C. ARUM

EVALUATION OF POST-FIRE RESIDUAL STRENGTHS OF IN-SITU CONCRETE AND STEEL REINFORCEMENT SAMPLES USING NON-DESTRUCTIVE AND DESTRUCTIVE TEST METHODS

By
C. ARUM and P. O. AWOYERA
Department of Civil Engineering

The Federal University of Technology, Akure

P. O. AWOYERA

ABSTRACT

The post-fire residual strengths of in-situ concrete and steel reinforcement samples were evaluated using non-destructive and destructive testing methods. Sixty samples of 320 mm x 150 mm x 100 mm reinforced concrete beams were cast in the laboratory in four batches of fifteen samples each with concrete cover for reinforcement varied at 10 mm, 15 mm, 20 mm and 25 mm respectively for each batch. At 28 days maturity, the beam samples were subjected to laboratory furnace temperatures ranging from 50 °C to 700 °C in steps of 50 °C. Thereafter, the samples were subjected to rebound hammer and Ultrasonic pulse velocity tests after cooling. It was observed that rebound number initially increased from 27 at room temperature to a maximum of 35 at 250°C, representing a 29.6 % increase over the pre-fire value. As temperature increased beyond 250 °C the rebound number decreased continuously up till a value of 12 representing a reduction of 65.7 % at 700 °C. On the other hand, the pulse velocity decreased from 4.302 Km/sec at room temperature to 1.080 Km/sec at 700 °C reducing by 74.9 %. Results of tensile tests on reinforcements extracted from the fire-exposed beams showed decrease of ultimate tensile strength of steel with increasing temperatures, especially for bars with 10 mm concrete cover which at 700 °C lost % of its pre-fire strength.

1. INTRODUCTION

Fires occur in structures with a wide range of different construction systems which use different types of materials and their combinations for their structural members. The heat associated with fires may vaporize trapped concrete pore water. The lack of continuous voids for pressure relief creates internal tensile stresses that are relieved by cracks and spalls extending to the surface, and also that spalling may be explosive in higher strength concretes. Additionally, severe heat may cause chemical changes that lead to micro cracking (visible only under magnification) and loss of strength and integrity (Jeremy, 2009). Furthermore, intense heat may cause chemical reactions that form crystals or change the properties/color of the matrix and/or aggregates in concrete. The changes in concrete (color, surface appearance, and condition) by temperature can be used to estimate the effect of the fire (Ufuk, 2007). However, the users of these concrete structures are often concerned with the strength of concrete as most other properties controlling durability and service performance of concrete are in one way or another related to its strength properties. The strength forms the basis for structural evaluation and possible structural intervention in existing structures which are typically subjected to a different degree of uncertainty than the design of new structures.

There are three stages of assessing the structure to determine what, if any, repairs are required. The stages of evaluation are visual assessment, non-destructive testing and consequent laboratory testing of the residual strengths of concrete and reinforcements (Concrete Society, 2008). Non-destructive tests are done during construction for quality control and monitoring of strength in the long term development and in-service to establish structural adequacy and material deterioration against time or environment. Researchers have therefore devised various non-destructive test (NDT) methods which enable certain properties of concrete to be measured in-situ, from which estimate of concrete strength may be made. This investigation aimed to determine the post-fire in-situ residual compressive strengths of concrete beams subjected to simulated fire in the laboratory furnace at temperatures ranging from 50 °C to 700 °C in steps of 50 °C using the Schmidt Rebound Hammer and the Ultrasonic Pulse Tester, and also the ultimate tensile strength of steel reinforcement embedded in the fire-affected beams and protected using different sizes of concrete cover.

2. FIRE ON CONCRETE

Due to its low thermal conductivity, a layer of concrete is frequently used for fireproofing of steel structures. However, concrete itself may be damaged by fire. Up to about 300 °C, the concrete undergoes normal thermal expansion. Above that temperature, shrinkage occurs due to water loss; however, the aggregate continues expanding, which causes internal stresses. Up to about 500 °C, the major structural changes are carbonation and coarsening of pores. At 573 °C, quartz undergoes rapid expansion due to phase transition, and at 900 °C calcite starts shrinking due to decomposition. At 450-550 °C the cement hydrate decomposes, yielding calcium oxide. Calcium carbonate decomposes at about 600 °C Rehydration of the calcium oxide on cooling of the structure causes expansion, which can cause damage to material which withstood fire without falling apart. Concrete in buildings that experienced a fire and were left standing for several years shows extensive degree of carbonation. Concrete exposed to up to 100 °C is normally considered as healthy as pre-exposure. The parts of a concrete structure that are exposed to temperatures above approximately 300 °C (dependent on water/cement ratio) will most likely get a pink color. Over approximately 600 °C the concrete will turn light grey, and over approximately 1000 °C it turns yellow-brown. One rule of thumb is to consider all pink colored concrete as damaged and should be removed. Fire will expose the concrete to gases and liquids that can be harmful to the concrete, among other salts and acids that occur when gasses produced by fire come into contact with water. (Wikipedia, 2009)

