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Investigation on Structural Behaviour of R.C.Columns Subjected To Steel Corrosion Under Loading

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Abstract

This paper examines and analyzes the test results produced from a comprehensive test program designed to study the axial behaviour of reinforced concrete columns subjected to steel corrosion. A total of thirty scale model circular columns (150 mm diameter and 300mm height) were tested. The primary variables of this investigation were the sustained loadings, different degrees of steel corrosion and position of longitudinal reinforcement. A direct current was applied to accelerate the corrosion process. Open circuit potential (OCP) and direct current polarization resistance were obtained to evaluate the steel (rebar) corrosion. Axial behavior and residual loading of the columns were measured at the end of the experiment. In addition Non-Destructive Test (NDT) were conducted to assess the condition of corroded concrete. Chloride content in the cover concrete was found out by the standard titration method. Uncorroded columns were used as control samples and tested in parallel with the corroded samples The results indicated that when 8 percent of the mass of steel is corroded the ultimate load carrying capacity is reduced upto 40 percent of uncorroded column under loading condition.

Keywords : Corrosion, Concrete, Cracks, Sustained load, NDT, OCP.

Introduction

Corrosion of steel in concrete is continually growing and causes multibillion-dollar problem. This affects the performance and durability of concrete structures world wide. Corrosion of reinforcing steel in concrete is not only a very costly problem, but it also endangers structural safety[1]. Normally concrete acts as a physical barrier to the ingress of aggressive environment for the reinforcing steel because of its high alkalinity. However, corrosion of the reinforcing steel in concrete may result from the use of inappropriate concrete mixes, lack of quality control in mixing, placing and consolidation and entrapping of air in the concrete, resulting in a relatively permeable concrete. In addition, incorrect use of the different types of cements, supplementary cementing materials, super plasticizers and other commercially available additives has also resulted in deterioration of reinforced or pre-stressed concrete structures because of steel corrosion. Reinforced Concrete columns the main structural members of R.C. structures, usually sustain axial forces of dead loads and live loads[2]. When the structures are exposed to corrosive environment premature failure of R.C. Columns due to corrosion leads to ultimate structural failure. Particularly, with the extensive use of de-icing salt in cold weather regions, key bridge components such as bridge decks and bridge piers are vulnerable to corrosion of steel reinforcement [3,4]. Design inadequacies and substandard detailing practices in these structures are often compounded by the effects of reinforcement corrosion owing to carbonation or to chloride exposure in marine environments[5]. Although some experimental work has been undertaken to investigate structural behavior of Beams and slabs. in corrosive environment by the researchers, literature on corrosion study on column is extremely limited.

Pantazopoulou[6] et.al investigated the effectiveness of FRP wraps in inhibiting the corrosion process of steel reinforcement embedded in columns. Tastani[3] et al., investigated the structural behaviour of corrosion – damaged columns with substandard detailing of reinforcement. Han-seung lee[2] et.al carried out an experimental study on the retrofitting effects of reinforced concrete columns damaged by rebar corrosion strengthened with carbon fiber sheets. Bousias[7] et.al studied the seismic behavior of rectangular underdesigned columns wrapped with CFRP and GFRP and the effect of corrosion was also studied.

Tests have also been performed to assess the possibility of preventing the failure of the column due to lap splices. Lap splices in reinforced concrete columns typically consists of bars overlapped and placed in contact with each other. In the case of multistoried structures It is not possible to have the longitudinal bars to be continuous nor is it possible to provide standard lap splice. David et al investigated the behavior of non contact lap splices in bridge column shaft connections[8].

Chung[9] et al. demonstrated that glass FRP (GFRP) confinement could be able to avoid the lap splice failure on circular columns. Haroun[10] et al analyzed the lap splice study on both circular and rectangular half-scale column confined with either CFRP or GFRP laminates. This was also confirmed by tests on large- scale square columns confined with CFRP by saatcioglu and Elnabelsy[11].

