The moment capacity is based on concrete nominal compressive strength, f'_c, of 33 MPa, and nominal yield strength, f_y, of 412 MPa (grade 60 Saudi rebar).

2. A more refined nominal moment capacity was evaluated using the following assumptions:

   (a) The relationship between concrete stress and strain is assumed to follow Hognested model [3] shown in Fig. 3(a). Stress-strain distribution in the compressive zone of the concrete was divided into two parts. The first one was assumed to be parabolic up to a strain value of 0.002. The second part is considered to be linear up to an ultimate strain, ε_{su}, 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 [4], 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 rebar [1]. Based on the average of 20 sample tested the following properties were adopted for the stress-strain behavior shown in Fig. 3 (b).

f_y (average value)	=	487 MPa
f_{sh} (average value)	=	515 MPa
f_u (average value)	=	623 MPa
Yield strain ε_y	=	0.0024
Strain hardening strain, ε_{sh}	=	0.004
Modulus of elasticity, E_s	=	200,124 MPa
Strain hardening slope, E_{sh}	=	58,860 MPa

Using a computer program, based upon the above values, the mean moment capacity, 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_n. The parameter f_c is a variable which represents the in-situ compressive strength while f'_c is the nominal compressive strength of 33 MPa. The parameters M and M_n are the bending moment variable and nominal bending moment capacity corresponding to f_c and f'_c respectively.

EVALUATION OF RESULTS

Nominal Compressive Strength

The average compressive strength of thirty specimens from the standard laboratory tests was 34.34 MPa with the standard deviation of 1.43 MPa. According to ACI 318-83 Section 5.3, these values maintain a specified compressive strength of 32.4 MPa which is close to the value of 33 MPa assumed at mix design stage. Also, it was found that the requirements of ACI Section 5.6.2 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 bench mark 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 reduction in strength were mainly due to the field curing because both subgroups were compacted by the same method.

The ACI committee 305 (Hot Weather Concreting) [5] reports that the test specimens molded and cured in air at a temerature of 38°C and 25 percent relative humidity produced only 62 percent strength of that obtained for standard specimens moist-cured at 23°C for 28 days. The strength produced in the test program under more sever weather condition, however, was significantly higher (81 percent of the standard specimens). The limitted reduction in strengths of test specimens may be attributed to the improved workability and relatively low temprature concrete used in the study.

ACI code Section 5.7.3 provides 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 were accepted because ACI requirements mentioned above were satisfied. On the other hand, subgroup CN specimens were not accepted.

ACI Section 5.7.4 recommends core tests if tests of field-cured cylinders indicate deficiencies in curing. This requirement was achieved in this study through PC beams, 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, as shown in Table 1, with average reductions of 15% and 21% as compared with average compressive strength of subgroup CS. It can be noted that there was no additional reduction when a mechanical consolidation was performed in subgroup PI.

The provisions 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 1 that in the case of PI specimen the reduction in the compressive strength observed was within the limit provided by this section, 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 are summarized in Table 2 and compared with the predicted nominal capacity. The mean values of flexural strength of subgroups RI, RA and RP were 51.13, 47.5 and 45.3 KN.m, respectively. The corresponding values of coefficient of variation are respectively 2.8, 3.67 and 4.18 percent. Fig. 5 presents standards deviation eror bars for the three sugroups.

The average reduction in flexural strength of subgroups RA and RP were 7% and 12% respectively when compared with the average capacity of subgroup RI. As indicated by Fig.4 these reductions relatively correspond to 16% and 27% reduction in the in-situ compressive strengths. 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.

The moment capacity predicted using Hognestad Model, M_H, of 53.11 KN.m was in a good agreement with results obtained in case of subgroup RI, 51.13 KN.m. Therefore, one can say that the actual nominal 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 load-steel strain relationship measured during the test for beam subgroups RI, RA and RP respectively. It can be noted that, the plastic deformation of reinforcing steel was the largest for the beams of subgroup RI, the beams of low reduction in concrete in-situ 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 indicated that the reduction in concrete compressive strength could increase the tendency of brittle failure of RC beams in flexural. This effect is expected to be more critical in the case of beams having higher values of reinforcement ratios.