Sometimes, when concrete is exposed to fire, material from the hot fire exposed surface is flaked away in a more or less violent manner. Under some circumstances, the whole cross-section of an element or detail exposed from more than one direction can disintegrate instantaneously, e.g. the web of a beam. (Jansson, 2007).

2.1 Effects on Compressive Strength

When normal-weight concrete cools after a fire, its residual strength varies, depending on the temperature attained, mix proportions, and loading conditions during heating (Bhal and Jain, 1999). For temperatures up to 300° C (572° F), the loss in residual strength often is not cause for concern. Since concrete's pre-fire compressive strength often exceeds design requirements, a modest strength reduction can be tolerated. According to Chakrabarti and Aravind (1999) temperatures greater than 500° C (932° F) can reduce the compressive strength of concrete so much that the material retains no useful structural strength. Concrete that is under compression during a fire loses less strength than concrete that is not carrying loads (Bruce, 1996). He further stated that colour changes can be used to estimate concrete compressive strength, and the estimates can be verified by core removal and testing. Sometimes, rebound hammer and pulse velocity values are correlated with the core tests and then used to survey the structure for likely areas of significant damage.

2.2 Effects on Reinforcing Steel

Cold-worked steel subjected to temperatures of less than 450° C (842° F) typically recovers all of its yield strength after cooling. Hot-rolled steel can be exposed to temperatures as high as 600° C (1,112° F) and recover its yield strength. But higher temperatures may cause significant strength loss in reinforcing steel, and this is usually responsible for any excessive residual deflections of reinforced members. On-site hardness testing also can estimate reinforcement tensile strength and ductility. The hardness of the surface, however, may differ from that at the center of the bar due to quenching when putting out the fire. Whenever possible, hardness test results should be correlated with the actual strength and ductility of steel specimens removed from the structure (Bruce, 1996).

3. NON-DESTRUCTIVE TESTING

According to Narendra, Ray and Dilip (2008), the extent of delamination can be determined by means of chain dragging for large horizontal areas such as slabs, and by means of hammer sounding for vertical and overhead surfaces. Impulse response can be used to rapidly screen large areas for potential damage. Impact echo testing can also be used to determine the depth and extent of internal fractures. Finally, rebound hammers are frequently used to compare the surface hardness of concrete to locate potential damage.

3.1 Rebound (Schmidt) Hammer Method

The rebound hammer test is described in BS 1881 part 202 (1986). The test is classified as a hardness test and is based on the principle that the rebound of an elastic mass depends on the hardness of the surface against which the mass impinges. The energy absorbed by the concrete is related to its strength. The concrete surface to be tested is carefully selected and prepared by using an abrasive stone to grind it smooth. Energy is then applied by pushing the hammer against the test surface. The plunger is allowed to strike perpendicularly to the surface. The angle of inclination of the hammer affects the result. After impact, the rebound number is recorded. At least 10 readings must be taken from each tested area.

Paschale (2003) revealed there is no unique relation between hardness and strength of concrete but experimental data relationships can be obtained from a given concrete. However, this relationship is dependent upon factors affecting the concrete surface such as degree of saturation, carbonation, temperature, surface preparation and location, and type of surface finish. The result is also affected by type of aggregate, mix proportions, hammer type, and hammer inclination. Areas exhibiting honeycombing, scaling, rough texture, or high porosity must be avoided.

3.2 Ultrasonic Pulse Velocity Method

The ultrasonic pulse velocity method is a stress wave propagation method that involves measuring the travel time, over a known path length, of a pulse of ultrasonic waves. The pulses are introduced into the concrete by a piezoelectric transducer and a similar transducer acts as receiver to monitor the surface vibration caused by the arrival of the pulse. A timing circuit is used to measure the time it takes for the pulse to travel from the transmitting to the receiving transducers. The ultrasonic pulse velocity method has been used successfully to evaluate the quality of concrete for more than 7 decades (Hazmeen, 2009). This method can be used for detecting internal cracking and other defects as well as changes in concrete such as deterioration due to aggressive chemical environment and freezing and thawing (IAEA, 2002). By using the pulse velocity method it is also possible to estimate the strength of concrete test specimens and in-place concrete.