With this background it is essential to clarify the influence of steel corrosion on the strength of an R.C. column in order to ensure its structural performance. This research involved the testing of two series (AR1 and AR2) of reinforced concrete column subjected to three levels of corrosion under non-loading condition. Another set of AR1 and AR2 specimens were subjected to reinforcement corrosion while maintaining a sustained load.

Experimental investigations

Materials used and their properties

Ordinary Portland cement (OPC) 53 grade confirming to IS 12269(1987)[15] obtained from a single source having a specific gravity of 3.15 was used in this study. Locally available river sand which falls in zone III of IS 383- 1970[16], and with a specific gravity of 2.55, fineness modulus of 2.81 were used as fine aggregate. The crushed blue granite having a specific gravity of 2.69 were used as coarse aggregate.

Mix proportions

A mix M30 grade was designed as per IS 10262-1981[17] and the same was used to prepare all the column specimen and other test samples. The design mix proportion is shown in Table.1.

Specimen preparation

Thirty reinforced concrete columns (scale model[6]) of size 150 mm diameter and 300mm height were cast with a clear cover of 20mm using two different arrangements of longitudinal reinforcement (1.78 percent and 2.66 percent)were prepared. The specimen were labelled as two series, each comprising 15 specimens referred to herein as AR1and AR2. AR1 specimens were reinforced with straight bars of 4 Nos - 10mm diameter high yield strength deformed bar (HYSD) as longitudinal bars and lateral ties of 6 mm diameter with spacing as shown in Figure.1(a). AR2 specimens were reinforced with straight bars of 2 Nos -10mm diameter (P and R), cranked bars (detailed as per SP 34 (S &T) – 1987)[24] of 2Nos - 10mm diameter(Q and S) as longitudinal bar and lateral ties of 6 mm diameter with different spacing as illustrated in Figure-1(b). In the case of bars overlapped, cover to reinforcement was adopted same as that of straight bars. (P and R)

The high yield deformed bar with yield strength of 415 MPa was used for both longitudinal and lateral ties. The weight of reinforcement cage was taken before introducing into the concrete.

Casting

The casting was done in ten batches of concrete mix and for each batch of concreting cubes, cylinders and prisms were cast (as per IS 516 -1959)[18] along with the column specimens for the determination of compressive strength, split tensile strength and flexure strength. The strength results are given in Table 2.

Method of Accelerating Reinforcement Corrosion

Galvanostatic method[12] was used to accelerate the corrosion of the reinforcement bar. The three different levels of corrosion are studied at 30, 60 and 90 days and are designated as first, second and third level respectively. Three identical specimens of each series were subjected to three levels of corrosion under non-loading condition. As a second phase of the work another set of AR1, AR2 specimens were subjected to corrosion process under loading condition. A load of 45kN[12] was applied to the specimens. The loading was exerted by keeping two 2.5cm thick steel plates one at the top and the other at the bottom of the specimens and the whole assembly is tightened with threaded rod and nuts which was controlled by a torque wrench. Figure.2. gives the schematic arrangement of the sustained load corrosion process. The load decreases as the corrosion proceeds (Creep effects) with increase in time. To maintain the loads constant throughout the duration of the test, continuous monitoring and adjustment of nuts were required. The steel plates, nuts and bolts were coated with Zinc – copper alloy to avoid the galvanostatic corrosion. Figure.3 and Figure.4 shows the specimens after the accelerated corrosion process.

Results and Discussions

Extent of Corrosion

The corrosion process was carried out at three levels (AR1C1, C2,C3 and AR2C1, C2, C3specimens) for the two different arrangements of longitudinal reinforcement. Upon removal of the reinforcement after the ultimate load test, procedure outlined in ASTM G1 – 67[23] was used to determine final extent of corrosion. The results are presented in Table 3. It can be seen that the R1 specimens carries a higher percentage of weight loss than R2 specimens at all levels of corrosion under non-loading condition. This is due to the increased electrical resistance of R2 specimens owing to the larger overall amount of reinforcing steel. (greater anodic area leading to lower corrosion current density for a given current) It is also possible that the expansive forces resulting from buildup of corrosion products were partially resisted by the bonding action of the overlapping reinforcement. The increased percentage weight loss noted in the AR1 series specimens C1, C2, C3 and C4 are 0.431%, 0.813%, 1.312% and 1.018% respectively with respect to the corresponding AR2 series specimens.