The comparison between the measured moments and M_n indicateed that all the tested beams sustained flexural strength higher than that defined by the code. The mean to nominal ratios for the subgroups, RI, RA and RP were 1.4, 1.3 and 1.23 respectively. It can be seen that the measured average moment capacity in subgroup PI was 1.4 the nominal moment capacity defined by the ACI although the field cured specimens and the core tests showed a reduction of 15% in the average concrete compressive strength for the same type of curing and consolidation used. In the case of beam of subgroup RP the mean to nominal ratios was 1.23, i.e the average measured nominal strength was greater than M_n by 23%.

For comparison, Allen [6] used the variabilities of basic strength parameters, concrete compressive strength, steel yield strength and 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 in the range of 0.09 to 0.21, the higher values being for shallow members and poor workmanship.

The observed overstrength in the beams flexural strength 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 its effect varies depending on the reinforcement ratios [7]. 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 evaluation of the results several conclusions can be drawn as follows,

1. Even where a good quality concrete with improved workability was used, there was a reduction of up to 15% in the compressive strength of concrete cast in hot and dry climate although an adequate procedure for curing and consolidation was used. A reduction of up to 21% resulted when improper curing and consolidation were used.

2. Theoretical evaluation of the reduction in flexural strength of the test beams indicated that, improper consolidation and curing may cause a reduction in compressive strength of concrete in the member higher than that indicated by field specimens cured in a manner similar to that used for the member.

3. The mean value of flexural strengths measured experimentally were noticeably higher than the nominal value calculated under provisions of the ACI code. This was noted even in the case of field cured beams which showed a significant reduction in the core compressive strength. Therefore, a reduction in the compressive strength of field

cured cylinders and cores in general, is not indicative of the strength of flexural members in existing structure.

4. The reduction in concrete compressive strength could increase the tendency of brittle failure of RC beams in flexural.

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.

REFERENCES

1. Arafah, A., et al, "Development of a Solid Foundation for a Local Reinforced Concrete Building Code", The Final Report of a Research Sponsored by KACST, Civil Engineering Department, KSU, 1992.

2. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-89)" and Commentary - ACI 318R-89", American Concrete Institute, Detroit, 1989.

3. Hognestad, E., Honson, N.W. and McHenry, D., "Concrete Stress Distribution in Ultimate Strength Design", ACI Journal, Vol. 52, No. 6, Dec. 1955, p. 455-479.

4. Park R. and Paulay T., "Reinforced Concrete Structures", John Wiley & Sons, Inc. 1975, pp. 50-52.

5. ACI COmmittee 305, " Hot Weather Concreting", ACI-Manual of Concrete Practice, Part 2, American Concrete Institute, Detroit, 1992.

6. Allen, D.E., "Probabilistic Study of Reinforced Concrete in Bending", Technical Paper No. 311,

Division of Building Research, National Research Council of Canada, Ottawa, Jan. 1970, 60 pp.

7. Russo, G., "Beam Strength Enhancement at Design Ductility Factor Demands", ASCE, Journal of Structural Engineering, Vol. 116, No. 12, Dec. 1990, pp. 3402-3416.

Table 1. Mean compressive strength for core specimens

Beam No.	Average* Comp. Strength 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	
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	
Average	27.00	82	79

* Average of five cores for each beam

** $\overline{f_{CS}}$ stands for the mean of strength of subgroup CS.

Table 2 Comparison Between Experimental and Theoretical Moment Capacities

	Experimental Flexural Strength (KN.m.)		
Beam	Subgroup RI	Subgroup RA	Subgroup RP
1	49.1	45.0	45.8
2	50.7	49.1	42.5
3	50.7	45.8	45.8
4	52.7	47.4	45.8
5	53.1	49.1	42.5
6	52.3	49.1	47.4
7	52.3	45.8	44.1
8	49.1	45.8	47.4
9	50.5	49.1	47.4
10	50.9	49.1	44.1
Average	51.13	47.50	45.30
COV %	2.80	3.67	4.18
M_{av}/M_{ACI}	1.4	1.3	1.23
Reduction %*	-	7	12

* Based on subgroup RI

Fig. 1 Scheme of the Experimental Work
Fig. 2 Reinforced Concrete Test Beam

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

$f_s = E_s \cdot \varepsilon_s$

Region BC

$f_s = f_y + (\varepsilon_{sh}-\varepsilon_y)$ (خطأ!)

Region CD

$f_s = f_{su}-(f_{su}-f_{sh})$ خطأ!

Concrete [3]

Steel Reinforcement [1]

Fig.3 Stress - Strain Model for Concrete and Steel

Fig.4 Relationship between in-situ compressive strength reduction and flexural strength reduction by using Hognestad model

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