4. METHODOLOGY

4.1 Concrete Mix

Sixty samples of 320 mm x 150 mm x 100 mm reinforced concrete beams were cast at the Structures Laboratory of the Civil Engineering Department of the Federal University of Technology Akure. The beams were cast using crushed stone with maximum nominal size of 10 mm as the coarse aggregate, natural river sand as the fine aggregate, and ordinary Portland cement with potable water as the binder. The water - cement ratio used was 0.45 while four 10 mm bars were used as longitudinal reinforcement for each beam. The shear reinforcement consisted of 6 mm diameter bars. A mix ratio of 1:2:4 by volume of cement, fine aggregate and coarse aggregate was adopted, with the concrete cover for reinforcement being varied. Accordingly, fifteen beam samples each were cast for 10 mm, 15 mm, 20 mm and 25 mm concrete cover. The beams were thereafter cured for a period of 28 days in order to ensure adequate maturity before testing.

4.2 Testing of Concrete

After 28 days curing, the beam samples were removed from the water tank and air dried before they were subjected to varying furnace temperatures ranging from 50°C to 700°C at an interval of 50°C in the Physical Metallurgical Laboratory of the Metallurgical and Material Engineering Department of the Federal University of Technology Akure. The beams were allowed to cool and thereafter were subjected to non-destructive tests using the Schmidt rebound Hammer and the Ultrasonic Pulse velocity tester. Four readings of the rebound hammer number were taken on the beams in a diamond pattern as recommended in BS1881 part 202 (1986) and two different readings of the pulse velocity were obtained on the beams using the direct transmission procedure as recommended in BS1881 part 203 (1986). Curves of average Rebound Numbers and the Pulse velocity values versus temperature were plotted using Microsoft Office Excel program.

4.3 Testing of Steel Reinforcement

In order to study the significance of concrete cover for reinforcement at elevated temperatures, the reinforcement steel bars were removed from the beams and later subjected to ultimate tensile strength test using the Universal Material Testing Machine SM 100. The ultimate tensile strength was calculated according to the following formula:

$$f_y = \frac{F}{A} \qquad \text{......................................(1)}$$

f_y is the nominal ultimate tensile strength in N/mm^2
F is the maximum load carried before failure in N, and
A is the original area of the steel sample in mm^2

The tensile strengths of reinforcements were noted at various temperatures with respect to the different concrete covers, and curves of tensile strength versus temperatures were plotted.

5. RESULTS AND DISCUSSION

5.1 Rebound Number and Pulse Velocity

Table 1 shows the rebound hammer numbers and the furnace temperature variation for the beam samples. For the beam tested at room temperature and temperatures range from 50 °C to 700 °C, the average rebound hammer number initially increased from 27 at room temperature to a maximum of 35 at 250°C, and later decreased as the temperature increased towards 700 °C. The curve of

4. METHODOLOGY

4.1 Concrete Mix

Sixty samples of 320 mm x 150 mm x 100 mm reinforced concrete beams were cast at the Structures Laboratory of the Civil Engineering Department of the Federal University of Technology Akure. The beams were cast using crushed stone with maximum nominal size of 10 mm as the coarse aggregate, natural river sand as the fine aggregate, and ordinary Portland cement with potable water as the binder. The water - cement ratio used was 0.45 while four 10 mm bars were used as longitudinal reinforcement for each beam. The shear reinforcement consisted of 6 mm diameter bars. A mix ratio of 1:2:4 by volume of cement, fine aggregate and coarse aggregate was adopted, with the concrete cover for reinforcement being varied. Accordingly, fifteen beam samples each were cast for 10 mm, 15 mm, 20 mm and 25 mm concrete cover. The beams were thereafter cured for a period of 28 days in order to ensure adequate maturity before testing.

4.2 Testing of Concrete

After 28 days curing, the beam samples were removed from the water tank and air dried before they were subjected to varying furnace temperatures ranging from 50°C to 700°C at an interval of 50°C in the Physical Metallurgical Laboratory of the Metallurgical and Material Engineering Department of the Federal University of Technology Akure. The beams were allowed to cool and thereafter were subjected to non-destructive tests using the Schmidt rebound Hammer and the Ultrasonic Pulse velocity tester. Four readings of the rebound hammer number were taken on the beams in a diamond pattern as recommended in BS1881 part 202 (1986) and two different readings of the pulse velocity were obtained on the beams using the direct transmission procedure as recommended in BS1881 part 203 (1986). Curves of average Rebound Numbers and the Pulse velocity values versus temperature were plotted using Microsoft Office Excel program.