Under Loading Condition

The simultaneous load and accelerating corrosion process of the columns was carried out for the AR1C4 and AR2C4 specimens. For the same level of corrosion the increased percentage weight loss noted in the AR1C4 is 5.096percent with respect to AR1C1, while 5.319 percent is noted in AR2C4 with respect to AR2C1. This is due to the sustained loading and corrosion process deteriorate the bond strength at the steel / concrete interface and thus increased the cracking of the specimens leading to higher extents of corrosion.

Distribution of cracks and corrosion activities

Figure.5. gives overall views of cracking in the specimens observed after carrying out electrolytic corrosion. The transverse cracks were more serious with many inclined branches. The pattern of the cracks was irregular. Both crack width and number of cracks increases as the level of corrosion increases. The width of crack increases which indicate a decrease in the bond capacity between the corroded steel and concrete. According to visual observations carried out on both longitudinal and hoops after the destructive load test, the steel had black rust caused by the electrolytic corrosion all over their surfaces and partial losses of the sectional area particularly notable was AR1C3 specimen. In the third level of corrosion, some of the transverse steel lost more than half of their sectional area. It is given in Figure.6. The reason for this seems to be that the proximity of the stirrups to the exposed later surface they are to be first affected by corrosion and as it is usually fabricated from small diameter bars.

Residual axial load test

Axial compression test were conducted after completion of the accelerated corrosion process, and the obtained failure load and failure modes are summarized in Table 3. Axial strain was measured using demountable mechanical strain gauge which is positioned at diametrically opposite sides of the specimen. Figure.7 and Figure.8. shows the test set up of conventional and corroded columns. The failure of conventional columns occurred due to the cracking and spalling of the concrete cover. Figure. 9. shows the failure pattern of conventional column. However, it was noticed that the spalling of the concrete cover of the corrosion damaged columns occurred along the height of the columns (Figure. 10. shows the state of failure of the corroded specimen) almost at the same time, showing significant loss of failure load as compared to the conventional column. This is probably because the concrete cover of the corroded column were already delaminated , prior to the failure tests, due to the cracks formed around the spiral reinforcement and cross-sectional loss of steel reinforcement. From the Table 3 it can be seen that R2 specimens which has higher load carrying capacity than R1 specimens because of increased percentage of area of reinforcement. For 33 % increase in the area of reinforcement the percentage increase in load in AR2UC is 6.02% than AR1UC specimen. The percentage decrease in the series AR1C1, AR1C2, AR1C3 and AR1C4 are 15.38%, 29.10%, 39.48%and 45% respectively with respect to control specimen, whereas those noted in the series AR2C1, AR2C2, AR2C3 and AR2C4 are 12.40%, 26.51%, 35.06%, and 40 %respectively with respect to control specimen. For the same level of corrosion the percentage decrease in the load noted in AR1C4 is 35 % with respect to AR1C1, whereas 31.49% is noted in AR2C4 with respect to AR2C1.

Stress- strain Response

Figure.11 represent stress - strain diagrams for conventional and corroded columns at different levels. Examination of stress- strain curves clearly shows, that corroded columns give higher strain value at all the increment of axial load. It is difficult targets to predict the strain values of corroded concrete using the demountable strain gauge at higher values of loading, since the pellets which were fixed to the concrete surface fell off due to the spalling of concrete cover. It can be noticed through the R1 specimens where the maximum strain corresponding to a load of 590 kN could be measured, even though the specimen was subjected to a maximum load of 660kN. The same trend has been noticed for all other specimens in R1 and R2 series.