4.3 Testing of Steel Reinforcement

In order to study the significance of concrete cover for reinforcement at elevated temperatures, the reinforcement steel bars were removed from the beams and later subjected to ultimate tensile strength test using the Universal Material Testing Machine SM 100. The ultimate tensile strength was calculated according to the following formula:

$$f_y = \frac{F}{A} \qquad \text{.................................(1)}$$

f_y is the nominal ultimate tensile strength in N/mm^2
F is the maximum load carried before failure in N, and
A is the original area of the steel sample in mm^2

The tensile strengths of reinforcements were noted at various temperatures with respect to the different concrete covers, and curves of tensile strength versus temperatures were plotted.

5. RESULTS AND DISCUSSION

5.1 Rebound Number and Pulse Velocity

Table 1 shows the rebound hammer numbers and the furnace temperature variation for the beam samples. For the beam tested at room temperature and temperatures range from 50 °C to 700 °C, the average rebound hammer number initially increased from 27 at room temperature to a maximum of 35 at 250°C, and later decreased as the temperature increased towards 700 °C. The curve of variation of average rebound hammer number with temperature is shown in Figure 1. The R^2 value is found to be 98.7%, which indicates a strong correlation. Table 2 shows the ultrasonic pulse velocity and the furnace temperature variation for the beam samples. For the beam tested at room temperature and temperatures range from 50 °C to 700 °C, the average ultrasonic pulse velocity decreased from 4.302 Km/sec at room temperature to 1.080 Km/sec at 700 °C. The curve of variation of average ultrasonic pulse velocity with temperature is shown in Figure 2. The R^2 value is found to be 98.4%, which indicates a strong correlation.

5.2 Ultimate Tensile Strength of Reinforcement

Tables 3 to 6 show the variation of ultimate tensile strength of reinforcement with temperature for beam samples with 10 mm, 15 mm, 20 mm and 25 mm concrete cover for reinforcements. For beams with 10 mm concrete cover, the average ultimate tensile strength decreased from 592.0 N/mm^2 at room temperature to 224.50 N/mm^2 at 700 °C as shown in Table 3. Figure 3 shows the variation of ultimate tensile strength with temperature. The R^2 value is found to be 99.2%, which indicates a strong correlation.

For beams with 15 mm cover, the average ultimate tensile strength decreased from 592.0 N/mm^2 at room temperature to 272.04 N/mm^2 at 700 °C as shown in Table 4. Figure 4 shows the variation of ultimate tensile strength with temperature. The R^2 value is found to be 98.9%, which indicates a strong correlation. Also for beams with 20 mm cover for reinforcement, the average ultimate tensile strength decreased from 592.0 N/mm^2 at room temperature to 300.97 N/mm^2 at 700 °C as shown in Table 5. The curve of variation of ultimate tensile strength with temperature is shown in Figure 5. The R^2 value is found to be 98.0%, which indicates a strong correlation. Finally for the beams with 25 mm cover for reinforcement, the average ultimate tensile strength decreased from 592.0 N/mm^2 at room temperature to 313.96 N/mm^2 at 700 °C as shown in Table 6. The curve of variation of ultimate tensile strength with temperature is shown in Figure 6. The R^2 value is found to be 98.0%, which indicates a strong correlation. The superposition of Figures 3 to 6 is shown in Figure 7, showing variation of ultimate tensile strength with temperature for beams with 10 mm, 15 mm, 20 mm and 25 mm cover for reinforcement.

6. CONCLUSIONS

This investigation showed that rebound hammer number initially increased with increasing temperature to about 250°C, and later decreased continuously as the temperature increased towards 700 °C. However, the ultrasonic pulse velocity decreased with increasing temperatures. These are shown in Figures 1 and 2.

Furthermore, the significance of concrete cover for reinforcement as investigated using the tensile strength tests on the steel bars removed from the beams showed that the ultimate tensile strength of steel in concrete decreased with increasing temperatures, and in particular, that its value for steel samples from beams with larger concrete cover was greater than that of steel samples from beams with smaller concrete covers, at elevated temperatures See Figure 7).

REFERENCES

Bhal A.S and Jain J.P (1999): "Compressive Strength of Concrete Subjected to Elevated Temperatures. " The Indian Concrete Journal, pp. 503-506.