Effect of Corrosion Behavior

According to ASTM C 876 – 91[20] When OCP is higher than –127 mV , the probability ere is less than 10 % that reinforcing may corrode. If the potential is from -127 to -276 mV (SCE – Saturated Calomel Electrode), corrosion probability is uncertain. Corrosion probability may be higher than 90% for OCP between -276 and -427 mV (SCE). Corrosion test results as shown in Figure.12. indicates that without sustained loading, (in AR1C1specimen) the OCP drops to - 276 mV after 14 days accelerated exposure. With 20 % loading (in AR1C4) the OCP drops to -276 mV after 8 days accelerated exposure. Whereas those noted in the series AR2C1 and AR2C4 are 16days and 10 days, which reveals that the sustained loading or cracks plays paramount in the corrosion process of reinforcing steel in concrete. The potential value measured at a well defined grid points in AR1C4 and AR2C4 specimens is given in Figure. 13.

Along with OCP observations, the polarization resistance was obtained for computing the corrosion current density. The corrosion current density is estimated from Stern-Geary equation[13]. Figure.14. demonstrates the relationship between OCP and corrosion current density measurements from the present research. It is seen that the OCP readings increased with corrosion current density. The value observed at a well defined grid points in AR1C4 and AR2C4 specimens is given in Figure. 15.

The non-uniformity of corrosion current density in the reinforcement (P-Q-R-S) was due to the variations in permeability of concrete and availability of oxygen to continue the corrosion process.

Non- Destructive Test.

Non-Destructive testing (NDT) such as Rebound Hammer Test and Ultrasonic Pulse Velocity value(UPV) were conducted to assess the condition of corroded concrete. Rebound test (as per IS 13311(Part – II)[22] was conducted of each specimen once in a weak and the graphs were plotted between OCP Value and Rebound number. It is given in Figure.16. The average value recorded at the end of the test programme is presented in Table 3. It can be seen that conventional specimens has a rebound number of 40 which indicates the cover concrete is good. For the corroded specimens the value was in the range of 20 – 34. Locations possessing very low rebound numbers identified as weak surface concrete and such locations are defined as corrosion prone locations. The value observed at a well defined grid points in AR1C4 and AR2C4 specimens is given in Figure. 17.

The Ultrasonic Pulse Velocity value(UPV) (as per IS 13311(Part – I)[21] measured at the end of the test programme is presented in Table 3. It can be seen that conventional specimens has a higher UPV than corroded specimens which indicates that there is no loss of integrity. Low velocity indicate a really weak surface concrete. For the corroded specimens the UPV was in the range of 3.0 -3. 9 km/ sec which may be due to initiation of delamination of outer portion of concrete. (cover concrete) The value observed at a well defined grid points in AR1C4 and AR2C4 specimens is given in Figure. 18.

Chloride content

Chloride content can be determined from broken samples or core samples. Primarily, the chloride content upto the cover thickness is of prime importance. This was conducted by standard titration method[14]. The threshold value of chloride content in concrete to initiate corrosion is generally considered as 0.40 – 0.60 % (IS 456 - 2000)[19] by weight of cement. Table.3. gives the chloride content of the cover concrete ranging from 0-20 mm depth. It can be concluded that the corrosion of reinforcement was caused by the high chloride content. The value observed at the end of the test programme in AR1C4 and AR2C4 specimens is given in Figure. 19.

Conclusions

Based on the results reported in this paper and the observations made during the experimental investigation, the following conclusions are drawn.

This paper explored the behaviour of R.C. Corroded columns under loading and non-loading condition. For this purpose an extensive experimental study was conducted on several small-scale concrete columns with two different arrangement of longitudinal reinforcement such as R1 and R2. After the corrosion process the R1 series of specimens was more severe damage than in R2 specimens, most likely because the electrical resistance of the R2 specimens was greater due to overall amount of reinforcing steel. (greater anodic area leading to lower corrosion current density for a given current)

The causes for the decline in the strength of an R.C. column with corroded rebars (steel) are the decline in the mechanical property of rebars due to corrosion are loss of bond between rebars and concrete and decline in the confining effect of concrete due to the falling of concrete cover.

This paper also provides some non-destructive electrochemical tools that are commonly used in corrosion investigations. The condition assessment gives the condition of structure required for repairs and rehabilitation structural audit for strength, stability and durability of structure.