Bruce A. (1996): "Evaluating Fire-Damaged Concrete" The Aberdeen Group Publication #R970020

British Standards Institution (1986): "BS 1881 part 201" Guide to the use of non-destructive method of test for hardened concrete.

British Standards Institution (1986): "BS 1881 part 202" Recommendations for surface hardness by rebound hammer.

British Standards Institution (1986): "BS 1881 part 203"Recommendation for measurement of velocity of ultrasonic pulses in concrete.

Chakrabarti S.C and Aravind k.J (1999): "Strength Properties of concrete elevated temperature - a review" The Indian Concrete Journal. pp. 495-500.

Concrete Society (2008): "Assessment, Design and Repair of Fire-Damaged Concrete Structures" The Concrete Society, Camberley, technical report 68

Dilek U. (2007): "Comparison of Pulse Velocity and Impact Echo findings to properties of thin Disks from a fire damaged slab "Journal of Performance of Constructed Facilities, pp. 13 - 21.

Ferhat A. and Mehmet S. (2010):"Correlation between Schmidt Hammer and Destructive Compressions Testing For Concretes In Existing Buildings" Scientific Research and Essays Vol.5 (13) Pg. 1644-1648.

Francesco N. and Raffaele P. (2006):"Destructive And Non-Destructive Testing On Reinforced Concrete Structure: The Case Study of The Museum Of Magna Graecia In Reggio Calabria" Department of Mechanics and Materials, Mediterranean University of Reggio Calabria, Italy.

Hazmeen H.M (2009)."Multivariable analysis to determine in-situ concrete strength - column" University of Technology Malaysia
http://www.academicjournals.org/SRE

International Atomic Energy Agency (2002): "Guidebook on non-destructive testing of concrete structures". Vienna, Viewed 2nd December, 2010, <http://wwwpub.iaea.org.>

Jansson R.(2004): "Measurement of Concrete Thermal Properties at High Temperature" Proceedings from the fib Task Group 4.3 workshop, Milan Italy, <www.byggnadsmaterial.lth.se>

Jeremy I. (2009): "Forensic engineering of fire damaged buildings" A journal of the Institution of Civil Engineers (ICE), paper 800040, pg. 12-17, http://www.civilengineering-ice.com

Narendra K., Ray F. and Dilip C. (2008): "Evaluation and repair of fire-damaged buildings" A joint publication of the NCSEA.

Paschale G. (2003): "Nondestructive Assessment of Actual Compressive Strength of Higher Strength Concrete". Journals of Materials in Civil Engineering, pp. 452-459.

Ufuk D. (2007):"Assessment of fire damage to a reinforced concrete structure during construction" Journal of Performance of Constructed Facilities, Vol. 21, No. 4

Wikipedia (2009):"Fire-damage to buildings", Norwegian Building Research Institute, viewed 18 December, 2009, <http://en.wikipedia.org/wiki/Concrete>

TABLES and FIGURES

Table 1. Rebound Hammer Tests on Beams

S/N	Temperature (°C)	Rebound Numbers				Average Rebound Number
		i	ii	iii	iv	
1	0	24	24	28	30	27
2	50	32	32	34	30	32
3	100	30	32	36	34	33
4	150	36	34	32	34	34
5	200	34	32	36	34	34
6	250	36	34	34	36	35
7	300	34	32	36	32	34
8	350	30	28	34	34	32
9	400	26	24	28	28	27
10	450	24	24	22	26	24
11	500	22	20	24	20	22
12	550	20	18	16	18	18
13	600	16	18	20	14	17
14	650	14	16	14	12	14
15	700	12	10	14	12	12