The results of this investigation show that, in order to better understand the effects of corrosion on structural performance, it is necessary to conduct such tests under loading corrosion condition.

The formation of cracks and the possibility of extent such crack is likely to be dependent on aspects such as member geometry, cover depth, steel bar diameter and amount of reinforcing steel

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Table 1: Mix proportions of the concrete used for preparing the columns

Sl.No	Ingredient	kg / m^3	Proportion
1	Portland Cement	450	1: 1.12 : 2.687 W/ C = 0.425
2	Water	191.6	
3	Fine Aggregate	504	
4	Coarse Aggregate	1209	

Table 2: Strength Results For The Concrete

Sl.No	Description	Value in MPa
1	Characteristic Compressive Strength	43
2	Split Tensile Strength	4.8
3	Modulus of Rupture	5.98

Table 3: Test results of the column

Sl. No	Nomenclature	Failure Load in KN	% Weight Loss	Chloride Content 0- 20 mm	Rebound Hammer Value	Ultrasonic Pulse Velocity km/ sec	Mode of Failure
1	AR1UC	780	-	-	40	5.65	Yielding/ concrete crushing
2	AR1C1	660	3.050	4.02	28	3.90	Early Yielding / Concrete crushing
3	AR1 C2	553	6.200	5.21	26	3.50	“
4	AR1 C3	472	8.759	5.47	22	3.45	“
5	AR1C4	429	8.956	5.61	20	3.25	“
6	AR2 U C	830	-	-	40	5.72	Yielding/ concrete crushing
7	AR2 C1	727	2.619	4.01	30	4.10	“
8	AR2 C2	610	5.319	5.11	27	3.89	“
9	AR2 C3	539	7.447	5.32	25	3.68	“
10	AR2C4	498	7.938	5.52	22	3.31	“

AR1UC and AR2UC – Uncorroded specimens.

AR1C1 to AR1C3, AR2C1 to AR2C3 Corroded specimens under non loading Condition

AR1C4 and AR2C4 Corroded specimens under loading Condition

Figure 3: Scale Model Columns After The Accelerated Corrosion Process

Figure 4: Scale Model Columns After The Accelerated Corrosion Process Under Loading Condition

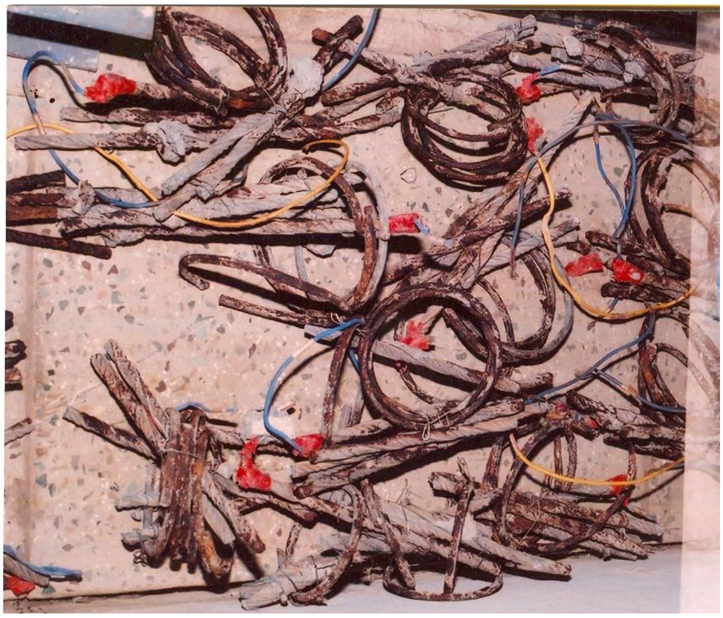

Figure 6: Reinforcement (Rebar) corrosion

Figure 7: Concentric Axial Load Test Conventional Specimen

Figure 8: Concentric axial load test Corroded specimen

Figure 9: Failure mode of Conventional specimen

Figure 10: Views of failure of the specimen.

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