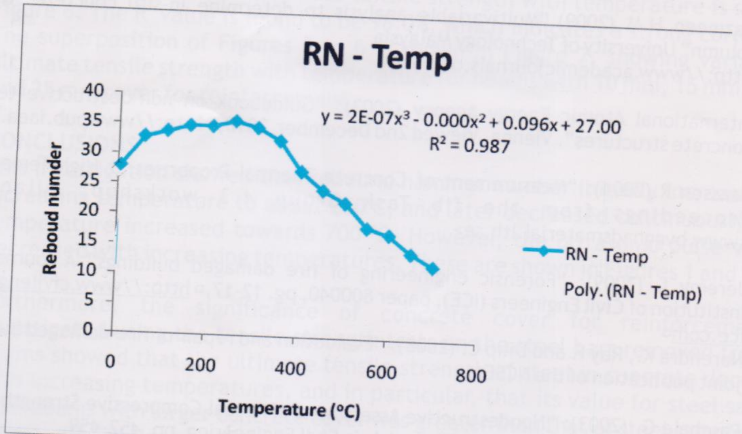

Figure 1. Variation of Average Rebound Hammer Numbers with Temperature

Table 2. Ultrasonic Pulse Velocity Tests on Beams

S/N	Temperature (°C)	Transit Time T (μ sec)		Pulse Velocity = L/T (Km/sec)		Average Velocity (Km/sec)
		i	ii	i	ii	
1	0	23.6	22.9	4.237	4.367	4.302
2	50	24.2	23.9	4.132	4.184	4.158
3	100	25.5	26.0	3.922	3.846	3.884
4	150	26.4	26.9	3.788	3.718	3.753
5	200	27.2	28.1	3.677	3.559	3.618
6	250	29.0	28.6	3.448	3.497	3.472
7	300	31.3	30.8	3.195	3.247	3.221
8	350	39.4	40.0	2.538	2.500	2.519
9	400	45.2	40.9	2.212	2.445	2.329
10	450	45.7	47.2	2.188	2.119	2.153
11	500	51.4	56.1	1.946	1.783	1.864
12	550	60.0	59.2	1.667	1.689	1.678
13	600	75.6	66.7	1.323	1.499	1.411
14	650	84.4	90.2	1.185	1.109	1.147
15	700	90.6	94.7	1.104	1.056	1.080

L = 100 mm i.e. Path length through which pulse velocity was measured on the beams.

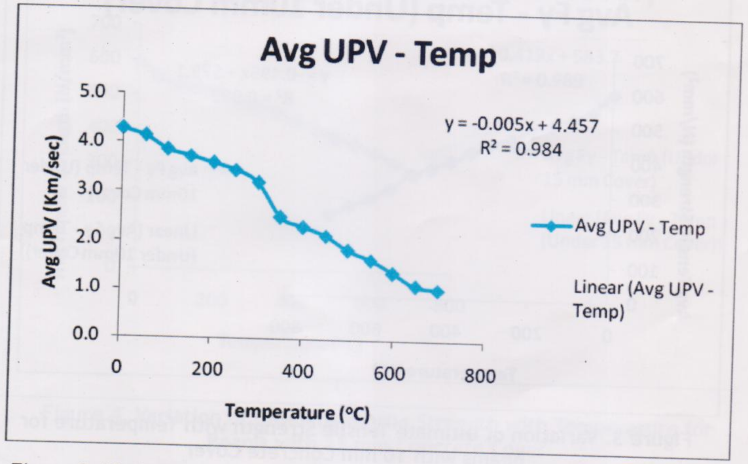

Figure 2. Variation of Average Ultrasonic Pulse Velocity with Temperature

Table 3. Ultimate Tensile Test Results on Bars in Beams with 10 mm Concrete Cover

S/N	Temperature (°C)	Tensile Strength F_y (N/mm²) i	ii	iii	Average F_y (N/mm²)
1	0	579.71	591.20	605.02	592.00
2	50	540.78	554.31	522.61	539.23
3	100	524.56	521.18	505.62	517.12
4	150	498.72	506.09	484.60	496.47
5	200	490.15	480.91	476.43	482.50
6	250	460.41	457.54	447.74	455.23
7	300	451.23	432.42	441.32	441.66
8	350	423.40	411.21	406.90	413.84
9	400	394.70	384.93	377.64	385.76
10	450	376.34	362.14	349.50	362.66
11	500	359.51	341.32	340.62	347.51
12	550	306.55	297.63	304.02	302.73
13	600	276.72	284.76	263.09	274.86
14	650	251.11	262.34	240.68	251.38
15	700	206.96	236.72	229.83	224.50

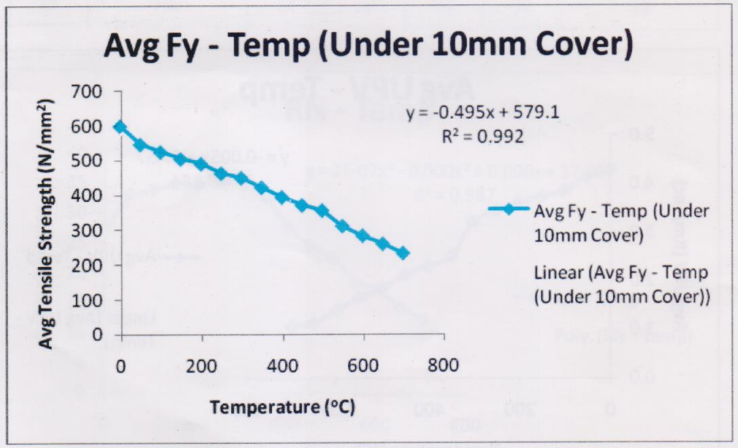

Figure 3. Variation of Ultimate Tensile Strength with Temperature for Beams with 10 mm Concrete Cover

Table 4. Ultimate Tensile Test Results on Bars in Beams with 15 mm Concrete Cover

S/N	Temperature (°C)	Tensile Strength F_y (N/mm^2)			Average F_y (N/mm^2)
		i	ii	iii	
1	0	579.71	591.20	605.02	592.00
2	50	549.23	569.42	530.11	549.59
3	100	532.30	524.51	520.62	525.81
4	150	515.00	521.42	518.11	518.18
5	200	499.45	487.39	512.06	499.63
6	250	477.72	469.46	486.66	477.95
7	300	460.20	472.29	470.06	467.52
8	350	444.17	451.01	456.12	450.43
9	400	420.91	419.23	433.44	424.53
10	450	394.62	401.06	416.31	404.00
11	500	372.24	383.85	369.94	375.34
12	550	364.54	361.04	349.86	358.48
13	600	346.43	337.41	316.17	333.34
14	650	308.12	318.84	286.04	304.33
15	700	284.62	271.33	260.17	272.04

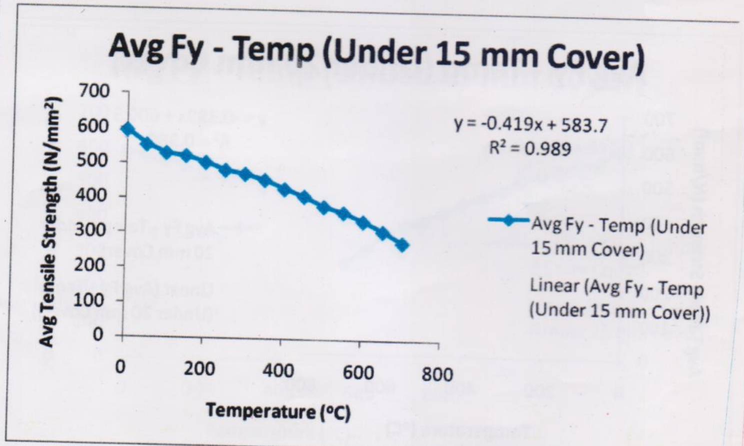

Figure 4. Variation of Ultimate Tensile Strength with Temperature for Beams with 15 mm Concrete Cover

Table 5. Ultimate Tensile Test Results on Bars in Beams with 20 mm Concrete Cover

S/N	Temperature (°C)	Tensile Strength F_y (N/mm^2) i	ii	iii	Average F_y (N/mm^2)
1	0	579.71	591.20	605.02	592.00
2	50	560.53	572.41	550.56	561.17
3	100	559.10	562.52	545.32	555.65
4	150	542.22	539.61	540.10	540.64
5	200	536.12	512.36	539.86	529.45
6	250	532.00	506.71	515.17	517.96
7	300	491.83	488.84	489.52	490.06
8	350	473.52	468.74	475.43	472.56
9	400	468.13	443.43	460.15	457.24
10	450	434.04	424.56	432.35	430.32
11	500	400.24	410.02	417.79	409.35
12	550	382.31	401.5	397.41	393.74
13	600	369.44	378.17	382.33	376.65
14	650	321.52	340.50	336.46	332.83
15	700	299.97	311.23	291.74	300.97

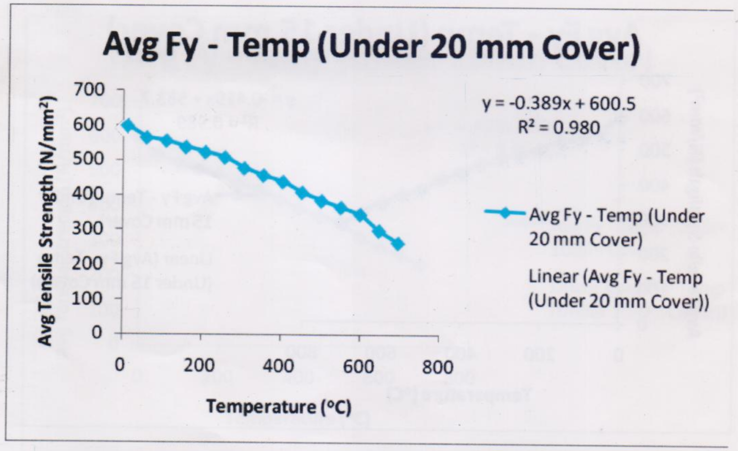

Figure 5. Variation of Ultimate Tensile Strength with Temperature for Beams with 20 mm Concrete Cover

Table 6. Ultimate Tensile Test Results on Bars in Beams with 25 mm Concrete Cover

S/N	Temperature (°C)	Tensile Strength F_y (N/mm²)			Average F_y (N/mm²)
		i	ii	iii	
1	0	579.71	591.20	605.02	592.00
2	50	571.20	588.50	579.20	579.63
3	100	569.30	566.12	574.40	569.94
4	150	549.45	561.40	552.14	554.33
5	200	534.50	541.61	525.24	533.78
6	250	534.00	526.50	522.68	527.73
7	300	515.61	520.80	517.73	518.05
8	350	491.70	502.65	488.36	494.24
9	400	487.52	475.34	469.40	477.42
10	450	461.90	470.93	463.40	465.41
11	500	405.20	442.53	434.60	427.44
12	550	405.00	411.64	406.75	407.80
13	600	396.53	389.60	395.89	394.01
14	650	376.54	366.80	369.72	371.02
15	700	310.44	324.52	306.91	313.96

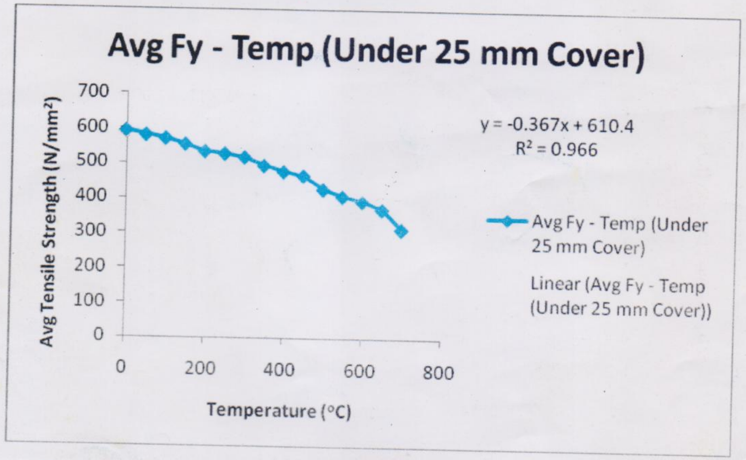

Figure 6. Variation of Ultimate Tensile Strength with Temperature for Beams with 25 mm Concrete Cover

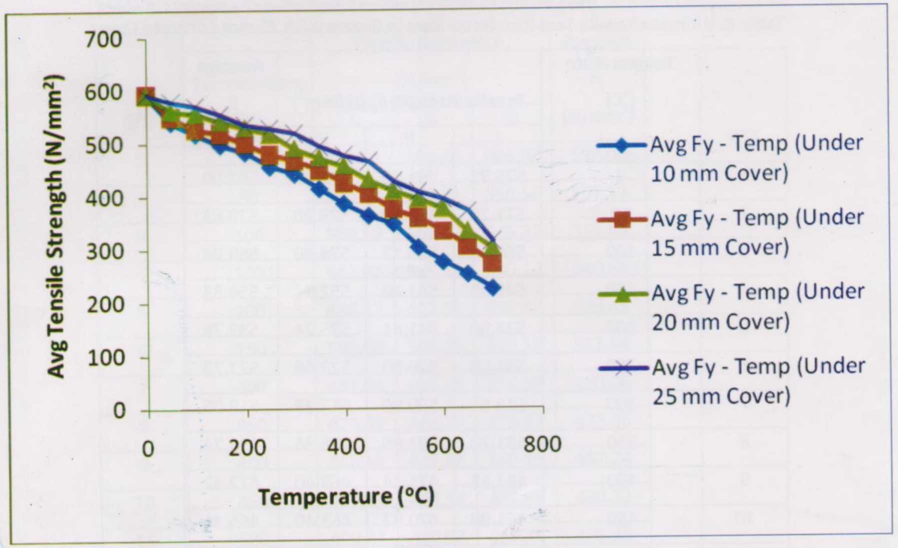

Figure 7. Variation of Ultimate Tensile Strength with Temperature for Beams with 10 mm, 15 mm, 20 mm and 25 mm Concrete Cover

Fig. 4.8: Pundit Ultrasonic pulse tester